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Satyajit Patel
C. H. Solanki
Krishna R. Reddy
Sanjay Kumar Shukla *Editors*

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Preface

The Indian Geotechnical Society, Surat Chapter, and Sardar Vallabhbhai National Institute of Technology (SVNIT), Surat, India, organized the Indian Geotechnical Conference (IGC) in Surat during 19–21 December 2019. The main theme of the conference was "GeoINDUS: Geotechnics for INfrastructure Development and UrbaniSation". The sub-themes of the conference included:

1. Characterization of Geomaterials and Physical Modelling
2. Foundations and Deep Excavations
3. Soil Stabilization and Ground Improvement
4. Geoenvironmental Engineering and Waste Material Utilization
5. Soil Dynamics and Earthquake Geotechnical Engineering
6. Earth Retaining Structures, Dams and Embankments
7. Slope Stability and Landslides
8. Transportation Geotechnics
9. Geosynthetics Applications
10. Computational, Analytical and Numerical Modelling
11. Rock Engineering, Tunnelling and Underground Constructions
12. Forensic Geotechnical Engineering and Case Studies
13. Others Topics: Behaviour of Unsaturated Soils, Offshore and Marine Geotechnics, Remote Sensing and GIS, Field Investigations, Instrumentation and Monitoring, Retrofitting of Geotechnical Structures, Reliability in Geotechnical Engineering, Geotechnical Education, Codes and Standards and other relevant topics.

The proceedings of this conference consists of selected papers presented at the conference. The proceedings is divided into six volumes, including a special volume with all keynote/invited presentations.

We sincerely thank all the authors who have contributed their papers to the conference proceedings. We also thank all the reviewers who have been instrumental in giving their valuable inputs for improving the quality of the final papers. We greatly appreciate and thank the student volunteers, especially Vemula Anand Reddy, Mohit Mistry, Rahul Pai, Manali Patel, Rohan Deshmukh, Hrishikesh Shahane, Anand M. Hulagabali, Jiji Krishnan and Bhavita Dave, for their unwavering support that was

instrumental in preparation of this proceedings. Finally, thanks to the Springer team for their support and full cooperation for publishing six volumes of this IGC-2019 proceedings.

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Surat, India — Satyajit Patel
Surat, India — C. H. Solanki
Chicago, USA — Krishna R. Reddy
Joondalup, Australia — Sanjay Kumar Shukla

Contents

About the Editors

Dr. Satyajit Patel is Associate Professor at the Civil Engineering Department, Sardar Vallabhbhai National Institute of Technology, Surat, India. His research area includes utilization of industrial solid wastes in civil engineering constructions, geoenvironmental issues, soil stabilization, ground improvement, and geosynthetics for road pavements. He has published 13 journal papers and presented 7 research papers internationally. He has guided more than 29 M.Tech. students and 9 students are currently pursuing their Ph.D. under his guidance. He is a life member of Indian Geotechnical Society, Institution of Engineers (India), and Indian Road Congress (IRC). He has received a research grant from the Department of Science and Technology, Government of India.

Dr. C. H. Solanki is Professor (Geotechnical Engineering) at the Civil Engineering Department, Sardar Vallabhbhai National Institute of Technology, Surat, India. He has guided 50 postgraduate dissertations and 16 Ph.D. thesis and he is currently supervising 11 Ph.D. research scholars. He has published 165 research papers in the reputed national, international journals and conferences. Dr. Solanki received the "Shri. M. S. Jain Biannual Award" for the Best Paper in IGC 2013, Prof. Dinesh Mohan Award in IGC 2017 and "Distinguished Faculty Award" form The Venus International Faculty Awards-2016. He has organized 20 national level events including STTP, FDP, conferences and seminars and he was the chairman for the Indian Geotechnical Conference (IGC 2019) held at Surat, Gujarat. Dr. Solanki has been elected as a member of executive committee of IGS for the terms (2015–2022). His research interests include subsoil characteristics, predictions in geotechnical engineering, soil dynamics, ground improvement and geoenvironmental engineering. He has given 50 expert talks throughout India and abroad. He is a life member of Indian Geotechnical Society, Indian Society for Technical Education (ISTE), and Institution of Engineers (India).

Dr. Krishna R. Reddy is Professor of Civil and Environmental Engineering, Director of Sustainable Engineering Research Laboratory, and also Director of the Geotechnical and Geoenvironmental Engineering Laboratory at the University of Illinois at Chicago, USA. He received his Ph.D. in Civil Engineering from the Illinois Institute of Technology, Chicago, USA. Dr. Reddy has over 28 years of teaching, consulting, and research experience in the fields of civil engineering, geotechnical and geoenvironmental engineering, environmental engineering, and sustainable and resilient engineering. He is the author of 4 books, 246 journal papers, 21 edited books/conference proceedings, 22 book chapters, and 225 full conference papers. He has received several awards for excellence in research and teaching, including the ASCE Wesley W. Horner Award, ASTM Hogentogler Award, UIC Distinguished Researcher Award, the University of Illinois Scholar Award, and the University of Illinois Award for Excellence in Teaching.

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Behaviour of Square Footing on Cement Modified Fibre Reinforced Sand Layer Underlain by Soft Clayey Soil

Banchiva K. Marak and Nirmali Borthakur

Abstract Deep foundations are generally suggested when soil at the site is weak soft clayey soil. The cost of construction of deep foundations like piles and piers is generally very expensive. However, bearing capacity of weak foundation soil can be improved by various soil stabilization methods so that it becomes suitable for shallow foundation. In present investigation, a layer of sand modified with 5% cement and reinforced with 0.75% polypropylene fibre was placed on top of soft clayey soil. Optimum percentage of cement and fibre required was first determined from proctor compaction tests and direct shear tests. Three different percentages of cement as 2, 3.5 and 5% and four different percentages of fibre as 0.25, 0.5, 0.75 and 1% were used for the study. Laboratory modelling was carried out on steel tank with square concrete block of size 20 cm $\times$ 20 cm and thickness 4 cm used as footing. Loads were applied by hydraulic jack and recorded in proving ring. Two dial gauges were used in diagonally opposite direction to measure settlement. Load-settlement behaviour of footing was studied by varying the ratio of thickness of top sand layer (H) to the width of footing (B) (as $H/B = 0$, 0.5, 1, 1.5 and 2) for both unreinforced and cement modified fibre reinforced sand layer underlain by soft clay. Optimum H/B ratio for cement modified fibre reinforced sand layer was found to be 1. However, for unreinforced sand layer, the H/B ratio was found to be significantly higher.

Keywords Soft clayey soil · Polypropylene fibre · Cement modified fibre reinforced sand layer

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1 Introduction

Existing soil in the given site for construction may not always be acceptable for the intended purposes. Different types of problematic soils in the sites are inevitable and avoiding the site not suitable for construction is not a good option anymore. There has been an advancement in the geotechnical engineering area, which has provided today a solution for tackling numerous problems of soil by various new techniques like soil stabilization and ground improvement techniques. In order to provide a solution in the most economical and safest way possible, many researches are being done in various areas. Soil stabilization proved as an effective and promising method in significantly improving the geotechnical soil properties.

When the foundation soil at the site is a soft clayey soil and is very weak to support a shallow foundation, deep foundations are generally suggested. However, deep foundations are expensive and sometimes become cost effective only when the structures to be supported are quite huge and heavy. There has been various developments in this research field, that goes beyond only piles and piers and are pushing the industry forward with their new alternative solutions so that a simpler and less costly shallow foundation can be used instead of deep foundations. Therefore, in some cases, the foundation soil bearing capacity improvement can be done by various new ground improvement methods and make it suitable for shallow foundation. Practically, placing a stronger sand layer of proper thickness on top of weak soft clay can improve the bearing capacity [1]. Many researches are being done relating to inhomogeneous layer of soil bearing capacity, but relating to this problem detailed design information is still lacking. Many studies are going on to modify the geotechnical properties of poorly graded sandy soil which is just locally available, using various soil stabilization methods like cement, lime, fly ash, bitumen, natural and artificial fibres. Stabilization of soil using cement is popular and attractive method having both soil and rock characteristics [2]. Cement modified soil is one type of soil cement which contains cement percentage of 5% or less [2]. Adding cement in proper percentages increases soil capacity to bear and reduces displacement which are two key parameters needed in the design considerations in geotechnical engineering problems. However, the artificially cemented soil exhibits a noticeably brittle behavior at peak shear strength at higher percentages of cement which causes sudden failure of soil structures [3]. Hence, its usage is not allowed at shallow depths. However, this problem of sudden failure due to brittleness can be overcome by addition of reinforcing materials like fibres to artificially cemented sand which provides ductile characteristics to the soil at greater peak shear strength [3].

2 Materials Used

2.1 Study Materials

The materials necessary for the present study are as follows:

Clay. Clayey soil used for the study was collected from the paddy fields of Bariknagar. Wet sieving was done for 150 g of the sample in which 91–93% of the soil pass through 75 μm sieve, and for further analysis, hydrometer tests were done on 50 g of soil and the results show 44.02% clay and 53.73% silt. In accordance to Indian Standard Classification System (ISCS), the soil is classified intermediate plasticity clay (CI) (Table 1).

Sand. Sand used for the study was locally available, from NIT Silchar. In accordance to the Unified Soil Classification System (USCS), the soil was poorly graded sand (SP) (Table 2).

Cement. Cement used for the study is a Portland cement. In accordance to ASTM C150 (2007), the Portland cement was classified as type II.

Fibre. The type of fibre used for the study was an artificial fibre called polypropylene fibre. The unit weight of this fibre is 8.93 kN/m^3 with average diameter and length of 0.034 mm and 12 mm, respectively (Fig. 1).

Table 1 Properties of clay

Properties	Values
Liquid limit	40%
Plastic limit	22%
Plasticity index	18%
Shrinkage limit	13%
Specific gravity	2.64
Moisture content during model tests	29%
Unit weight during model tests	17.8 kN/m^2
Gravel	0%
Sand	2%
Silt	53.73%
Clay	44.02%
IS classification system	CI group

Table 2 Properties of sand

Properties	Values
Effective grain size	0.15 mm
D_{30}	0.20 mm
D_{60}	0.23 mm
Coefficient of uniformity (C_u)	1.53
Coefficient of curvature (C_c)	1.16
Optimum moisture content (OMC)	16.5%
Maximum dry density (MDD)	1.63 g/cc
Angle of internal friction (ϕ)	31°
Cohesion (c)	0.015 kg/cm^2
Specific gravity of solids (G)	2.74

Fig. 1 Polypropylene fibre

3 Experimental Investigation

3.1 Methodology and Testing Programme

Standard Proctor Compaction Tests. Standard proctor compaction tests were done in accordance to Indian Standard Code IS: 2720 (Part VII) at three cement percentages (2, 3.5 and 5%) and four fibre percentages (0.25, 0.5, 0.75 and 1%). For sand–cement mixture, the required first cement percentage was mixed thoroughly with the oven dried sand till uniformity in colour was achieved. For process of mixing and compaction, required water is added. The bulk density is calculated from compacted specimen, and hence MDD and OMC are found out and the same is repeated for each cement percentage. Optimum cement is found out. Similarly, for sand–cement–fibre mixture, the optimum cement percentage was mixed thoroughly with the oven dried sand till uniformity in colour was achieved and then required first fibre percentage is added and mixed properly with required water. Then, the MDD and OMC are found out. The same is repeated for each fibre percentage.

Direct Shear Tests. Conventional direct shear tests apparatus was used to perform direct shear tests. 0.5, 1 and 1.5 kg/cm^2 were normal stress considered. Tests were performed by making cube specimens of size slightly smaller than 6 cm × 6 cm × 3 cm at maximum dry unit weight and optimum water content obtained previously from standard proctor compaction tests. The required amount of cement and fibre mixed is calculated based on the sand dry weight. First, for sand–fibre mixture at four fibre percentages (0.25, 0.5, 0.75 and 1%); the fibre is mixed directly with sand thoroughly with optimum water. Secondly, for sand-cement–fibre mixture at optimum cement determined from compaction tests and four fibre percentages, the sand and optimum cement mixture was made thoroughly first so that a uniform colour is acquired and then the fibre is added and mixed well with desired water. In three layers equally, the specimen was statically compacted. Specimen curing for seven days was done and the tests were conducted. The soil characteristic shear strength determined. The optimum fibre percentage is determined (Fig. 2).

Experimental Modelling. Experimental test was carried out in Geotechnical Laboratory at NIT Silchar with the help of a steel tank having size 100 cm (length) × 100 cm (width) × 100 cm (height) with square reinforced concrete block of size 20 cm × 20 cm and thickness 4 cm as footing and a steel plate of same size was placed over it to distribute the load equally. First for preparing a layer of clay bed, oven dried clay was pulverized properly and then batches of dried clay selected randomly were thoroughly mixed with calculated water content and density achieved by hit and trial method from unconfined compressive strength tests, to make a clay bed of desired soft consistency. The layer of clay bed of required density was made by dropping freely about 5 kg mass of lumps of clay from height of about 1 m. With the help of

(a) (b)

Fig. 2 **a** Sample preparation for direct shear tests, **b** stabilized specimen failure at the end of direct shear tests

Fig. 3 **a** Model tank with loading arrangement on soft clay layer, **b** placing the cement modified fibre reinforced sand layer on top of soft clay layer

wooden tamper, each clay bed layer was levelled properly before next clay lumps layer was placed till the required height. Then, to achieve saturation, for about seven days, the clay mass was covered properly using a polythene sheet. Then, for placing a layer of unreinforced sand on top of soft clay bed, the oven dried sand at maximum dry density and optimum moisture content is placed on top of clay bed. For sand layer which is cement modified fibre reinforced, the layer is prepared by manually mixing thoroughly determined optimum cement and optimum fibre with sand at respective optimum moisture content and maximum dry density and in layers is placed and compacted to the required thickness over the clay bed. The model square footing is centrally placed on the top levelled surface sand with the steel plate of same size over it. Loads were applied by hydraulic jack and recorded in proving ring. Two dial gauges were used in diagonally opposite direction to measure settlement (Fig. 3).

4 Results and Discussions

4.1 Standard Proctor Compaction Tests Results

Figure 4 explains, with cement percentage increased as 2, 3.5 and 5%, the optimum water content values reduced and maximum dry density increased till maximum cement percentage of 5%. Hence, the maximum 5% cement is considered optimum cement for the study.

Figure 5 explains, with including of fibre at percentages like 0.25, 0.5, 0.75 and 1% with optimum cement, the optimum water content values increased and maximum dry density reduced.

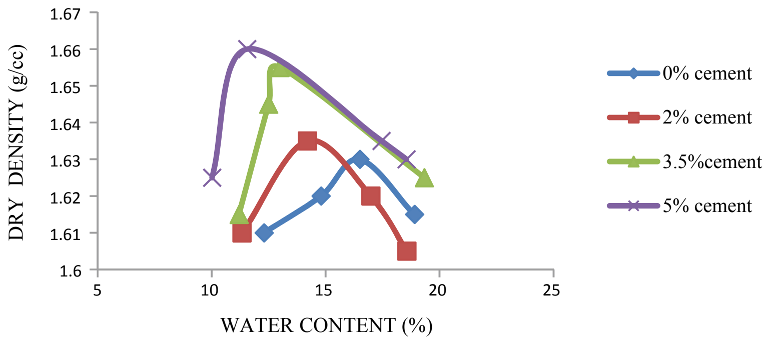

Fig. 4 Compaction curve for different cement percentages

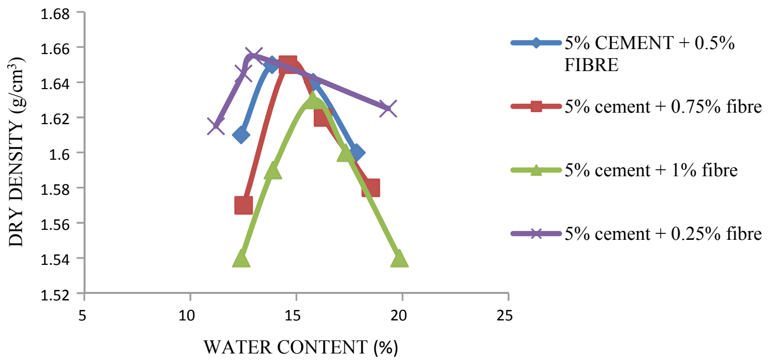

Fig. 5 Compaction curve for different fibre percentages at optimum cement

4.2 Direct Shear Tests Results

From Fig. 6, both the parameters of shear strength increases with the including of fibre at increasing percentage at 0% cement, however, upto 0.75%. At 1% fibre inclusion, the internal friction angle value remained constant.

Figure 7 explains that the internal friction angle and cohesion values start increasing with increasing fibre percentage with percentage of optimum cement, again upto 0.75% fibre. Beyond this fibre percent, the value remained reduced.

From Fig. 8, for certain normal stress value, the maximum shear stress increased for cement modified sand without fibre. But, the soil exhibited a brittle failure, as the maximum shear stress remarkably dropped.

However, from Fig. 9, reinforcing cement modified sand with fibre showed ductile characteristics and maximum shear stress also increased remarkably.

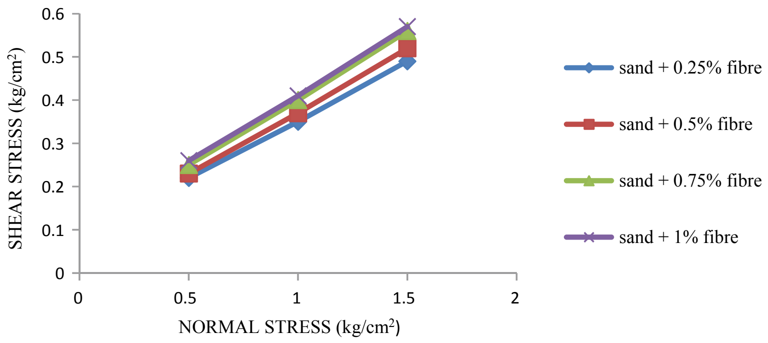

Fig. 6 Shear stress variation with normal stress for different fibre percentages without cement

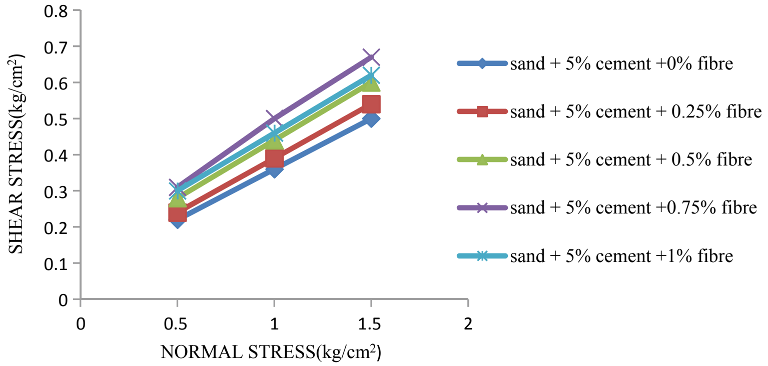

Fig. 7 Shear stress variation with normal stress for different fibre percentages at optimum cement

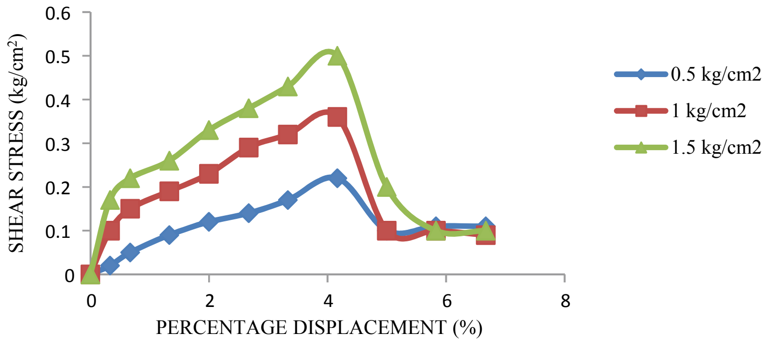

Fig. 8 Shear stress variation with shear strain for 5% cement + 0% fibre

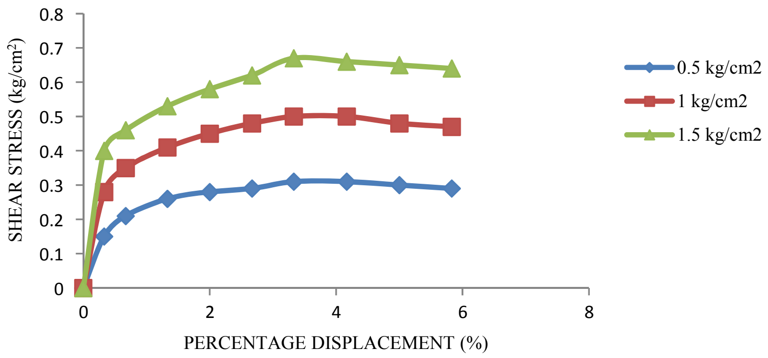

Fig. 9 Shear Stress variation with shear strain for 5% cement + 0.75% fibre

4.3 Experimental Modelling Results

Figure 10 presents the load versus settlement curve for top sand layer unreinforced with soft clay below for different H/B ratios (0, 0.5, 1, 1.5 and 2). The increase of H/B ratios increases the soil bearing capacity, calculated by two tangent method and reduces settlement upto $H/B = 1$, beyond which the increase was not significant.

Figure 11 shows a load versus settlement curve for the case when the weak soft clay is overlain by sand layer modified with optimum cement and fibre. The results showed that there was an increase in soil bearing capacity and settlement reduced upto $H/B = 1$, beyond which the increase was not significant again. Hence, the optimum H/B ratio is taken to be 1 (Table 3).

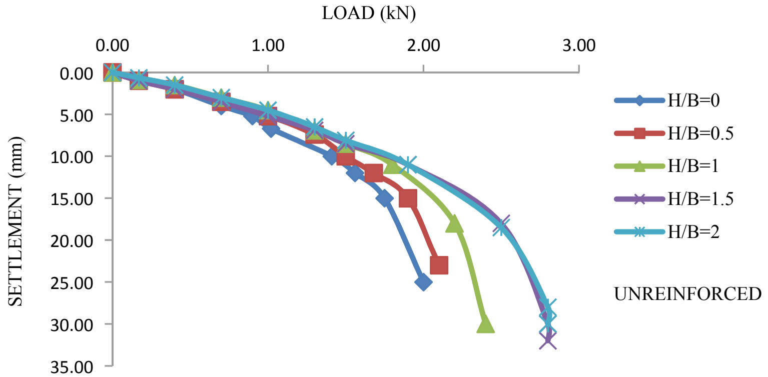

Fig. 10 Load versus settlement graph for top sand layer unreinforced with soft clay below

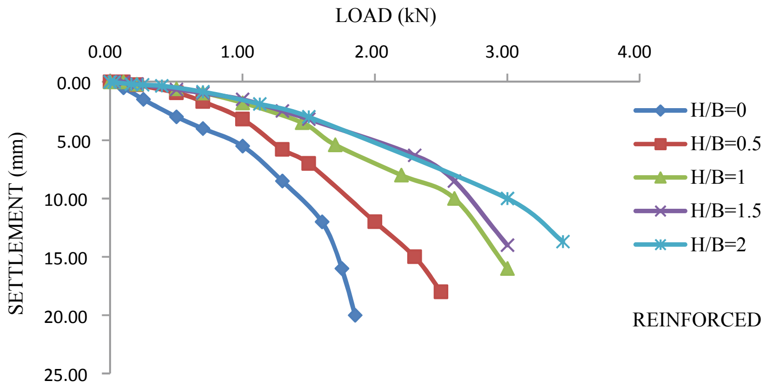

Fig. 11 Load versus settlement graph for top sand layer cement modified fibre reinforced with soft clay below

Table 3 Comparison of experimental results

Variable parameters	Experimental results			
	Unreinforced sand layer		Cement modified fibre reinforced sand layer	
	Ultimate bearing capacity (kN/m^2)	Settlement (mm)	Ultimate bearing capacity (kN/m^2)	Settlement (mm)
$H/B = 0$	37.5	9	37.5	9
$H/B = 0.5$	40	8.5	45	6
$H/B = 1$	47.5	8.5	55	4
$H/B = 1.5$	50	8	57.5	3.5
$H/B = 2$	50	7	60	3.5

5 Conclusion

The conclusions drawn from various tests results and analysis of the study are presented as follows:

1. The sand geotechnical characteristics which were cement modified and fibre reinforced were analysed with the help of compaction tests and direct shear tests. As the cement percentage increased, decrease in optimum moisture content (OMC) and increase in maximum dry density (MDD) were seen. Hence, the maximum cement percentage was considered as optimum cement. Then, the fibre percentage was varied at the maximum cement percentage which led to OMC increase and MDD decrease.
2. The interaction of sand, cement and fibre was studied from direct shear tests results which showed internal friction angle and cohesion significantly increasing as the fibre percentage is increased at maximum cement percentage upto 0.75%

fibre and the internal friction angle value became constant at 1%. Hence, 0.75% was taken as the optimum fibre percentage. Reinforcing cement modified sand with polypropylene fibre showed ductile characteristics and maximum shear stress also increased remarkably, otherwise showing brittle nature.

3. From laboratory modelling for the case when the weak soft clay is overlain by sand layer which is unreinforced, the soil bearing capacity increased with H/B ratio increased, which was calculated by two tangent method and reduces settlement upto $H/B = 1$, beyond which the increase was not significant.
4. From laboratory modelling for the case when the weak soft clay is overlain by sand layer modified with optimum cement and fibre, the bearing capacity increased and settlement reduced upto $H/B = 1$, beyond which the increase was not significant again. Hence, the optimum H/B ratio is taken to be 1.

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Enhancing Properties of Black Cotton Soil Using Bacterial Culture

Ratna Jadvani and Nilesh Savani

Abstract Black cotton soil is considered as most traitorous soil because of is high swelling and shrinking properties and extremely low bearing capacity. Pavements or structures constructed directly on black cotton soil without any treatment may lead to failure of the structure. Many stabilization techniques like use of fiber, geotextiles, fly Ash, rice husk ash, plastic waste, etc., have already been successfully used to stabilize such soils. In this work, an attempt has been made to stabilize and improve the engineering properties of black cotton soil of Surat zone using bacterial culture technique. Many bacteria have been found to precipitate calcium carbonate, which is a cementetious material, increases the strength of clayey soils by altering their properties.

Keywords Soil stabilization · Bacterial culture · Black cotton soil unconfined compressive strength · California bearing ratio

1 Introduction

Due to increasing population and urbanization, there is drastic need of infrastructure in cities. In fast developing country like India, population growth is a burning issue. Migration of people from rural to urban areas is constantly increasing. This has led to considerable increase in construction activities in urban areas. Land is becoming more and more scarce, and therefore, the construction of structure is to be carried out on the land areas having weaker soils which were earlier not considered suitable for construction.

The trend of improving engineering properties of weak and problematic soil is not new. Lot of research work is being done in this field. Many soil stabilization techniques are being successfully implemented till date and is still in progress.

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The properties of weaker soils have been enhanced using chemicals, additives, various types ashes, waste materials and many more. One of such weaker and problematic soil types is black cotton soil which is the dominant soil type in south Gujarat of India. Surat is one of the rapidly growing cities in India due to diamond and textile industries. There is constant migration of people from various parts of country and to provide infrastructure, on agricultural lands, construction is being done. Surat is mainly covered with black cotton soil. Earlier, the areas of Surat like Vesu, Pal, etc., which were basically agricultural land areas and are now included in Surat Municipal Corporation (SMC) limit which are completely zones of black cotton soil.

2 Description of Problem

Black cotton soil is highly expansive soil and having high swelling and shrinking properties. It has considerably low bearing capacity. Structures, pavements constructed on untreated black cotton soil leads to failure of structure. Many materials have been tried and successfully used to enhance the properties of black cotton soil.

Certain types of bacteria also have been found suitable for being used in stabilizing the soil [1]. These bacteria have been found to precipitate calcite material by biological process which is cementatious material and has tendency to get solidified in the soil voids and thereby increasing the strength of soil, reducing the swelling of expansive black cotton soils [2]. These bacteria are *Bacillus pasteurii*, *Bacillus megaterium*, Acetobacter, *Sporosarcina pasteurii*, etc. [3]. These bacteria grow or multiply in suitable culture medium and can be used in geo-technical engineering field for enhancing the black cotton soil properties.

After studying the literature available in this area, it was decided to select *Bacillus megaterium* for enhancing the black cotton soil of Surat.

3 Materials and Methodology

3.1 Soil

Approximately, 150 kg of soil sample, reported as black cotton soil had been collected from Rajhans Synfonia, a developer's construction site at Vesu, Surat at the depth of 5 ft. below the ground level.

Table 1 Sample classification

Bacteria	Soil sample name	Concentration of bacteria (cfu/ml)	Curing period (days)
Bacillus megaterium	P	10^8	7
	P1	10^8	14
	Q	10^9	7
	Q1	10^9	14
	R	10^{10}	7
	R1	10^{10}	14

Note 10^8 cfu/ml is approximately 1 ml liquid concentration of bacteria in which 10^8 colony of bacteria are present

3.2 Bacteria for Stabilizing of Black Cotton Soil

Bacillus megaterium bacteria type has been selected for improving the soil properties which is a gram-positive, rod shaped bacteria which can be grown under aerobic as well as anaerobic conditions [4]. It is capable of precipitating calcite material [5].

Culture of *Bacillus megaterium* bacteria type was prepared, and growth of bacteria was obtained under the guidance of expert of microbiology. For that the autoclaved tubes were taken and liquid agar was poured into them. Then, *Bacillus megaterium* bacteria were injected in these tubes and allowed them to grow. After the growth of bacteria in required quantity (10^8 cfu/ml), they were sprinkled on dry black cotton soil sample and the soil samples were kept covered using moist news papers. The samples of soil were kept at room temperature for 7 and 14 days. California bearing ratio (CBR) [6] and unconfined compressive strength (UCS) [7] tests were performed on these soil samples after curing period of 7 and 14 days. Both the tests were repeated for concentration 10^9 and 10^{10} cfu/ml. Three sets of tests were conducted for each concentration as shown in Table 1, and average value had been considered.

3.3 Test Conducted as Per IS Code

The following tests were conducted on untreated black cotton soil sample:

1. Grain Size Distribution
2. Atterberg limits
3. Free Swell Index
4. Specific Gravity
5. Compaction
6. California Bearing Ratio
7. Unconfined Compressive Strength.

Finally, California bearing ratio (CBR) and unconfined compressive strength (UCS) tests were conducted on treated black cotton soil, i.e. After 7 and 14 days of curing period.

4 Results and Analysis

Test results of untreated sample are presented in Table 2.

Results of California bearing ratio (CBR) and unconfined compressive strength (UCS) tests performed on bacteria treated soil samples after curing period of 7 and 14 days for three concentrations of *Bacillus megaterium* are as under.

From Table 3, it can be seen that bacterial culture has considerably increased the strength of black cotton soil samples.

As illustrated in Fig. 1, we can see that CBR value is considerably increasing with increase in concentration of bacteria as well as increase in curing period.

As illustrated in Fig. 2, it can be seen that UCS value is also increasing with increase in concentration of bacteria as well as increase in number of curing days. Comparision of CBR and UCS values of three concentrations with untreated soil have been shown in Fig. 3.

Table 2 Geotechnical properties of untreated soil

Sr. no.	Properties	Values
1	*Grain size distribution*	
	(a) Gravel (%)	0
	(b) Sand (%)	6.91
	(c) Silt + clay (%)	93.1
2	*Atterberg limits*	
	(a) Liquid limit (W_L) (%)	60.34
	(b) Plastic limit (W_P) (%)	21.60
	(c) Plasticity index (I_P) (%)	38.74
3	Differential free swell index (%)	54.54
4	Specific gravity	2.439
5	*Compaction characteristics*	
	(a) M.D.D. (g/cc)	1.5
	(b) O.M.C. (%)	20.73
6	*California bearing ratio value (%)*	
	(a) 2.5 mm penetration	9.47
	(b) 5.0 mm penetration	8.92
7	Unconfined compressive strength (kN/m^2)	2.99

Table 3 Results of CBR and UCS tests carried out with bacterial concentrations

Sample no	Days	CBR value	UCS value
Normal BC soil		9.47	2.99
P	7	8.61	2.68
P1	14	9.29	2.89
Q	7	9.54	3.09
Q1	14	10.58	3.26
R	7	12.96	3.97
R1	14	13.14	4.12

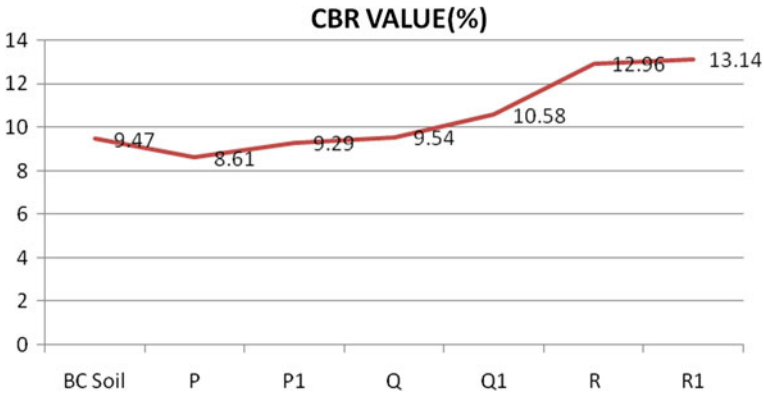

Fig. 1 CBR value (%) graph

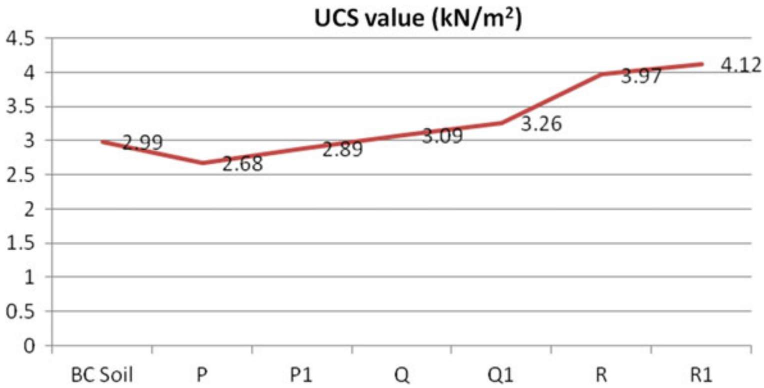

Fig. 2 UCS value for all samples

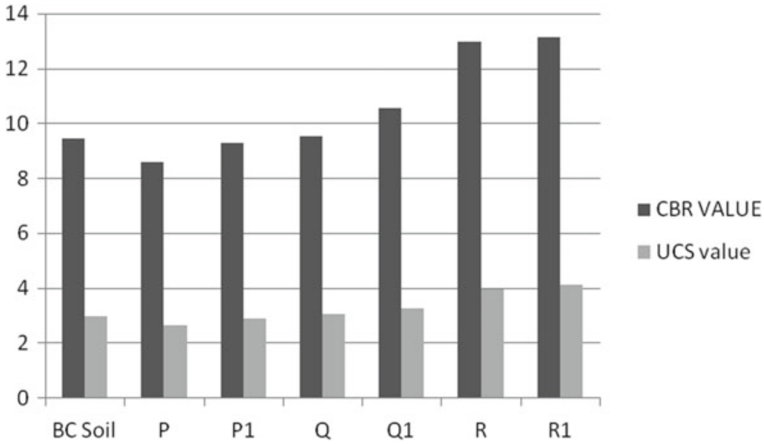

Fig. 3 Comparison of all tests carried out on bacterial culture

5 Conclusion

Expansive black cotton soil of Surat is treated with three different concentrations of *Bacillus megaterium* bacteria. Three samples of each concentration and is kept for a curing periods of 7 and 14 days. CBR and UCS tests were conducted on each and average value is reported.

After conducting the tests, following conclusions can be made:

1. From the results, the selected bacteria type *Bacillus megaterium* has been found quite effective to enhance the properties of expansive black cotton soil.
2. Though, there is no substantial increase in CBR, UCS values of black cotton soil, mixed with bacterial concentrations in 7 days curing period, there is considerable enhancement of these values in 14 days curing period. Hence, broadly it can be concluded from the above results that, as the days are increasing, the above-mentioned values increase.

The tests were conducted for three concentrations for 7 and 14 days curing periods only. The tests should also be performed for longer period and status of bacteria in the soil should be consequently checked.

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Comparative Study of Subgrade Strength of Soil Using Bio-Enzyme

Wafa Yousef and Soumya Jose

Abstract Environmental concerns have significantly influenced the construction industry regarding the identification and use of environmentally sustainable construction materials. Bio-Enzymes (organic materials) have been introduced recently for ground improvement projects such as pavements and embankments. The present experimental study was carried out in order to evaluate the subgrade strength of two different clayey soil treated with two different types of enzymes, namely TerraZyme and EarthZyme, as assessed through California bearing ratio (CBR) test and standard Proctor test. Soil specimens to be tested are prepared using different dosages of Bio-Enzyme (1, 2, 3 and 4 ml) mixed with water at optimum moisture content, and then it is sprayed over soil and compacted. Bio-Enzyme reduces the voids between the particles of soil and minimizes the amount of absorbed water in the soil so that the compaction caused by the enzymes can be maximized. It is seen that with the addition of Bio-Enzyme, OMC first decreases and then increases while MDD increases and then decreases. In case of CBR test, CBR value increases and then decreases.

Keywords TerraZyme · EarthZyme · California bearing ratio test · Compaction

1 Introduction

The demand for new construction materials which maintain a balance between cost and performance while at the same time satisfying environmental problems is one of the main challenges, construction firms face nowadays. Another difficulty faced by the engineers is the unavailability of sufficient land for construction. Hence, construction on available land becomes necessary even if it is weak in strength. Suitable stabilizing material is adopted considering the type of soil, cost-effectiveness

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and availability of materials. These challenges have led to the introduction of new ground improvement methods. Recently, an effective ground improvement method was introduced, that is stabilization using Bio-Enzyme. Bio-Enzyme is an organic, natural, inflammable and non-corrosive liquid which are the products of fermentation of organic matter. They are mainly used in highway projects. Bio-Enzyme improves the properties of soil and gives higher soil strength. They can easily get breakdown and dissolve with time as they are extracted by the fermentation of vegetables and sugar canes; hence, they are biodegradable. As they are in liquid form, it is easily soluble in water which saves time and cost of mixing. There are many Bio-Enzymes available in market like TerraZyme, PermaZyme and EarthZyme. In this study, two types of Bio-Enzymes, namely TerraZyme and EarthZyme, were used. Most of the informations about the enzymes are provided by the suppliers, and hence, independent testing information is not available.

The clay content plays a vital role in Bio-Enzyme stabilization. When the amount of clay content in the soil increases, the improvement in the soil properties by the addition of Bio-Enzyme also increases [1]. Here, two types of soil with varying clay content are used. Enzyme accelerates reaction between clay and organic cations, and cation exchange process takes place which reduce the adsorbed water layer thickness [2]. The use of Bio-Enzyme as a stabilizing material is accepted worldwide due to its effectiveness.

2 Experimental Program

The California bearing ratio (CBR) test and standard Proctor test were conducted to evaluate the effect of TerraZyme and EarthZyme on geotechnical properties of soils.

2.1 Materials Used

Soil. Two types of soil with varying clay content were used in this study. The first soil was collected from Poochinnipadam, Mukundapuram Taluk, Thrissur, and the next soil was collected from Thejus Engineering College campus of Thrissur District, Kerala. The geotechnical properties of soil sample 1 and soil sample 2 are given in Table 1. The soil sample 1 and soil sample 2 were classified as high plasticity clay (CH) and intermediate plasticity clay (CI), respectively, by IS plasticity chart.

Bio-Enzyme. Two types of Bio-Enzyme, namely TerraZyme and EarthZyme, were selected for this study. TerraZyme was purchased from Avijeet Agencies Pvt Ltd, Chennai, and EarthZyme was collected from Neha Infra Services, Mumbai. The main ingredients of two enzymes are non-ionic surfactant and carbohydrates. EarthZyme contained carbohydrates, and TerraZyme consisted of fermented vegetable extract.

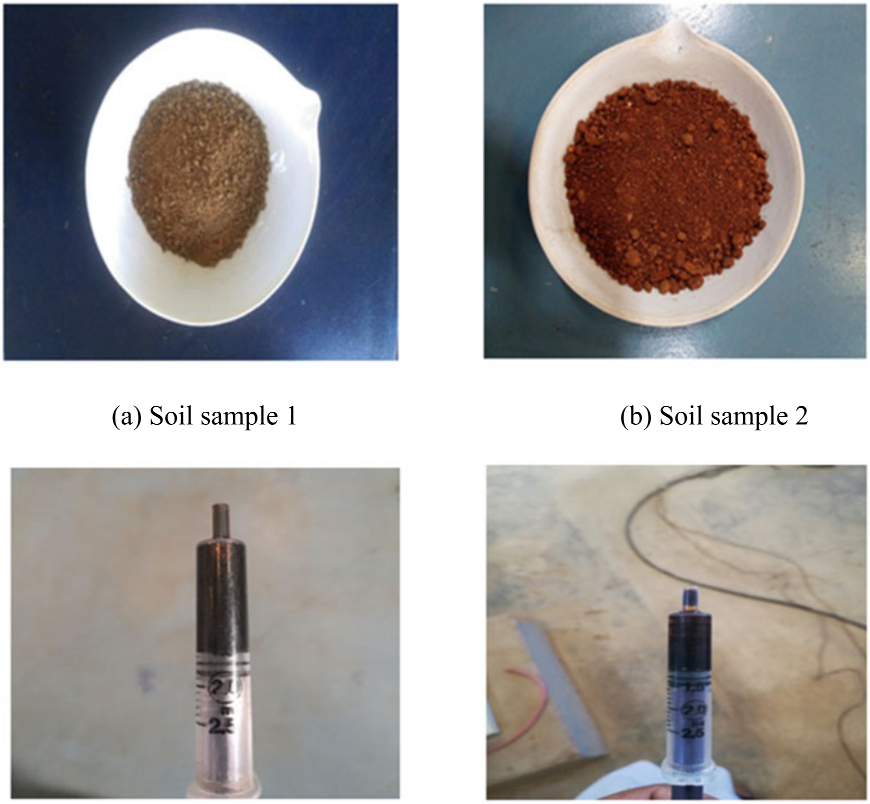
Table 1 Geotechnical properties of soils

Properties	Results	
	Sample 1	Sample 2
Specific gravity	2.68	2.71
Percentage of gravel (%)	5	0
Percentage of sand (%)	9	41
Percentage of fines (%)	86	59
Liquid limit (%)	52	48
Plastic limit (%)	25	22
Shrinkage limit (%)	16	18
Plasticity index (%)	27	26
Soil classification system	CH	CI
Optimum moisture content (%)	18	16
Maximum dry density (kN/m^3)	16.1	16.8
Unsoaked CBR value (%)	2.3	3.3
Soaked CBR value (%)	1.7	2.3

The chemical and physical properties of TerraZyme and EarthZyme are shown in Table 2.

Table 2 Chemical and physical properties of Bio-Enzymes

Properties	TerraZyme	EarthZyme
Water	21.06%	>50%
Alcohols, C12–C16, ethoxylated	NA	<30%
Fermented vegetable extract	NA	<20%
Non-ionic surfactants	55%	NA
Polysaccharides	2%	NA
Oligosaccharides	3%	NA
Disaccharides	5%	NA
Monosaccharide	8%	NA
Lactic acid	3.5%	NA
Potassium as the chloride	1.2%	NA
Aluminium as the sulphate	0.04%	NA
Magnesium as the sulphate	1.2%	NA
Specific gravity	1.0–1.1	1.0–1.1
pH	3–6	2.8–3.5
Boiling point	>100 °C	>100 °C
Ultimate biodegradability	Dissolved organic content reduction > 90% after 28 days	NA



(a) Soil sample 1

(b) Soil sample 2

(c) EarthZyme

(d) TerraZyme

Fig. 1 Materials used for the study

2.2 Sample Preparation

Standard Proctor Test. The collected natural soils were in the form of wet condition and placed in the oven for 24 h and crushed into dry powder form in a mortar. The light compaction test of both plain soils was conducted in order to determine the optimum moisture content and dry density. The measured quantity of plain soil and water for compaction test was taken. Bio-Enzyme with different dosages that is 1, 2, 3 and 4 ml were diluted with the water taken and mixed with the soil, and compaction test is done. Same procedure was repeated using other soil sample. From the compaction test, the optimum moisture content (OMC) and maximum dry density (MDD) of treated soil samples were obtained. The optimum value of OMC and MDD was also obtained.

California Bearing Ratio Test. For the sample preparation of CBR test, modified Proctor test should be done in order to obtain the optimum moisture content of the plain soil. With this optimum moisture content, CBR test was done. The measured quantity of plain soil sample and water for CBR test was taken. Bio-Enzyme with different dosages, 1, 2, 3 and 4 ml mixed with water at optimum moisture content of plain soil sample. This is then mixed with the soil sample taken and tied in a polyethylene cover and placed in a desiccator for two days for the reaction to take place. This sample is then taken, and CBR test is conducted. The CBR value for each dosage of Bio-Enzyme is obtained, and also the optimum CBR value is obtained. In the same way, soaked CBR value is also founded. The same procedure was repeated for the other soil sample. The results obtained from the above tests for two different soil sample were compared with the untreated soil samples results.

2.3 Testing Program

Standard Proctor Test. Compaction characteristics of both soil (untreated) were determined using standard compaction effort, and the same procedure was used to identify any change in compaction characteristics due to enzymes. Three important factors that affect the compaction are moisture content, type of soil and compaction effort. Compaction test is conducted in a compaction mould of diameter 100 mm, height 123 mm and weighs 4.5 kg. Bio-Enzyme of different dosages is added to the measured water and mixed with the soil. During the preparation of untreated and treated soil samples, an increment of 2% moisture content was chosen so that precise compaction characteristics could be determined. The mixed soil is placed into the mould in three layers, where each layer is compacted giving 25 blows using 2.5 kg hammer. The mould is filled with soil, and the collar is removed. The excess soil is trimmed off using a trowel. The weight of the mould with base plate and soil is taken.

The procedure is repeated until the weight of the mould, and soil is reduced by increasing the moisture content. From each soil sample made, small amount of soil is taken in a container and wet weight is taken and kept in the oven for 24 h. After 24 h, the soil sample is taken and dry weight is taken. From the calculation of compaction, the values of dry density and water content are obtained. The graphs were plotted showing the variation in optimum moisture content and maximum dry density of soil sample 1 and soil sample 2 for different dosages of Bio-Enzymes.

California Bearing Ratio Test. The procedure according to IS 2720 Part XVI (1987) was followed for the sample preparation and testing. Modified proctor test was carried out first in order to obtain the optimum moisture content (OMC) of plain soil. All specimen (untreated and treated) of CBR test were prepared using this OMC. The amount of soil taken was more than that of standard proctor test that is 5 kg of soil. The different dosages of Bio-Enzyme were added to this OMC, and then mixed with the soil. This mixture is placed in a polyethylene cover tied and placed in a desiccator for two days so that reaction can take place and adsorbed water can be removed.

The CBR mould is taken, and a weight is kept at the bottom of the mould. On the top of the weight, a filter paper is kept. The soil is taken out and placed in a CBR mould on the top of the filter paper in five layers where each layer is compacted giving 56 blows using hammer which weighs 4.5 kg. The soil is filled, and the collar is taken out. The excess soil is trimmed off, and the mould is kept upside down and the weight and filter paper which was kept at the bottom was taken out. This mould filled with soil is kept in the CBR testing machine by placing two weights inside the mould. Testing is done by noting the load against the corresponding penetration. From the reading, load against 2.5 mm penetration and 5 mm penetration is noted. The graphs were plotted showing the variation in unsoaked and soaked CBR value of soil sample 1 and soil sample 2 for different dosages of Bio-Enzymes.

3 Results and Discussions

3.1 Effect of Bio-Enzyme on Geotechnical Properties of Soils

Standard Proctor Test. EarthZyme and TerraZyme of varying dosages were added to the soil, and compaction test was carried out. The optimum moisture content (OMC) and maximum dry density (MDD) were found to be varying with the addition of EarthZyme and TerraZyme. Tables 3 and 4 show the variation in OMC and MDD of soil sample 1 and soil sample 2 with the addition of TerraZyme and EarthZyme, respectively.

The compaction characteristics of the soil samples improved when treated with Bio-Enzyme. From the graphs, it is clear that the OMC goes on decreasing and the MDD goes on increasing first with the increase in Bio-Enzyme. This improvement is mainly due to the reduction in the void ratio of the soil samples when treated with Bio-Enzyme [4]. When comparing both the soil samples, soil sample 1 has got more

Table 3 Variation in OMC of soil samples 1 and 2 with the addition of Bio-Enzyme

Dosage (ml)	Optimum moisture content (%)			
	Soil sample 1		Soil sample 2	
	EarthZyme	TerraZyme	EarthZyme	TerraZyme
0	18	18	16	16
1	16.8	16.4	15.2	15
2	14.6	14.3	13.9	13.6
3	16.5	16.2	12.3	12.1
4	17.6	17.1	14.7	14.1

Table 4 Variation in MDD of soil sample 2 with the addition of Bio-Enzyme

Dosage (ml)	Maximum dry density (kN/m^3)			
	Soil sample 1		Soil sample 2	
	EarthZyme	TerraZyme	EarthZyme	TerraZyme
0	16.1	16.1	16.8	16.8
1	16.7	16.9	17	17.1
2	17.4	17.7	17.3	17.5
3	17.1	17.4	17.6	17.8
4	16.9	17.2	17.4	17.7

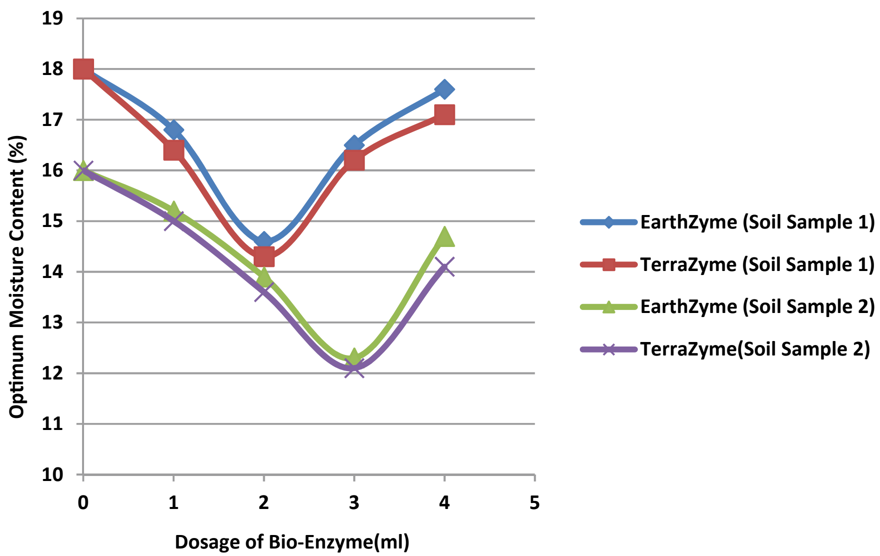


Fig. 2 Variation in optimum moisture content by the addition of Bio-Enzyme

MDD value than soil sample 2. And when comparing both the Bio-Enzymes used, TerraZyme has got more MDD value than EarthZyme. The MDD of soil sample 1 and soil sample 2 increases up to 2 ml and 3 ml dosage of Bio-Enzyme, respectively, and decreases thereafter. The OMC of soil sample 1 and soil sample 2 decreases up to 2 ml and 3 ml dosage of Bio-Enzyme, respectively, and increases thereafter. Hence, the optimum value is obtained at 2 ml and 3 ml dosage of Bio-Enzyme for soil sample 1 and soil sample 2, respectively. Compaction tests conducted on CH soil revealed that MDD increased and OMC decreased on adding various dosage of Bio-Enzyme [5].

California Bearing Ratio Test. EarthZyme and TerraZyme of varying dosages mixed with the OMC of both soil samples were added to the soil, and CBR test was carried out. Tables 5 and 6 show the variation in unsoaked and soaked CBR

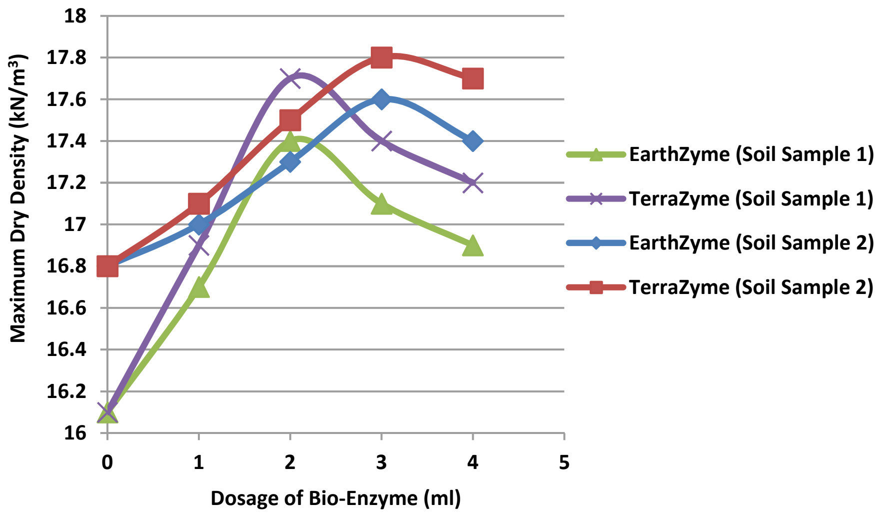


Fig. 3 Variation in maximum dry density with the addition of Bio-Enzyme

Table 5 Variation in unsoaked CBR value on addition of Bio-Enzyme

Dosage (ml)	Unsoaked CBR value (%)			
	Soil sample 1		Soil sample 2	
	EarthZyme	TerraZyme	EarthZyme	TerraZyme
0	2.3	2.3	3.4	3.4
1	8.9	9.4	8.18	8.9
2	13.6	14.9	10.8	11.6
3	11.6	12.2	14.1	15.1
4	9.4	10.7	11.8	12.9

Table 6 Variation in soaked CBR value on addition of Bio-Enzyme

Dosage (ml)	Soaked CBR value (%)			
	Soil sample 1		Soil sample 2	
	EarthZyme	TerraZyme	EarthZyme	TerraZyme
0	1.7	1.7	2.4	2.4
1	7.1	8.2	6.7	7.4
2	11.8	12	9.5	9.9
3	9.9	10.5	13.5	13.6
4	8.4	8.9	10.4	10.7

value of soil sample 1 and soil sample 2 on addition of EarthZyme and TerraZyme, respectively.

It can be inferred from the graphs that the CBR value has increased on addition of Bio-Enzyme and decreased after the optimum value. This is because the soil treated with Bio-Enzyme renders improved density values by reducing the void ratio.

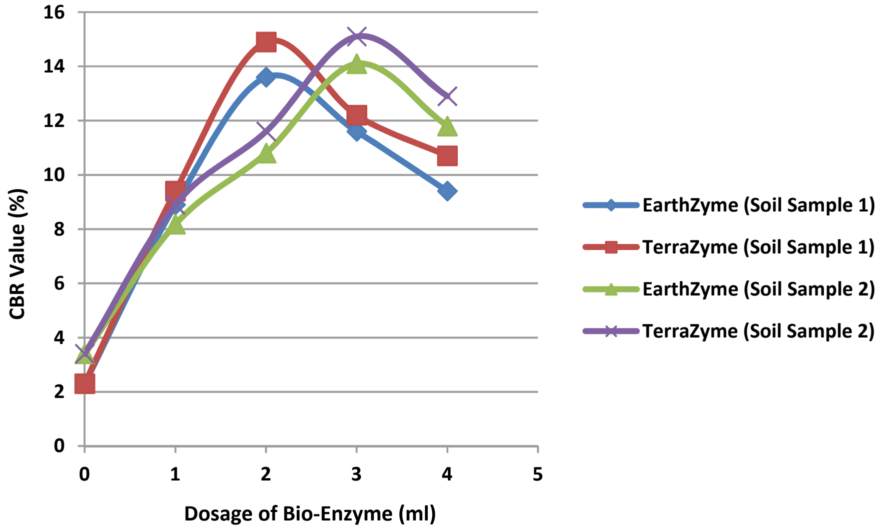


Fig. 4 Variation in unsoaked CBR value on addition of Bio-Enzyme

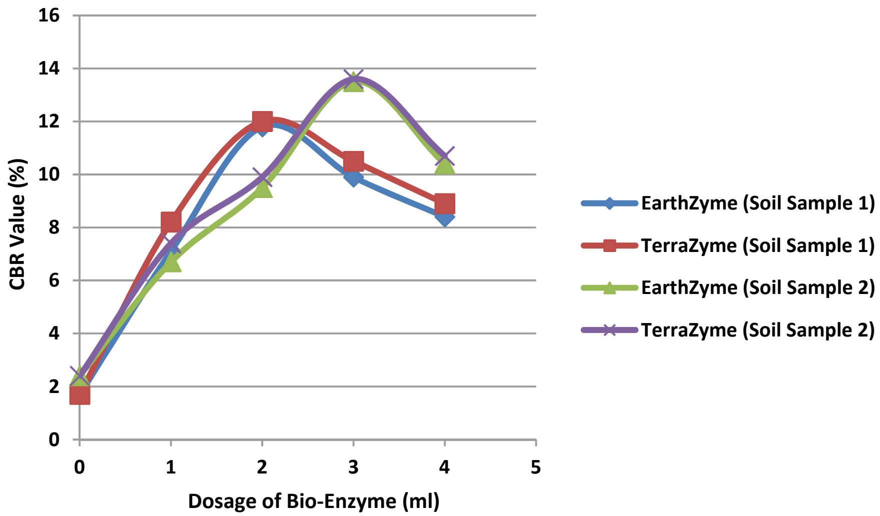


Fig. 5 Variation in soaked CBR value on addition of Bio-Enzyme

This may be due to effective cation exchange process which generally takes longer period in the absence of such stabilizers [6]. When comparing both the soil sample, soil sample 1 has got more CBR value than soil sample 2 in both unsoaked and soaked conditions. This is because when the clay content in the soil increases, the improvement in the soil properties by the addition of Bio-Enzyme also increases [3]. And on comparing both the Bio-Enzymes used, TerraZyme has got more CBR value of 547%. The unsoaked and soaked CBR values of soil sample 1 increase up to 2 ml dosage of Bio-Enzyme and decrease thereafter, whereas for soil sample 2, the value increases up to 3 ml dosage of Bio-Enzyme and decreases. Therefore, the optimum value is obtained at 2 ml and 3 ml dosage of Bio-Enzyme for soil sample 1 and soil sample 2, respectively.

4 Conclusions

It has been found that mixing of both type of Bio-Enzymes, namely TerraZyme and EarthZyme, has sufficiently improved the geotechnical properties of soil. The results showed that there is a significant effect on compaction characteristics of soil with the addition of Bio-Enzyme. At the same time, CBR value of the soil also increased with the increase in dosage of Bio-Enzyme.

- The variation in OMC of both soil sample goes on decreasing up to an optimum value and increases thereafter with the increase in dosage of EarthZyme and TerraZyme.
- The variation in MDD of both soil sample goes on increasing up to an optimum value and decreases thereafter with the increase in dosage of EarthZyme and TerraZyme.
- Maximum Dry Density was obtained at 2 ml dosage of Bio-Enzymes for soil sample 1 and at 3 ml dosage of Bio-Enzymes for soil sample 2.
- The unsoaked and soaked CBR values of both soil sample go on increasing up to an optimum value and decreases with the increase in dosage of EarthZyme and TerraZyme.
- Maximum CBR value was obtained at 2 ml dosage of Bio-Enzymes for soil sample 1 and at 3 ml dosage of Bio-Enzymes for soil sample 2.
- Better result was obtained for soil sample 1 that is CH soil due to the presence of more clay content in the soil when comparing with soil sample 2 that is CI soil. When the clay content in the soil increases, the improvement in the soil properties by the addition of Bio-Enzymes also increases.
- When comparing both TerraZyme and EarthZyme, soil treated with TerraZyme showed better results than the soil treated with EarthZyme.

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Determination of Shear Strength, Shear Viscosity and Liquidity Index Using Fall Cone Penetrometer

Jiji Krishnan, Ashish Juneja, and C. H. Solanki

Abstract In the usual case, the fall cone test has been used to estimate the undrained shear strength of insensitive remoulded clays. Its concept was based on the critical state soil mechanics and is well established. It is only recently that the use of this simple laboratory equipment has been extended to estimate the shear viscosity of soils which are well below their liquid limit. At these water contents, the viscous strength helps to understand the resistance of soils to flow in penetration tests, pile driving and landslides. This paper reports the results of fall cone tests under different loads. In this study, the effect of water content on shear viscosity was investigated by varying the weight of the cone. All the tests were conducted using speswhite kaolin clay. The samples were thoroughly mixed with water and then kept overnight in air-tight containers to ensure proper mixing of the clay with water. The fall cone apparatus was modified to accommodate an LVDT connected to a high-speed data logger which enabled the cone penetration to be logged at every 0.01 s. The mass of the standard cone was increased by 390 to over 1000 g with the use of stainless steel discs. In some tests, the cone was not permitted to fall freely but was driven at a constant rate of 1.25 mm/min. The load versus penetration resistance measured at different water contents showed that the penetration increased with the increase in LI and at a given load, the results were consistent with the triaxial test results. Shear viscosity was calculated using the rate at which the cone penetrated into the soil and was shown to decrease with the increase in LI. Design curves were obtained for the different weight of the cone which can be implemented in the selection of soil characteristics, for example, in estimating the capacity of different driven piles.

Keywords Fall cone · Viscosity · Liquidity index · Undrained shear strength

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1 Introduction

Remoulded sensitive clays have the ability to move or be moved freely and easily and often noticed in many landslides [1–3]. Flow properties are the main parameters used to estimate the behaviour of remoulded clays. Eden and Kubota [4] were the first to introduce the relationship of remoulded undrained shear strength and viscosity of sensitive clays. The viscosity of water changes with temperature and pressure [5–7]. The viscosity of a fluid can be altered by adding different minerals such as iron, potassium, chloride and sulphate into it [8, 9]. Yield stress measured from viscometer was later used to evaluate the liquid limit of soils [10]. However, Garneau and LeBihan [11] reported the possibility of using fall cone to measure the remoulded shear strength of the soil.

Shear viscosity measures the resistance of flow. Its knowledge plays a significant contribution to interpreting the viscous resistance in dynamic penetration tests. The soil penetration tests which make use the apprehension of shear viscosity are cone penetrometers, landslides and driven piles. This property can be calculated with the help of data obtained by fall cone penetration tests. The Swedish fall cone test for determination of shear strength and sensitivity which is now using as a standard tool of soil mechanics was first introduced in 1915 by a Swedish geotechnical engineer John Olsson (Secretary of the commission of Swedish State Railways) to measure the strength of soft clays in a remarkably simple manner. In regular geotechnical engineering practice, soft clay is considered as a soil having a shear strength less than 250 g/cm^2. Traditionally, fall cone tests have been used to determine the undrained shear strength of remoulded soils [12–15]. Using this device, Hansbo [13] made an extensive study on cone penetration testing with the help of four different cones to correlate their penetration to the shear resistance. Wroth and Wood [14] were developed a relationship between the plasticity index (PI) and the compression index (C_C). The ratio between the strength of the soils at its plastic limit and the strength of the soil at its liquid limit was taken to be a unique value. Shear viscosity is needed to compute the viscous resistance of soil and also fall cone test has recognized to be a favourable method for calculating the shear viscosity of soft soils at low liquidity indices [16].

Mudflows are considered to be initiated by the changes in soil viscosity. Many researchers were reported that Atterberg limits, liquidity index and flow velocity are the three main factors which influence the mudflow [17–19]. Widjaja and Lee [20] observed that the main criteria for describing the commencement of mudflows due to changes in the soil conditions are its viscosity. Mudflows occur very quickly and unexpectedly or without warning and thus it can be considered a very hazardous event [21]. Deformation of soil mass happens when the phase of soil changes from liquid to plastic. Soil mass will flow like a liquid when its water content reaches to the liquid limit. This is the state at which soil behaves like a liquid and thus mudflow occurs [22]. Viscosity is an important factor in predicting mudflows as well. The commercially available viscometer only predicts the liquidity index of liquids which is more than one [23–25]. But due to the lack of such an apparatus to measure the viscosity of soil near to their liquid limit, calculation of undrained shear strength to validate the shear viscosity is unavoidable.

In this paper, an attempt was made to compare the undrained shear strength calculated by the fall cone test with undrained shear strength using triaxial apparatus [26]. Also, an attempt was made to compare and validate the shear viscosity calculation [16] with the addition of weights up to 10.8 N using fall-cone apparatus.

2 Methods of Measuring Undrained Shear Strength of Cohesive Soils

An experiment in the field provides the scientific procedure to examine involvement in the original world (or from the view of experimentalists, naturally occurring environments) precisely than in the laboratory. Irrespective of this, tricky instrumentation and also complex and expensive equipment often lead to laboratory testing rather than in situ field testing even though independent laboratory testing provides defensible, repeatable results. Very recently, laboratory vane test and fall cone test overtopped to find the strength of cohesive soils. This simple testing cannot be extruded from their cores and easy in case of weak samples. European countries are following the fall cone test to calculate the strength from the last few decades. The notable words provided by Ladd and Foott [27] is that "the in-situ undrained shear strength of clay is a unique function of its water content and that it can be readily measured by virtually any in-situ or laboratory vane shear test that does not allow changes in water content". Any change in water content can easily be prevented using these tests. Thus, the strength measured is the undrained shear strength where the soil is sheared at constant water content, and if saturated, also at constant volume. Since the work is focused on kaolin clay samples fall cone is found to be best suited.

3 Overview of the Problem

Shear viscosity is an important factor which plays an inevitable position in studying the viscous resistance in the dynamic penetration of clays. Role of the viscosity of soil is applicable in soil engineering fields such as submarine and subaerial slide dynamics and also in cone penetrometers, landslides and jacked piles. Viscometers are used to measure the viscosities of soil in which their water content is greater than or equal to the liquid limit. It is known that no such basic method is there to calculate shear viscosities of clays at lower water contents (i.e. less than the liquid limit).

The proposed explanation has made on the basis of limited evidence for this study are:

1. Creeping flow hydrodynamics can be used to examine the viscous drag which is a part of the total penetration resistance. This total resistance is presented in rigid bodies such as shaft during uninterrupted penetration.
2. Shear viscosity of clay can be evaluated with the help of the fall cone test, which is a prospective device.

The purpose of the present research program is to make use of the application of viscous flow (hydrodynamics) principles to soils at a critical state. This will help to figure out the results of viscous resistance on penetrating rigid bodies into the soil. Additionally, this will act as an important parameter to measure the inducing factors during mudflow.

4 Goal and Objectives of the Current Research

(1) To develop a modified fall cone apparatus which will accept worldwide to relate liquidity index, shear strength and shear viscosity.
(2) To find out the variation of the pattern obtained to shear viscosity versus liquidity index graph with the addition of weights in fall cone so that we can correlate the same with the prevailing site conditions.
(3) To find out the variation of liquidity index with time using the help of modified fall cone apparatus as well as triaxial apparatus.
(4) The comparison of shear strength from the cone penetration test as well as from the strain-controlled loading frame test using the same kaolin with the same properties.

5 Experimental Program

5.1 Materials Used

The clay used in this study was commercially produced speswhite kaolin obtained from Kutch in Gujarat. All the basic properties of kaolin used for the experiment are shown in Table 1.

5.2 Methodology

The fall cone apparatus was slightly modified by removing the dial gauge and rack. The dial gauge was replaced by a potentiometer using 2 mm thick fabricated aluminium plate casing and fixed to it (Fig. 1a). All the procedures are followed as per BS 1377 [28]. Continuous penetration of fall cone was ensured with the help of a

Table 1 Basic properties of kaolin used for the experiment

S. no	Properties	Value
1	Natural moisture content (%)	1.5
2	*Gradation analysis*	
	Fine sand (%) Silt (%) Clay (%)	02 38 60
3	Specific gravity	2.65
4	*Atterberg limits*	
	Liquid limit (%) Plastic limit (%) Liquidity index (%) Shrinkage limit (%)	46.30 27.69 18.60 26
	Classification symbol	CI
5	*Compaction characteristics*	
	Max dry density (g/cm^3) Optimum moisture content (%)	1.46 26.5
6	pH	7.2
7	*Elemental analysis by EDAX*	
	SiK (%) OK (%) AlK (%) MgK (%) CaK (%)	39.90 35.81 11.08 7.07 6.14

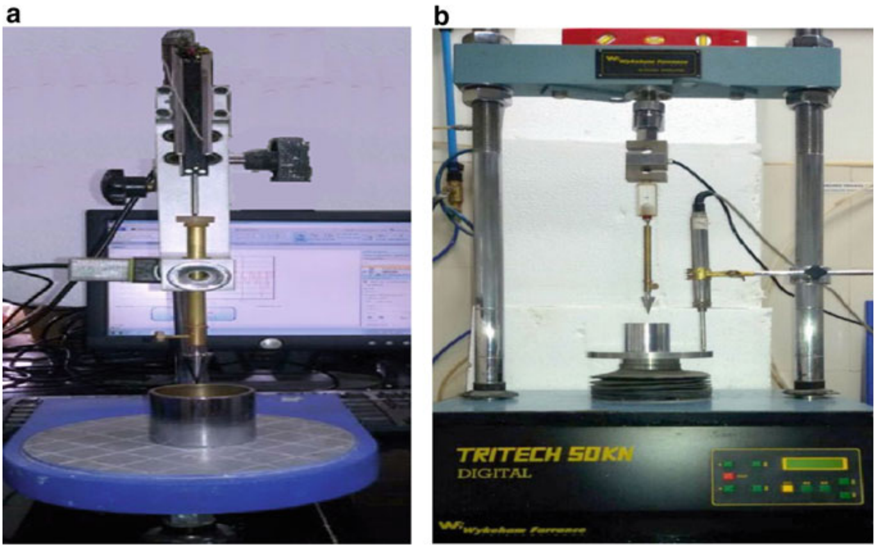

Fig. 1 **a** Modified fall cone apparatus, **b** cone connected to the loading frame

calibrated potentiometer. This was connected at the top of the shaft and readings are continuously measured and stored in a data logger. The angle at the tip of the cone was 30^0. The cone and the shaft together weighed 0.9 N in which all the test were conducted by changing the liquidity index from 0.29 to 2.

The apparatus was then connected to a data logger to receive the data at less than 0.01 s so as to obtain at least 50 data point at all water contents. The new fall cone device provides the same general principles as that of the traditional fall cone. The total weight of the cone was about 0.9 N.

Extra weights were added to vary the cone weight from 0.9 to 10.8 N. Weights were added to find out the difference in penetration rate and shear viscosity while changing the weights of the cone. Fall cone penetration apparatus (BS 1377, British Standard Institution, 1990) with a 30° cone was used during the experimental investigations. To ensure the homogeneity of the sample, the soil was kept inside the desiccator for an overnight.

Cone driving is also conducted with the help of a triaxial apparatus (Fig. 1b) in which the same cone, which is used for fall cone testing is utilized. A load cell and a potentiometer were fixed so that load and the cone was driven at a rate of 1.25 mm/min to the soil sample which is kept at the cylinder (used for the fall cone test). Both the load cell and potentiometer were connected to the data logger so that it directly provides the data to the computer.

6 Theoretical Considerations

The theory is based on the traditional fall cone test described by Hansbo in 1957 [13] and Wroth and Wood [14] in 1978 and Wodd in 1982 [15]. To explain the theory, a brief background for this experiment is reported as follows. Hansbo [13] conducted a comprehensive experiment with the help of different cones to observe the undrained shear strength. He used the expression

$$\tau_{\text{cs}} = \frac{KW}{h_{\text{f}}^2} \tag{1}$$

relating undrained static shear strength τ_{cs} with penetration h of a cone of weight W, and deduced different values of cone factor K (which depends on the angle of cone and sampler used to collect soil from the ground). For these correlations with the shear strength of undisturbed samples, he found that $K_{30^\circ} = 4 \times K_{60^\circ}$. Hansbo [13] suggested $K \approx 0.3$ for 60° cones for remoulded clays that correspond to a K value of 1.2 for 30° cones. Koumoto and Houlsby [29] developed a reduction factor (λ) to revise the fall cone factor (K) to estimate the value of static undrained shear strength and they also suggested that the undrained shear strength is a dynamic shear strength and is higher than the static undrained shear strength which has higher strain rates in the fall cone test. The equation of motion of a cone having mass m, acceleration due to gravity g, a is the acceleration of the cone at a depth h and τ is the dynamic shearing resistance and F is the non-dimensional cone resistance factor is

$$ma = mg - F\tau h^2 \tag{2}$$

and F is calculated as per the theory of Houlsby [30], Koumoto and Houlsby [29] having modified bearing capacity factor N_{ch} and semi-rough cone angle θ is

$$F = \pi N_{\text{ch}} \tan^2 \theta \tag{3}$$

(for a 30° cone $N_{\text{ch}} = 7.457$).

When cone intrudes into the soil, acceleration of the cone reduces from its commencing range (g). The acceleration of cone reaches to zero when we reach a certain depth. The depth is considered a dynamic equilibrium position h_{eq}. At final depth, h_{f} the cone will be at a position of rest.

At dynamic equilibrium condition ($a = 0$).

Considering viscous soil as a Casson fluid, the term $\mu_{\text{p}}\ddot{\gamma}$ can be calculated as

$$\mu_{\text{p}}\ddot{\gamma} = \left[\tau^{\frac{1}{2}} - \tau_{\text{cs}}^{\frac{1}{2}}\right]^2 = \left[\left(\frac{W}{Fh_{\text{eq}}^2}\right)^{\frac{1}{2}} - \left(\frac{KW}{Fh_{\text{f}}^2}\right)^{\frac{1}{2}}\right]^2 \tag{4}$$

$$K = 3\frac{\lambda}{F} \tag{5}$$

$K = 1.33$ and $\lambda = 0.74$ for a 30° cone.

$\mu_{\text{p}}\ddot{\gamma}$ (the viscous component of shear resistance) can be written as

$$\mu_{\text{P}}\ddot{\gamma} = KW\left[\frac{0.67}{h_{\text{eq}}} - \frac{1}{h_{\text{f}}}\right]^2 \tag{6}$$

The average shear strain rate calculated by Koumoto and Houlsby [29] with

$$\ddot{\gamma} = \frac{2\delta}{2.44}\sqrt{\frac{g\sqrt{2}}{h_{\text{f}}}} = 0.34\sqrt{\frac{1}{h_{\text{f}}}} \tag{7}$$

The shear viscosity of the soil at dynamic equilibrium is

$$\mu_{\text{p}} = 2.94KW\sqrt{h_{\text{f}}}\left[\frac{0.67}{h_{\text{eq}}} - \frac{1}{h_{\text{f}}}\right]^2 \tag{8}$$

With the help of penetration versus time graph velocity can be calculated.

7 Results and Discussion

Figure 2a shows the penetration of the 10.8 N cone at two different liquidity indices (i.e. 0.30 and 0.18). The two curves in Fig. 2a were differentiated to obtain the velocity of the cone as shown in Fig. 2b.

After extracting the data which was obtained from the data logger, shear viscosity was calculated by the relations provided by Budhu [16]. Figure 3 shows the variation of liquidity index with shear viscosity by changing cone weights. Figure 4 shows the relation between shear strength, S_u and liquidity index (LI) of kaolin clay for different cone weights. The shear strength reduces with increasing liquidity index for all different weights of the cone.

From plots of load against penetration depth (Fig. 5) obtained from cone driving, load at 20 mm penetration depth was considered and divide with the area to plot shear strength versus liquidity index (Fig. 6). Shear strength obtained from cone driving, load at 20 mm penetration depth showed that shear strength increases with reducing liquidity index.

The comparison of shear strength obtained from the cone penetration was done with the results obtained from the strain-controlled loading frame tests using the same kaolin with the same properties [26].

Strength equation

$$\frac{S_u}{p'} = 0.23 \times (\mathrm{OCR})^{0.8} \tag{9}$$

For normally consolidated clays, OCR is equal to 1. Furthermore p'_0 can be obtained for the equation

$$\upsilon = \upsilon_\lambda - \lambda \ln p'_0 \tag{10}$$

Substituting Eq. (2) in (1) leads to

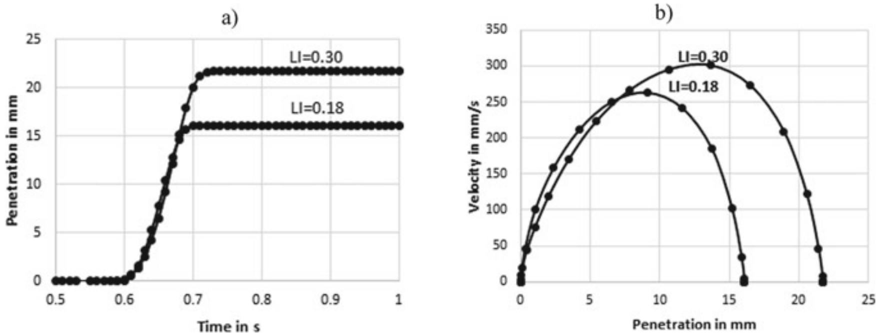

Fig. 2 **a** Penetration versus time, **b** penetration versus velocity

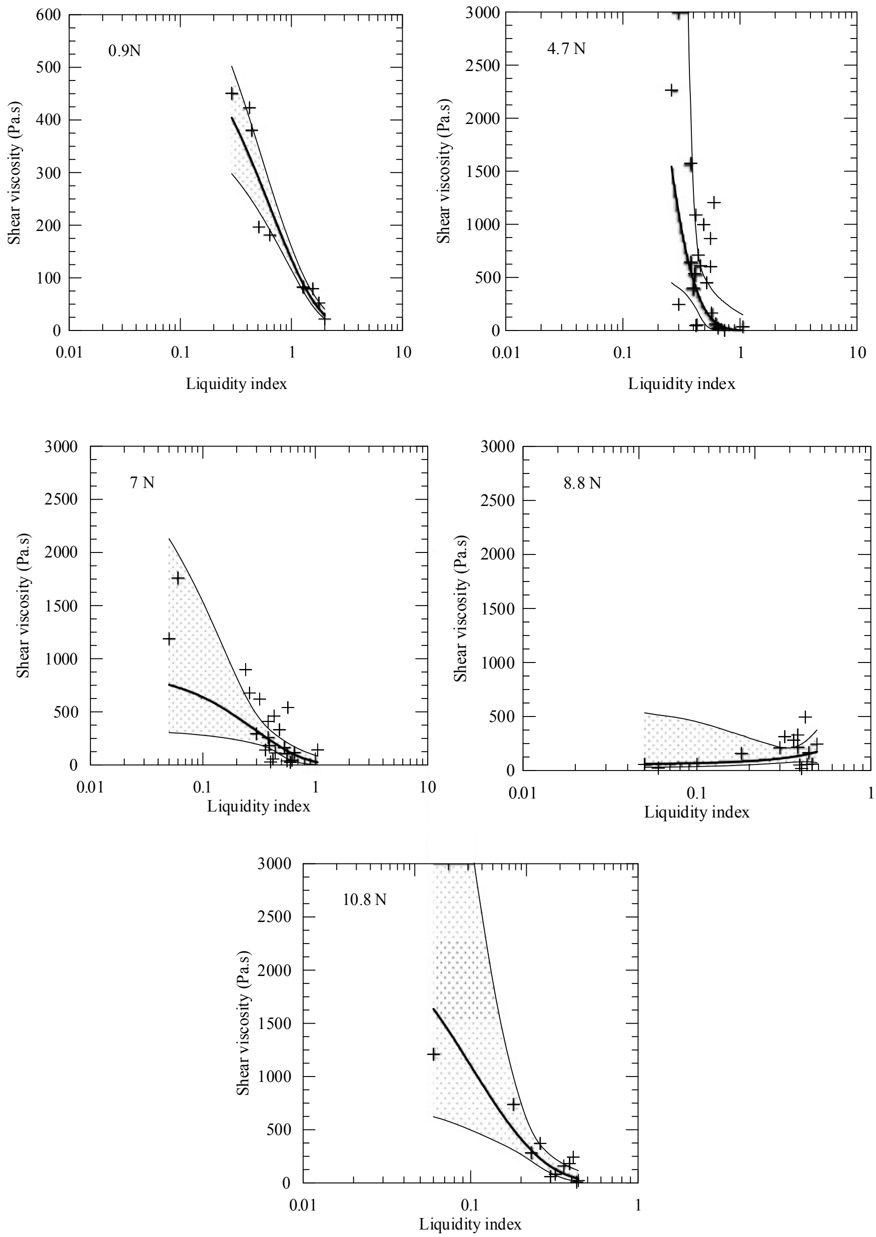

Fig. 3 Variation of shear viscosity with liquidity index (different weight of cone)

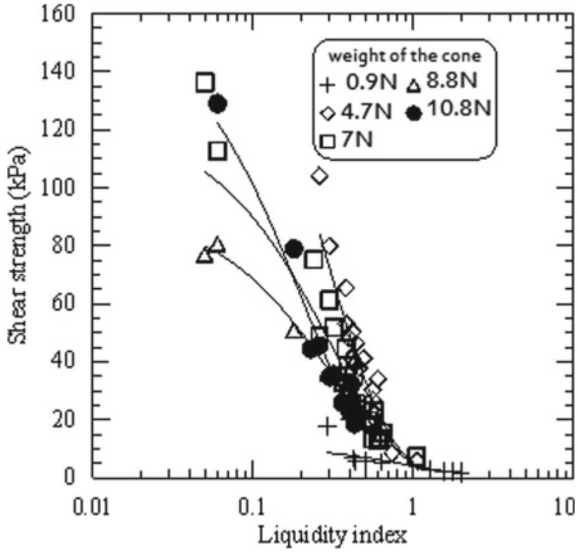

Fig. 4 Variation of undrained shear strength with liquidity index for the different weight of cone

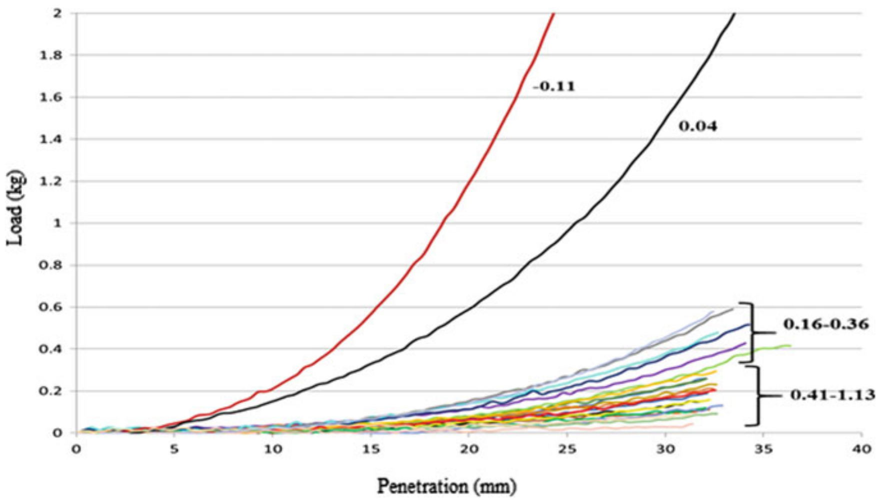

Fig. 5 Plots of load against penetration depth obtained from cone diving

$$S_u = 0.23 \times e^{\frac{0.451-w}{0.0379}} \tag{11}$$

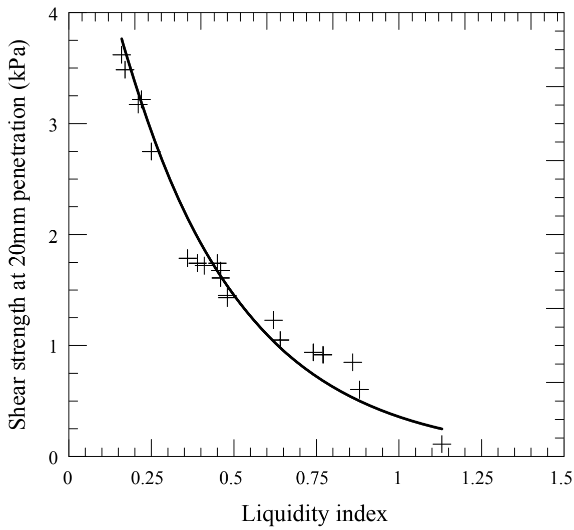

Fig. 6 LI versus shear strength at 20 mm penetration depth

8 Conclusions

The observations obtained from the laboratory experiments conducted by varying the weight of the cone are

- Liquidity index varies exponentially with time.
- Weight of the cone is an important factor which makes exponential changes in liquidity indices.
- Shear strength versus LI obtained from both triaxial test results and fall cone connected to loading frame are in good agreement.
- The shear strength reduces with increasing liquidity index for all different weights of the cone.
- The undrained shear strength obtained from cone driving, load at 20 mm penetration depth showed that shear strength increases with reducing liquidity index.
- The current research has also created a modified fall cone apparatus to effectively measure the viscosity levels as soil changes from the plastic state to the liquid state.
- The test results can be utilized to figure out the variations of viscous resistance on penetrating rigid bodies into the soil. Additionally, this will act as an important parameter to measure the inducing factors during mudflow.

9 Future Work

- Comparison of fall cone with Casagrande apparatus for liquid limit determination.

- Comparison of fall cone test results with laboratory vane shear test.
- To check the liquidity index versus shear strength variation using different cone angles (30^0).
- To check the liquidity index versus shear strength variation using blunt cones.
- To find out the penetration versus load graph of cone driving using different cones and thus to develop a relationship between load, penetration and different cone angles.

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Cost-Effective Foundation System for Medium Rise Residential Building on Typical Soft Kolkata Soil

Monojit Mondal, Shuvra Saha, and J. J. Mandal

Abstract In present scenario due to rapid growth in population, it is not possible to construct each and every residential building on hard soil. Some time, it is necessary to construct building on soft soil. One of the examples is Newtown, Rajarhat, Kolkata. Rajarhat is a developing area in Kolkata where most of the residential buildings are medium rise, i.e. four or five storied. In that area, soil consists of very soft clay having very low value of SPT value (2–5) up to a depth of 12 m or below, followed by hard strata. In most of the cases, shallow foundation cannot give feasible solution and structural designers prefer pile foundation before adopting other alternatives techniques to improve the soil. Pile foundations are not very cost effective. Cost of the foundation constitutes a significant part of the total cost of the structure. Hence, reduction in cost of the foundation leads to reduction in total cost of the structure. In this study, an effort has been made to recommend feasible cost-effective foundation system by using ground improvement technique (by using stone column) vis-vis pile foundation. A typical medium rise building (G + 4) has been chosen on a typical soft soil profile, typical to Rajarhat area. A comparative study has been made to suggest cost-effective foundation system for medium rise residential building in that area.

Keywords Ground improvement · Stone column · Pile foundation

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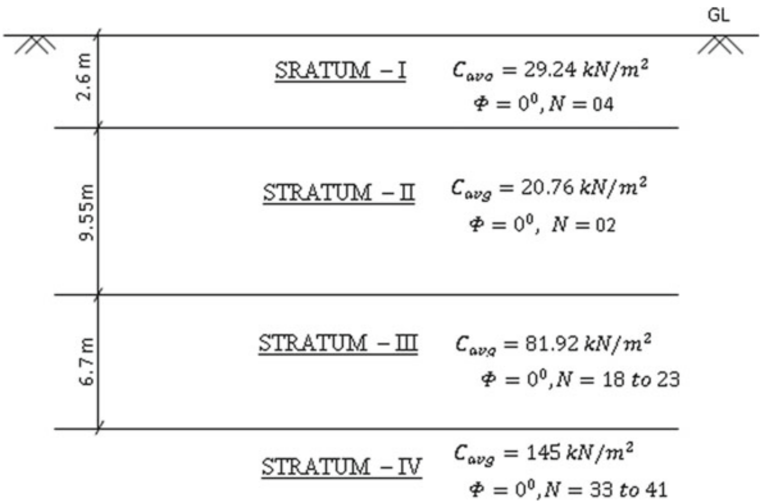


Fig. 1 Typical soil profile

1 Introduction

1.1 Problem Statement

A multi-storied building (G + 4) is taken in to consideration for this project work. Soil test of the proposed site is carried out by a reputed organization and it is adopted for analysis. A typical soil profile, floor plan, and grid plan are shown in Figs. 1, 2, and 3, respectively.

The soil profile consists of four layers including reclaimed top fill of mainly soft silty clay with grass roots as STRATUM I. STRATUM II consists of very soft silty clay with varying percentage of decomposed wood. In STRATUM III, very stiff bluish grey/brown silty clay/clayey silt with traces of kankar and micaceous fine sand are present. Dense brown silty fine sand with traces of mica and clay as binder are present in STRATUM IV.

1.2 Objective of the Work

Objective of the present study is to optimize, i.e. minimize the cost of foundation system of a medium rise building (G + 4) on a typical soft soil profile in Kolkata. After getting cost of construction of different foundation system, a comparative study has been made to choose best foundation for this particular building.

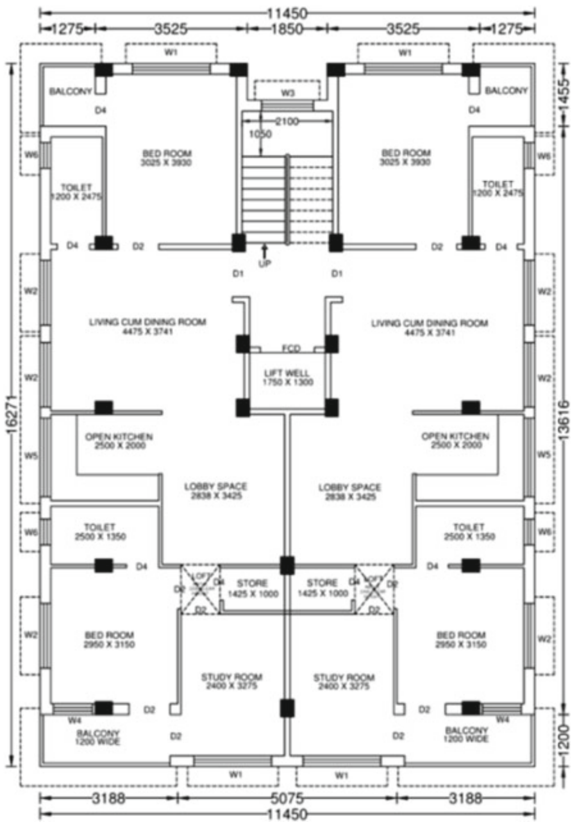


Fig. 2 Typical floor plan of the building

For solving the problem, building is modeled and analyzed by STAAD Pro for different combinations of loads. Then computer program is developed in Microsoft excel solver for designing different types of foundation system.

2 Methodology

Design of structural system should be done in a systematic manner and for a particular structural system; there are more than one solution which satisfies both safety and serviceability requirement. For designing a foundation system, some design parameters such as bearing capacity and settlement of the foundation are calculated according to IS 6403 [5] and IS 8009 (Part 1 and Part 2) [6, 7] respectively.

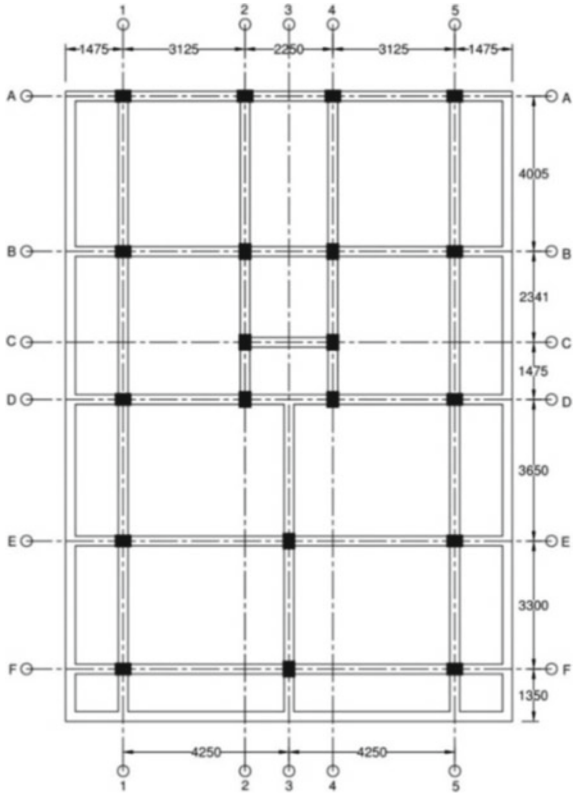


Fig. 3 Grid plan of the building

2.1 Ground Improvement Technique

Load carrying capacity of individual stone column as well as bearing capacity of improved ground is calculated according to IS 15284 [8].

2.2 Load Carrying Capacity of Pile and Pile Group

Load carrying capacity of pile and pile group in cohesive soil and granular soil is calculated in accordance with IS 2911 (Part 1) [4].

2.3 Structural Design of the Foundations

The structural design of the foundation systems was carried out as per the recommendations of IS 456-2000 [1].

2.4 Cost Analysis of Footing

Cost of the footing depends on the quantity of different items of work and the cost incurred per unit for these each item of work. Total cost of the footing is calculated by summing up the total cost incurred by concrete, steel, and formwork. The construction cost of footing can be estimated as

$$C_{\mathrm{Footing}} = Q_{\mathrm{c}} R_{\mathrm{c}} + Q_{\mathrm{st}} R_{\mathrm{st}} + Q_{\mathrm{f}} R_{\mathrm{f}}$$

where Q_{c} = Quantity of concrete = $B \times L \times D$.

Q_{st} = Quantity of steel = $A_{\mathrm{st}} \times L \times \gamma$.

Q_{f} = Quantity of formwork = $2(B + L)D$.

R_{c} = Unit price of concrete

R_{st} = Unit price of steel

R_{f} = Unit price of formwork.

The main objective of optimization is to select design variables in such a way so that the cost of the foundation will minimum. At the same time, the design variable should satisfy all the requirements and provisions specified in Indian standard.

3 Result and Discussions

The building model is done in STAAD Pro and it is analyzed to obtain the forces for which foundation is to be designed. For this analysis, following assumptions are done

i. Column section = 300 mm × 400 mm
ii. Beam section = 250 mm × 400 mm
iii. Dead load = As per IS 875 (Part I): 1987
iv. Imposed load = As per IS 875 (Part II): 1987
v. Wind load = As per IS 875 (Part III): 2015
vi. Earthquake load = As per IS 1893: 2016.

Table 1 Column load

Column marked	F_x	F_y	F_z	M_x	M_y	M_z	Remarks
A1 and A5	45.4	1136.5	32.1	40.3	0.9	47.2	
A2 and A4	55.9	939.6	27.6	35.6	0.7	53.7	
B1 and B5	33.5	1421.6	36.8	42.5	0.7	42.9	
B2 and B4	50.4	904.4	38.4	41.0	0.6	51.8	
C2 and C4	30.6	753.6	50.0	46.4	0.8	45.3	Max M_x
D1 and D5	36.4	1284.9	41.2	44.5	1.1	49.5	
D2 and D4	55.3	822.9	32.3	38.1	1.2	59.6	Max M_y
E1 and E5	45.0	1234.4	48.1	47.9	1.0	61.4	Max F_z
E3	51.3	965.3	41.3	43.8	0.9	65.6	
F1 and F5	40.6	1499.5	21.6	35.7	1.1	67.5	Max F_y
F3	60.7	1259.0	17.6	32.9	0.9	77.3	Max F_x and M_z

3.1 STAAD Pro Analysis Result

After analyze in STAAD Pro, maximum loads in columns are shorted out from different combination of load including earthquake load [2] and tabulated in Table 1.

3.2 Calculation of Allowable Bearing Capacity of Existing Soil

Net safe bearing capacity with factor of safety 2.5 for different types and size of foundation are calculated as per IS 1904 [3] and vertical load carrying capacity of different types of foundation based on net allowable bearing capacity is plotted in Fig. 4.

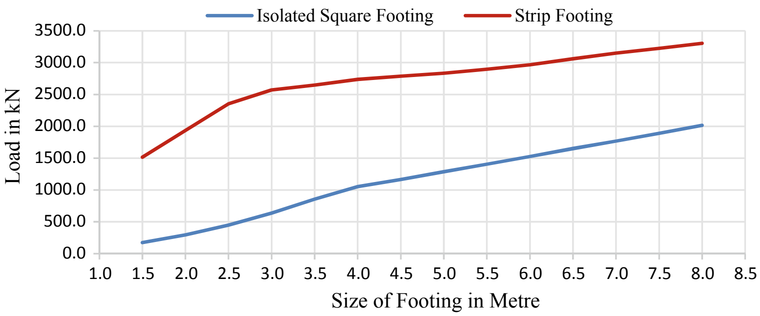


Fig. 4 Foundation size versus vertical load carrying capacity plot

Table 2 Grouping of foundation according to design load and foundation size

Group No	Foundation marked	Column No	Vertical load (kN)	Foundation size (m × m)
G1	F1	B1, B5, F1 and F5	1500	6 m × 6 m
G2	F2	D1, D5, E1, E5 and F3	1300	5 m × 5 m
G3	F3	A1 and A5	1150	4.5 m × 4.5 m
G4	F4	A2, A4, B2, B4, D2, D4 and E3	1000	4 m × 4 m
G5	F5	C2 and C4	800	3.5 m × 3.5 m

In practical situation, it is not possible to design each and every foundation for individual load. Hence, all the column loads are divided into five groups according to the vertical load only and foundations are design accordingly to support the group load. From the chart, dimension of the foundations is easily calculated and shown in Table 2.

3.3 Selection of Type of Foundation Based on Existing Condition

Bearing capacity of the soil on which these foundations are to be rest is very less. Therefore, area required for the foundation to transfer the load safely is more and it is seen from the foundation layout as shown in Fig. 5 that isolated footing is not possible for this type of soil because they overlap with each other. Again it is found that shallow foundation is not possible for this typical soil. Hence deep foundation such as pile may be adopted. But before going into the pile foundation, ground improvement technique may be adopted.

In this present study, it is seen that ***isolated footing is not possible as it overlap to each other*** as shown in Fig. 5. It is also seen that no shallow foundation is suitable for this particular type of soil without applying ground improvement technics.

3.4 Bearing Capacity After Ground Improvement by Stone Column

In soft soil, bearing capacity may be improved either by preloading with vertical drains or stone column. In this case, stone column method is considered. According to IS 15284 (Part 1) [8], capacity of a stone column is calculated with the parameter as shown in Table 3.

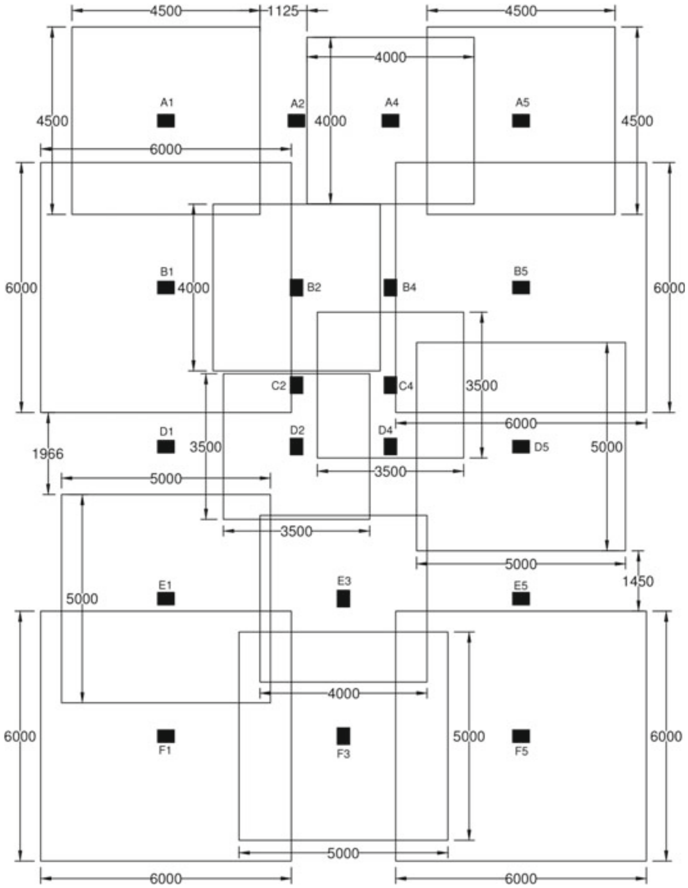


Fig. 5 Probable foundation layout of isolated footing

Table 3 Stone column parameter

S. No	Description	Value
1	Diameter of stone column	D
2	Spacing of stone column	2D, 2.25D, 2.5D, 2.75D, 3D
3	Angle of internal friction of column material	40°
4	Avg. co-efficient of lateral earth pressure for clay	0.6
5	Avg. value of cohesion	24.06 kN/m^2
6	Stone column pattern	Square

Fig. 6 Diameter of stone column versus load carrying capacity of a stone column plot

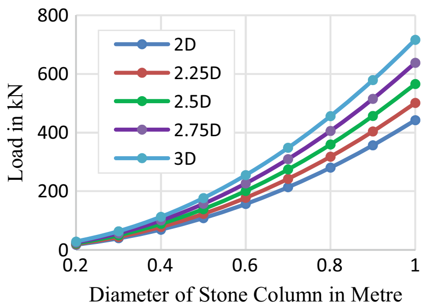
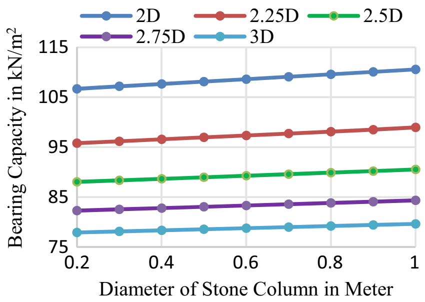


Fig. 7 Diameter of stone column versus bearing capacity of soil plot



Load carrying capacity of a stone column mainly depends on its diameter for a particular soil. A load carrying capacity of a stone column and bearing capacity of soil versus diameter of the stone column with different spacing is plotted in Figs. 6 and 7.

From Fig. 7, assume diameter of the stone column 0.5 m with a spacing of 2D, i.e. 1 m center to center for this building. Stone columns are used in square pattern. One unit cell of stone column carries a load of 108 kN.

Bearing capacity of treated soil may be calculated by dividing the load carrying capacity of unit stone column by its tributary area and is found as 108 kN/m^2. In case of ground improvement by stone column in a particular soil, bearing capacity mainly depends on the spacing between the stone columns as shown in Fig. 7.

3.5 Load Carrying Capacity of Pile and Pile Group

Capacity of Single Pile. Capacity of a single pile of different length in typical given soil is calculated as per IS 2911 [4] and shown in Fig. 8.

Adopt pile of length 20 m for this building and diameter of the pile is selected from Fig. 8 according to the required load carrying capacity of the pile. Use pile

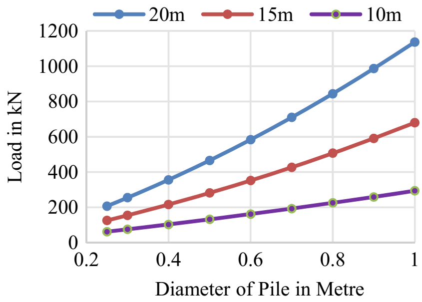


Fig. 8 Diameter of pile versus load carrying capacity of single pile plot

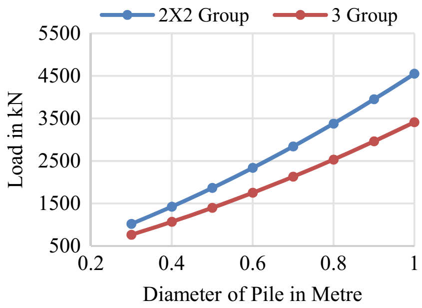


Fig. 9 Diameter of pile versus capacity of pile group plot

diameter of 0.45 m and load carrying capacity of single pile is 466 kN including its self-weight.

Capacity of Group Pile. Capacity of pile group is estimated assuming 20 m length of pile. Pile group capacity for different diameters of pile is shown in Fig. 9 with a spacing of 2D, i.e. 900 mm C/C. It is seen that group capacity of pile for this particular soil is always greater than the sum of the capacity of individual pile in a group.

3.6 Design of Foundation System (Isolated Footing) on Stone Column

After improving the soil parameter by stone column, it is possible to provide isolated footing as one solution for this typical soil profile. Now optimum dimension of isolated foundation is found out to transfer the column load safely. Size of the footing on stone column for different column loads is tabulated in Table 4.

Pre-assign Parameter for Design of Shallow Foundation. Isolated footing is designed, taking the following input parameters as constant

i. Grade of concrete and steel = M25 and Fe 500

Table 4 Size of the isolated footing on stone column

Group No	Foundation marked	Column No	Design load (kN)	Foundation size (m × m)
G1	F1	B1, B5, F1 and F5	1500	3.0 m × 3.0 m
G2	F2	D1, D5, E1, E5 and F3	1300	2.8 m × 2.8 m
G3	F3	A1 and A5	1150	2.6 m × 2.6 m
G4	F4	A2, A4, B2, B4, D2, D4 and E3	1000	2.4 m × 2.4 m
G5	F5	C2 and C4	800	2.2 m × 2.2 m

ii. Clear cover = 50 mm
iii. Diameter of reinforcing bar = 16 mm
iv. Pedestal size = 450 mm × 450 mm
v. Safe bearing capacity = 110 kN/m^2
vi. Weight of the footing and backfill—10% of the axial load.

Design Constrains. For designing isolated footing fixed constrains are size of the footing, i.e. length and breadth and variable constrains are depth of footing and percentage of reinforcement.

Rate of Different Item of Work Related to Shallow Foundation. Cost of different item of work is calculated as per West Bengal PWD, Schedule of Rates 2017 [9] for Kolkata. To get the current rate of different item of work a multiplying factor, i.e. cross index is applied with this rate as per WBPWD norms. Standard rate of the stone column is not specified in the WBPWD schedule, so site specific analysis of rate is carried out and calculated rate of the stone column of 450 mm diameter for per meter length is Rs. 995.90.

Different Design Options for Shallow Foundation. Design is carried out for different thickness and percentage of steel to obtain optimum solution.

From Fig. 10, it is seen that the optimum cost is Rs. 34,273, Rs. 27,899, Rs. 23,026, Rs. 18,589, and Rs. 14,137 for a footing depth of 500 mm, 420 mm, 400 mm, 390 mm, and 350 mm respectively.

3.7 Design of Foundation System (Pile) in Untreated Soil

Pile foundation gives a general solution for all type of foundation problem. For this building, optimum cost of the pile foundation is calculated. Different design options for pile cap are determined with the help of Microsoft excel spread sheet and presented below.

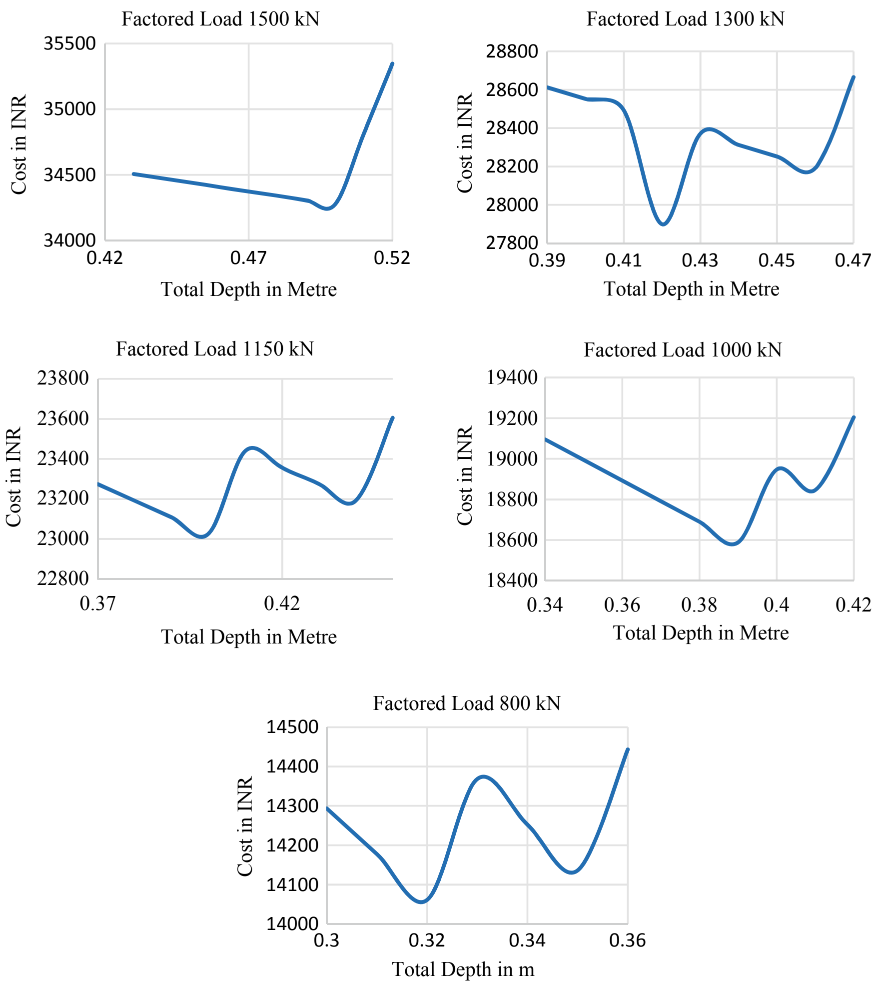


Fig. 10 Cost versus total depth of isolated footing plot for different factored load

Design Constrains. For designing pile length and diameter of the pile are fixed constrains. In case of designing of pile cap, fixed constrains are size of the pile cap and variable constrains are depth of the pile cap and percentage of reinforcement.

Design and Cost of Pile. Pile is designed as slender column with the following input parameters

i. Axial load on pile = 466 kN
ii. Length and diameter of pile = 20 m and 450 mm
iii. Depth of fixity = 7.44 m (Calculated as per IS 2911 [4])
iv. Clear cover to the reinforcement = 50 mm
v. Diameter of longitudinal bar and lateral tie = 20 and 8 mm

Table 5 Design details of a single pile

S. No	Description	Unit	Rate	Quantity	Cost
1	Pile of 450 mm diameter and 20 m length	m	1536.00	20	30,720
2	Steel reinforcement of grade Fe500				
	(a) 12 nos. longitudinal bar (16Φ)	kg	58.30	378.80	22,084
	(b) 8Φ lateral tie @ 150 mm c/c			54.50	3177
	(c) 16Φ spacer bar @ 1.5 m c/c			22.88	1334
Total					57,315

vi. Grade of concrete and steel = M25 and Fe 500.

Pile has been designed with the above mention input parameters. The design output parameters along with the total cost of the pile are tabulated in Table 5. Actual cost of the pile per ‘m’ length is obtained by adding cost of concrete for 1 m length of pile.

Pre-assign Parameter for Design of Pile Cap. Pile cap is designed to transfer the load from the column to the pile. It should be rigid. For designing of pile cap, following pre-assign parameters are given below.

i. Spacing between piles = 900 mm
ii. Minimum depth of pile cap = 300 mm
iii. Overhang of the pile cap from pile = 150 mm
iv. Cover to the reinforcement = 75 mm
v. Grade of concrete and steel = M25 and Fe 500.

Different Design Options for Pile Cap. Design is carried out for different thickness and percentage of steel to obtain optimum solution.

From Fig. 11, it is seen that the optimum cost is Rs. 15,839, Rs. 16,934, Rs. 15,948, Rs. 14,749, and Rs. 7669 for pile cap of depth of 490 mm, 540 mm, 520 mm, 520 mm, and 360 mm, respectively.

3.8 Comparison Between Two Types of Foundation System

From Fig. 12, it is seen that cost of the shallow foundation over stone column is 43.16% less than the cost of the pile foundation (Tables 6 and 7).

4 Conclusions and Future Scope

In this study, concluding remarks are also drawn based on the application of methodology and developed computer program for cost minimization of foundation system.

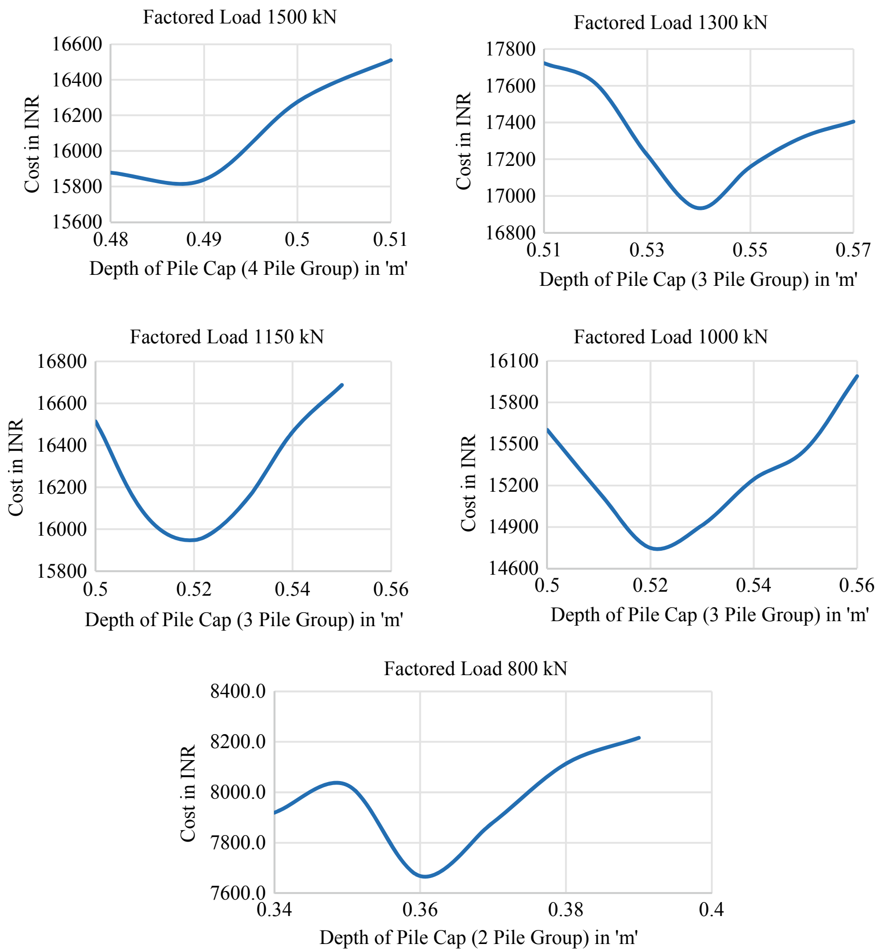


Fig. 11 Cost versus total depth of pile cap plot for different factored load

Main objective of this study is to minimize the cost of different type of foundation system for a typical medium rise residential building (G + 4) on typical soil profile. Cost of the footing is primarily depending on the area of footing and area of reinforcement. The following conclusions and future scope of this study are discussed below.

In this study, ground improvement method considered is installation of stone columns. Similar study can be carried by other type of ground improvement techniques. The present study is carried out in vertical axial load on footing. The study can be carried out for different loading like lateral load, moment, etc. Flat-type isolated square footing has been considered in this study. Similarly, it can be carried out other type and shape of footing.

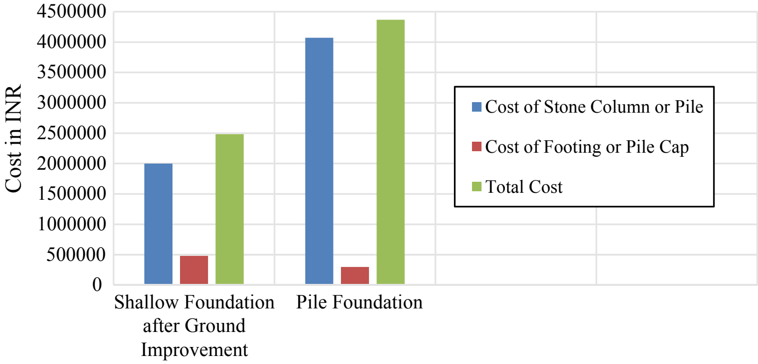


Fig. 12 Comparison between cost of shallow foundation over stone column and pile foundation

Table 6 Total cost of foundation system (isolated footing) with stone column

S. No	Description	Rate	Quantity	Cost
1	500 mm diameter stone column	9959	205	2,001,759
2	Isolated footing of			
	Size for column in Group 1	34,273	4	137,092
	Size for column in Group 2	27,899	5	139,495
	Size for column in Group 3	23,026	2	46,052
	Size for column in Group 4	18,589	7	130,123
	Size for column in Group 5	14,137	2	28,274
Total cost				2,482,795

Table 7 Total cost of foundation system (pile foundation) without stone column

S. No	Description	Unit	Rate	Quantity	Cost
1	450 mm diameter pile	per Pile	57,315	71	4,069,365
2	Pile cap of				
	4 pile group for Group 1	per Cap	15,837	4	63,348
	3 pile group for Group 2	per Cap	16,934	5	84,670
	3 pile group for Group 3	per Cap	15,948	2	31,896
	3 pile group for Group 4	per Cap	14,749	7	103,243
	2 pile group for Group 5	per Cap	7669	2	15,338
Total cost					4,367,860

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Strength and Compaction Behavior of Randomly Distributed Polypropylene Fiber-Reinforced Expansive Clay

Brijesh K. Agarwal, Shyam A. Hathiwala, and C. H. Solanki

Abstract Experimental investigations were conducted to study the effects of randomly distributed polypropylene (PP) fiber inclusions on the mechanical behavior of expansive soil. Reinforced soil specimens were prepared at four different fiber contents (0.05, 0.1, 0.15 and 0.2%), and the aspect ratio of fibers (*L*/*D*) was kept as 250. A series of compaction tests, unconfined compressive strength (UCS) and split tensile strength (STS) tests were performed on the unreinforced and fiber-reinforced soil specimens. The results proved that the UCS and STS values increased to a greater extent with the inclusion of fibers to the expansive soil. The inclusion of monofilament-type PP fibers within expansive soil contributes to increase the peak axial stress, improve the residual strength and increase the modulus of elasticity, toughness and ductility of the soil. It was noticed that the effect of polypropylene fiber inclusion on the compaction parameters was not much significant (less than 5% variation) due to lightweight and less water absorption capacity of PP fibers. The highest UCS values were obtained with 0.15% fiber content with 12 mm length of fibers for that UCS values increased up to 51% of that of the unreinforced soil. Similar behavior was also observed for STS of soil–fiber mixtures with a gain of 59% in tensile strength. From the UCS and STS test results, some other parameters like secant modulus, shear modulus, resilient modulus and deformability index were also reported for both unreinforced and reinforced specimens. It was seen that secant modulus of expansive soil increased up to 89% and resilient modulus was increased up to 17% on addition of 0.15% fibers. Similarly, other parameters were also improved with the inclusion of PP fibers within the expansive soil.

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Keywords Monofilament fibers · Split tensile test (STS) · Resilient modulus · Secant modulus · Deformability index

1 Introduction

Civil engineering structures transfer their load on the soil mass through foundation. Construction of embankments, backfill of retaining walls, highway subgrades and landfill liners and covers require utilization of locally available soil due to high transportation charges in bringing the soil from some other site [1]. Sometimes we come across such soils at project site which cannot be used as a construction material due to their low bearing capacity and shear strength, e.g., black cotton soil. In that case we have to improve the soil quality for its further use.

This problem leads to the development of various ground improvement techniques either by using a chemical stabilization or a mechanical stabilization [18]. Strengthening of weak soil by adding chemical additives such as cement, lime, ground granulated blast furnace slag, bagasse ash and fly ash has been suggested by many researchers in past [3, 8]. Another technique that is mechanical stabilization falls under the category of soil reinforcement. Soil reinforcement technique has been used since ancient times in the form of roots of tress, bamboos, straws, etc. There are many ancient structures which are built using soil reinforcement technique like the Ziggurats of Mesopotamia and Great Wall of China [12].

Reinforcement of soil not only improves its compressive strength but also its tensile strength to a great extent. Reinforcement reduces the brittle behavior and post peak strength losses in the soil [7]. Reinforcement technique is also classified into two categories: One is systematic reinforcement and another is randomly distributed reinforcement. Review of the literature reveals that random inclusion of fibers is more advantageous as compared to systematically oriented reinforcement because the latter creates weak zones in the soil mass. When a tensile crack appears into the soil due to shrinkage or loading, the fiber is supposed to prevent the propagation of crack [15]. Some researchers tried natural fibers like coir fiber, jute fiber with soil and some tried synthetic fibers like polyethylene, PET, polypropylene, etc. Both natural and synthetic fibers increase the compressive as well as tensile strength of soil, but natural fibers are more prone to decay with time when buried under moist soil. Nowadays, fiber-reinforced soil is being used in various civil engineering projects like in landfill liners for crack control, in pavement applications, behind the retaining wall as a backfill material, in slope protection works and below shallow footings [4].

In this study, a series of laboratory investigations were carried out to evaluate the performance of monofilament polypropylene fibers with expansive black cotton soil. Unconfined compressive stress (UCS), split tensile strength (STS) and standard proctor tests were performed to study the strength gain, failure pattern and compaction parameters of unreinforced and reinforced soils. A variety of soil indexes and modulus like secant modulus, resilient modulus, deformability index and shear modulus were also reported based on the UCS results.

Table 1 Physical properties of soil used in the study

Property	Value
Specific gravity	2.62
Grain size analysis	
Gravel (%)	0
Sand (%)	7
Silt (%)	72
Clay (%)	21
Consistency limits	
Liquid limit (%)	61.2
Plastic limit (%)	24.6
Plasticity index (%)	36.6
IS classification	CH
Compaction parameters	
OMC (%)	25.5
MDD (g/cc)	1.448

2 Materials Used

2.1 Expansive Soil

The expansive soil used in the present study was collected from Student Activity Center ground of SVNIT Campus, Surat. The soil is black cotton soil in appearance, and it is classified as CH soil as per IS 1498-1970. The physical properties of this soil are shown in Table 1.

2.2 Polypropylene Fibers

Polypropylene is a commercially available synthetic fiber. Monofilament-type fibers were used in this study. Physical and mechanical properties of these fibers are given in Table 2.

3 Experimental Program

To study the effects of randomly distributed PP fiber addition in black cotton soil, virgin black cotton soil was mixed with four different dosages of fiber (i.e., 0.05, 0.1, 0.15 and 0.2%). The percentage of fibers and length of fibers was decided based on the previous studies carried out using polypropylene fibers. The fiber content (C_f) is calculated as per Eq. (1).

Table 2 Physical properties of polypropylene fiber used in the study

Property	Value
Average tensile strength (MPa)	340
Avg. diameter (mm)	0.048
Moisture absorption (%)	0–0.04
Melting point (°C)	167
Chemical resistance	Good
Unit weight (g/cc)	0.90
Electrical insulation	Excellent
Average length	12 mm

$$C_{\mathrm{f}} = \frac{w_{\mathrm{f}}}{w} \tag{1}$$

where w_{f} is the weight of fibers and w is the dry weight of soil. Hand mixing method was adopted to get a uniform mixture of soil and fibers. The fibers were in the form of bundles which opens up on rubbing action against moist soil. Mixing of fibers with moist soil gave more uniform texture than with dry soil, so first predetermined amount of water was added and mixed to get a uniform color of soil, and then fibers were sprinkled over it and mixed by hand mixing.

First, compaction tests were performed so that UCS and STS samples can be prepared at predefined optimum moisture content and dry density. UCS and STS specimens were prepared having 100 mm height and 50 mm diameter. Later UCS and STS tests were conducted on both reinforced and unreinforced expansive soil. Split tensile test apparatus is used as shown in Fig. 1. The apparatus used in this study is made in accordance with many previous researchers [2, 6, 9, 11, 16]. Based

Fig. 1 Split tensile test apparatus

on the above research papers, a metal loading strip was prepared as shown in Fig. 1. The thickness of the strip is 1.778 mm and width is 6.35 mm such that width/sample diameter ratio is kept 1/8 for proper distribution of load. The loading was applied using an UCS testing machine at a rate of 1.25 mm per minute. The soil sample was kept horizontally below the loading frame as shown in Fig. 1. The failure load is recorded. The tensile strength of the soil is calculated as per Eq. (2).

$$\text{Split Tensile Strength (kPa)} = \frac{2P}{\pi l d} \tag{2}$$

where l is the length of the specimen, P is the load at failure, and d is the diameter of the specimen. The STS value is calculated for three identical specimens, and an average value is reported here.

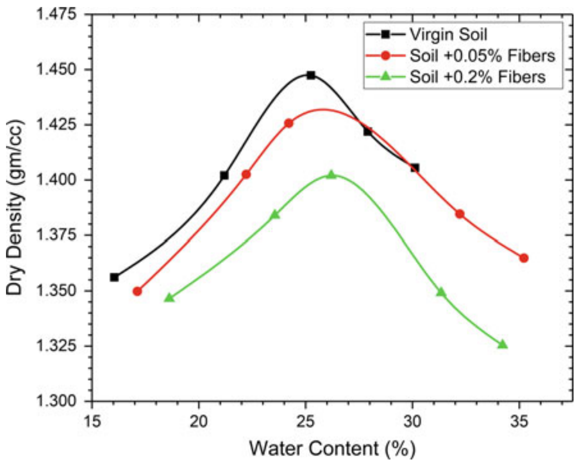
4 Results and Discussion

As discussed in previous sections, a number of soil samples with and without fiber reinforcement were tested to study their strength and compaction behavior. The results of compaction, UCS and STS are presented in the subsections.

4.1 *Effect of Fibers on Compaction Behavior*

Figure 2 presents the results of standard proctor test conducted on untreated and treated expansive soils. These tests were conducted to determine the optimum moisture content (OMC) and maximum dry density (MDD) of the soil samples to be

Fig. 2 Variation of dry density with water content



prepared for UCS and STS tests. The curves show that MDD decreases and OMC increases on addition of fiber to the expansive soils. Only two fiber contents were analyzed (i.e., 0.05 and 0.2%) which are minimum and maximum fiber dosages. It can be concluded from the curves that even up to maximum fiber addition there is a slight variation in the OMC and MDD of expansive soils. For untreated soil, OMC and MDD are 25.5% and 1.448 g/cc, respectively, which on addition of 0.2% fiber content changes to 26.2% and 1.420 g/cc, i.e., less than 5% variation. So there is not a significant effect of fibers on the compaction behavior of expansive soil. This can be attributed to the lightweight and less water absorption tendency of polypropylene fibers.

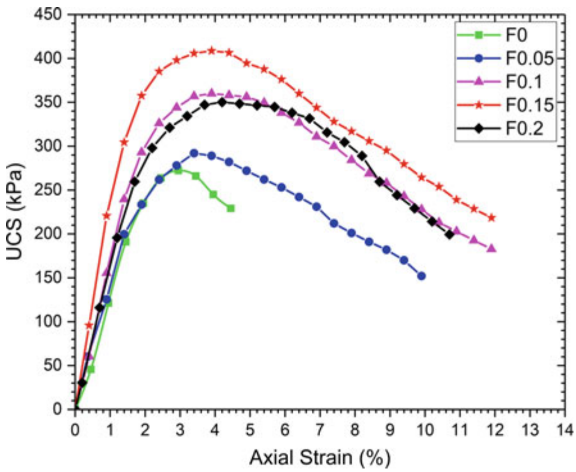
4.2 *Effect of Fiber Addition on UCS and Failure Pattern*

Table 3 shows the variation of UCS with variation in fiber dosages and also the gain in UCS values which is defined as a ratio of increase in UCS of reinforced soil to the UCS of unreinforced soil are presented along with. It is observed that the UCS increased with increase in fiber dosages up to 0.15% and then it decreased. According to the principles of fiber reinforcement of soil, the applied load is transferred to the interface between soil and fibers through friction [11]. As we increase the fiber content in the soil, the frictional interface between soil and fiber increases which contribute the increased shear strength of the soil. Another contribution is due to high tensile strength of fibers. They act like tree roots within the soil mass. Peak compressive strength of untreated soil is 269 kPa which increased up to 408 kPa (i.e., 51.67% gain in UCS). It can be seen from Fig. 3 that ductility of soil samples was increased due to inclusion of fibers to the soil. Also post peak strength losses were reduced with increased fiber content. The reduction in UCS after 0.15% fiber addition can be attributed to "balling effect". Balling effect is due to non-uniform mixing of fibers. After 0.15% fiber addition, it was not easy to uniformly mix the fibers with soil that causes lump formation which is called balling effect. These lumps are highly compressible and make soil more compressible.

Table 3 Results of UCS tests

Sample ID	UCS (kPa)	STS (kPa)	Gain in UCS (%)	Gain in STS (%)
Soil	269	35.27	–	–
Soil + 0.05% fiber	293.5	40.25	8.70	14.12
Soil + 0.1% fiber	360.48	48.38	33.51	37.17
Soil + 0.15% fiber	409.5	56.02	51.67	58.83
Soil + 0.2% fiber	350	50.2	29.63	42.33

Fig. 3 Stress–strain behavior for unreinforced and fiber-reinforced soil



4.3 *Effect of Fiber Inclusion on STS*

Split tensile strength of unreinforced and reinforced soil with different fiber dosages is shown in Fig. 4. For untreated soil, STS is found 35.27 kPa, which got increased up to 56.02 kPa on addition of 0.15% fibers (i.e., 58.83% increment). It shows that fibers are more effective in increasing the tensile strength with respect to the compressive strength of unreinforced soil. The gain in tensile strength of soil–fiber mixture can be credited to high tensile strength of polypropylene fibers. Moreover, fibers work as a bridging agent between crack openings and reduce the further propagation of cracks, and this effect is called the "bridging effect" of fibers as shown in Figs. 5 and 6.

Fig. 4 Variation of STS with fiber content

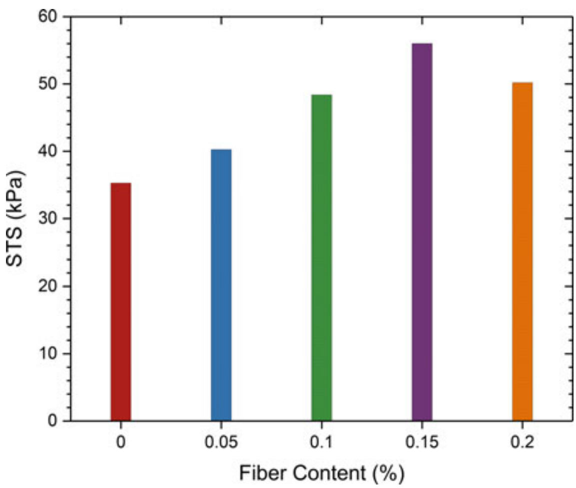


Fig. 5 Bridging effect of fibers

Fig. 6 Failure pattern of specimen in STS test

4.4 Effect of Fiber Inclusion on Deformability Index of Soil

It is an index that reciprocates ductility and brittleness behavior of materials. It is defined as per Eq. (3).

$$I_{\mathrm{D}} = \frac{\varepsilon_{\mathrm{r}}}{\varepsilon_{\mathrm{u}}} \tag{3}$$

where ε_{r} is axial strain conforming to peak UCS value for reinforced specimen and ε_{u} is axial strain conforming to peak UCS value for unreinforced soil. It can be noticed from Table 4 that deformability index increased with increase in fiber content which confirms that the addition of fiber makes soil more ductile in its failure behavior.

4.5 Effect of Fiber Inclusion on Resilient Modulus of Soil

Resilient modulus is associated with elastic response of soil to the given stresses. In the design of pavement layers, resilient modulus of subgrade materials plays a key role. The ratio of cyclic deviator stress to the resilient strain is defined as resilient

Table 4 Various modulus and indexes calculated using UCS value

Sample ID	Secant modulus (MPa)	Resilient modulus (MPa)	Shear modulus (MPa)	Deformability index (I_D)
Soil	12.62	102.28	4.21	–
Soil + 0.05% fibers	14.02	105.19	4.67	1.16
Soil + 0.1% fibers	17.11	113.50	5.70	1.29
Soil + 0.15% fibers	23.86	119.58	7.95	1.35
Soil + 0.2% fibers	16.37	112.20	5.46	1.43

modulus. It reciprocates the stiffness of the pavement materials and helps in pavement design [17].

Resilient modulus can be calculated from UCS value of soil as per Eq. (4) given by Thompson [16].

$$M_r\,(\text{MPa}) = 0.124 \times \text{UCS}\,(\text{kPa}) + 68.8 \tag{4}$$

The variation of resilient modulus with dosages of fiber is given in Table 4. It can be noticed that M_r for unreinforced soil is 102.28 MPa which is increased up to 119.58 MPa on addition of 0.15% fiber content (i.e., 16.91% increment in M_r).

4.6 Effect of Fiber Addition on Secant Modulus of Soil

The ratio of half of peak compressive stress to the conforming axial strain is defined as the secant modulus [5]. The secant modulus for unreinforced and fiber-reinforced soil is given in Table 4. It can be seen from the table that for unreinforced soil the value of secant modulus is 12.62 MPa which is increased on addition of polypropylene fibers up to 23.86 MPa for fiber content of 0.15% (i.e., 89% increment in secant modulus of soil). Further addition of fibers shows a reduction in secant modulus value because of their balling effect.

4.7 Correlation Between Tensile and Compressive Strength

A linear relationship was found between unconfined compressive strength (kPa) and split tensile strength (kPa) of soil–fiber specimens. Figure 7 shows the relationship between STS and UCS of fiber-reinforced specimens with R^2 value of 0.97.

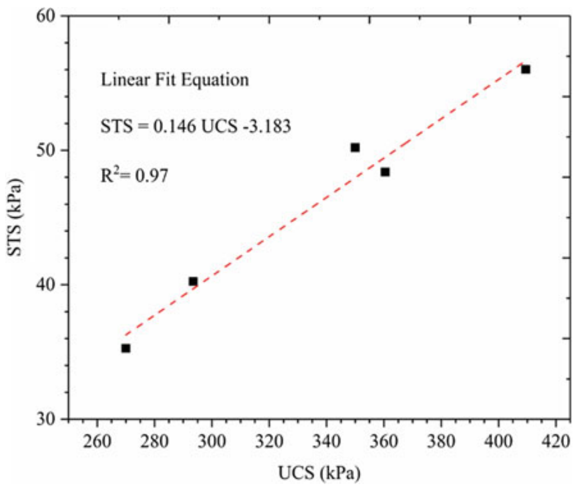


Fig. 7 Correlation between STS and UCS

4.8 *Effect of Fiber Addition on Shear Modulus of Soil*

Response of a soil mass to the stresses and design of any structure on it depends largely upon its shear modulus value, ignorance of which causes a lot of damage to structures [13]. Shear modulus of each combination of soil and fiber is calculated based on the following relation given in Eq. (5) [10].

$$G\ (\text{MPa}) = \frac{\sigma_{xy}}{\varepsilon_{xy} + \varepsilon_{yx}} = \frac{\sigma_{xy}}{2\varepsilon_{xy}} = \frac{\sigma_{xy}}{\gamma_{xy}} = \frac{E_\text{s}\ (\text{MPa})}{2(1+\nu)} = \frac{E_\text{s}\ (\text{MPa})}{3} \tag{5}$$

where E_s = secant modulus, ε is the shear strain, and σ_{xy} is the shear stress, and $\gamma_{xy} = \varepsilon_{xy} + \varepsilon_{yx} = 2\varepsilon_{xy}$ [14].

A change in secant modulus with fiber dosage is shown in Table 4. It was seen that shear modulus of soil is 4.21 MPa which is increased up to 7.95 MPa (approx. 89% increment) on addition of 0.15% PP fibers, and on further addition of fibers it decreases.

5 Conclusions

Based on the experimental investigations conducted on unreinforced and reinforced soil specimens, following conclusions are drawn:

- Effect of polypropylene fibers on compaction parameters is insignificant due to less than 5% variation in OMC and MDD on addition of 0.2% fiber content.
- Compressive strength of soil first increases on inclusion of fibers and after optimum fiber content it decreases due to balling effect. Maximum UCS is

obtained for 0.15% fiber content with a gain in UCS of 51.67% w.r.t. unreinforced soil.

- Due to inclusion of fibers, split tensile strength of soil increases by 58.83% on addition of 0.15% fiber content. It shows that the effect of fibers is more on tensile strength than compressive strength of soil–fiber specimens.
- A linear correlation between STS and UCS of soil fiber specimens is obtained with an R^2 value of 0.97.
- Resilient modulus, secant modulus and shear modulus of soil–fiber specimens increase by 89%, 16.91% and 88.83%, respectively, on addition of 0.15% PP fibers into the soil.
- Deformability index of soil goes on increasing with rise in fiber content which demonstrates that fiber addition makes the failure pattern of soil more ductile.

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Stabilization of Black Cotton Soil Using Calcium Carbide Residue

Mahesh Endait, Suyash Wagh, and Shubham Kolhe

Abstract A wide variety of soils are available in the world out of which some are good in views of construction engineers and few are problematic due to their swelling and shrinkage properties, and black cotton soil (BCS) is one of them. These properties can easily produce uplift movement and may lift the lightweight civil engineering structures and causing collapse, cracking and ultimately hazard to mankind. In such a case, there is an extreme need for soil stabilization and hence to find and introduce cheap and easily adoptable stabilizing agent. Calcium carbide residue (CCR) which is a by-product of acetylene gas manufacturing industry due to its alkaline properties can be effectively used as stabilizing material, and reducing the environmental problem of its disposal is taken into the frame. This study depicts the use of CCR in stabilizing BCS. Introducing CCR in BCS can increase the strength and swell property of soil, for this CCR fixation point was determined by sequentially adding 1–10% CCR to BCS and studying properties like pH and consistency limits. Strength and swell pressure of BCS were checked with varying percentage of CCR. To study strength properties, unconfined compression test is conducted, and to examine swell properties a portable and in-house fabricated swell pressure measuring device was used. Maximum strength development in stabilized clay is found at 7% CCR cured for 28 days at standard temperature. It was further observed that addition of CCR to BCS resulted in increased strength of about 5 times than that of virgin BCS and it was observed that stabilization of soil with 7% CCR reduces its swell pressure by around 88%.

Keywords Calcium carbide residue · Stabilization · Expansive soil

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1 Introduction

Increasing population in the world, especially developing nations, has led to increasing demand for roadways, railways, housing facilities and other infrastructures. Soil with higher stability is required to bear the weight of these structures [13]. Generally speaking, the stability of any construction-related structure directly or indirectly depends on soil stability. India is home to various geographical features such as rivers, mountains, valleys, tablelands, seashores, deserts and flat terrains, due to which a huge variety of soils like alluvial, red, mountain, black cotton and laterite soils are found in India. Basically mentioning black cotton soil, geographically black cotton soils are spread over 5.0 lakh km^2 [20] (i.e., 16.6% of the total geographical area of the country).

The black cotton soil is very hard when dry, but loses its strength when in wet condition. This property of black cotton soil is seen due to the presence of high clay mineral content of montmorillonite. Because of high swelling and shrinkage characteristics of black cotton soil, these have been a challenge to the construction field. Special attention is gained by the development of cracks of varying depth in highways and other structures due to settlement, heavy depression, cracking and unevenness, and wetting and swelling characteristics of this soil type [14]. As such, black cotton soil has very low bearing capacity and high swelling and shrinkage characteristics.

Considering problems encountered in the field of construction, many stabilization techniques have been used in the past to improve their engineering properties such as cement stabilization, chemical stabilization by using lime, cement etc. [15, 18, 19], and mechanical stabilization by using rollers, rammers, vibrators, etc. [1]. Other deep foundation techniques may prove to be uneconomical because of the input required and procedures involved [17]. Hence, many researchers have given their emphasis on study and utilization of industrial by-products for stabilizing such expansive soils, such as blast furnace slag, fly ash, stone dust, recycled materials to reduce their impact on the environment [3, 5, 21]. Utilization of industrial wastes in recent times has seen wide importance for stabilization like utilization of ground granulated blast furnace slag, cement wastes, bag gash ash, bassanite [2, 3, 9, 12]. One of these stabilization techniques can include the utilization of calcium carbide residue (CCR), an industrial by-product generated from acetylene gas manufacturing industries.

Nowadays, acetylene gas manufacturing industries are facing the problem of dumping this by-product (CCR) as shown in Fig. 2. So, the CCR is very cheaply available around these industries. CCR contains around 76% of lime; hence, it is a good binding agent. Hence, it proves to be an economical source of stabilizing agent and incorporated in the present study. Acetylene gas is produced when calcium carbide reacts with water, which evolves acetylene gas, heat and hydrated lime slurry which is further dried in the beds, and the calcium carbide residue is obtained which is highly basic in nature. The following reaction occurs when calcium carbide reacts with water [5, 7].

$$CaC_2 + 2H_2O = C_2H_2 + Ca(OH)_2 \quad (1)$$

This CCR is creating a dumping issue and thereby creating environmental hazards. Hence to treat expansive soils such as black cotton soil whose expansive properties are due to the presence of clay mineral montmorillonite, CCR as a stabilizing agent can prove to be an efficient technique [10, 11].

The CCR obtained is mixed with the soil in different percentages (2, 4, 6 and 8) where the soil samples collected from the field are mixed with the soil and maximum dry density and optimum moisture content are obtained. This optimum moisture content is used for preparing the unconfined compressive strength of samples. The test shows that 8% dose of CCR to soil gives 5 times increase in unconfined compressive strength. Effect of a varied percentage of CCR on density and swell pressure is also investigated and discussed in this paper.

2 Soil Sample

A natural disturbed soil sample was collected from the Nashik region in Maharashtra, India. The location is part of the area covered by the expansive soil commonalty known as black cotton soils of central India. The region is part of the Western Deccan Volcanic Province and is believed to be the youngest basalt rock of the Eocene age [8].

The natural water content of the soil was 34%, while hygroscopic moisture was 8–9%. Figure 1 shows the grain size distribution of the soil sample collected. It is composed of 8% sand, 44% silt and 48% clay. The specific gravity of soil was 2.68. The liquid and plastic limits were 64% and 24%, respectively. According to the Unified Soil Classification System (USCS), the soil was classified as clay with high plasticity (CH). Free swell ratio (FSR) was determined by taking the ratio of equilibrium sediment volume of 10 g of oven-dried soil passing through 425 μm

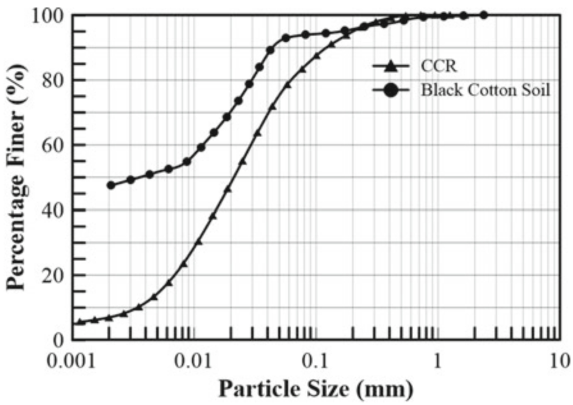


Fig. 1 Grain size distribution of silty clay and CCR

Table 1 Chemical property of silty clay and CCR

Chemical composition (%)	Black cotton soil	CCR
SiO_2	48.50	4.6
TiO_2	1.48	0.009
Al_2O_3	20.46	0.49
Fe_2O_3	15.73	0.10
MnO	0.14	0.002
MgO	4.38	0.22
CaO	9.60	90
Na_2O	1.54	0.06
K_2O	3.29	0.002

sieve in distilled water to that in kerosene [16] and found to be 1.55. Based on FSR, soil was classified as montmorillonitic with moderate expansivity. Table 1 shows the chemical composition of the soil sample. The presence of SiO_2, Al_2O_3 and Fe_2O_3 was high enough for the pozzolanic reaction.

3 Calcium Carbide Residue

Calcium carbide residue (CCR), a grayish-white powder, was obtained from Swastik Industrial Gas Manufacturing Pvt. Ltd., Nashik, India. CCR was collected from the disposal area in dry form as shown in Fig. 2b. The CCR was oven-dried at 105 °C for 24 h and ground in Los Angeles abrasion machine. The CCR was passed through 425 µm sieve for further use in experimentation. The specific gravity (Gs) of CCR was 2.30. Table 1 also shows the chemical composition of CCR. The presence of CaO content (90%) and other pozzolanic materials (SiO_2, Al_2O_3 and Fe_2O_3) indicates the suitability of CCR to produce cementitious material.

(a) (b)

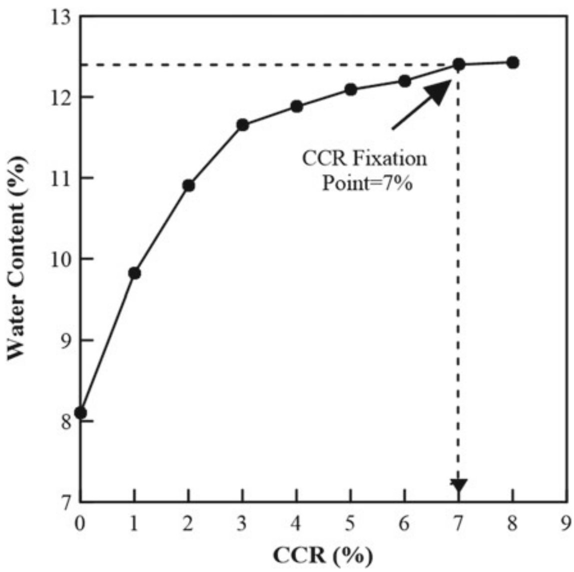
Fig. 2 Disposal of calcium carbide residue: **a** slurry form; **b** dry form

4 Properties of Stabilized Clay

4.1 Optimum Percentage of CCR

Two independent methods proposed by Eades and Grim [4] and Horpibulsuk et al. [6] were used to obtain an optimum dose of CCR. The first method is suggested for lime stabilization, in which a minimum pH value of 12.4 is necessary to initiate the pozzolanic reaction. The pH values measured for soil sample for the various percentages of CCR are shown in Fig. 3. As the CCR content increases, the pH of stabilized clay significantly increases. However, when CCR content is greater than 7% the pH was observed to be 12.4 and change in pH is minimal. This point thus can be chosen as an optimum percentage of CCR. Figure 4 shows the change in consistency indices of stabilized clay. Liquid limit (LL) was observed to be increased in proportion with increase in CCR content. However, the plastic limit (PL) was observed to decrease the resulting decrease in plasticity index (PI). The decrease in PI was minimal after 7% CCR content. These observations are in line with the observation made by Horpibulsuk et al. [6]. Based on the above two independent methods, 7% CCR content was chosen as the optimum percentage for stabilization of soil.

Fig. 3 pH of CCR stabilized soil



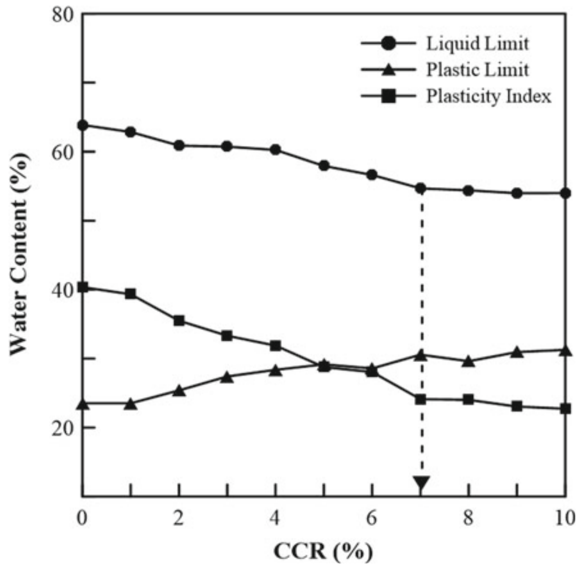


Fig. 4 CCR fixation point

4.2 Compaction Characteristics

The clay sample was sieved through 20 mm sieve and air-dried for compaction test. Compaction was carried out using standard Proctor test. The compact effort for the test consists of the energy derived from a 2.5 kg rammer falling through 30 cm onto three layers of soil, each receiving 25 blows. Compaction characteristics, i.e., optimum moisture content (OMC) and maximum dry unit weight (γ_{dry}) for clay stabilized with 7% CCR, were 28% and 14.4 kN/m^3. Having obtained compaction characteristics, the air-dried clay sample was mixed with various contents of CCR (0–8%). Figure 5 shows the compaction curves obtained from standard compaction test on clay and CCR stabilized clay. The OMC was found to be increased by 3%, and γ_{dry} was decreased by 6.5% at an optimum percentage of CCR. Lower specific gravity and affinity of CCR toward water reduce the γ_{dry}.

4.3 Unconfined Compression Strength

An unconfined compression strength test was conducted on clay and stabilized clay with CCR. Specimens were prepared by mixing the desired proportions of distilled water, soil and CCR. Percentages of CCR ranged from 0 to 8% by weight of dry soil. The soil–CCR mixtures were prepared by first thoroughly mixing dry predetermined quantities. The required amount of water determined from the compaction curve was later added to the dry mix and compacted at the energy levels of the standard Proctor. Specimens were cured for 7, 14 and 28 days.

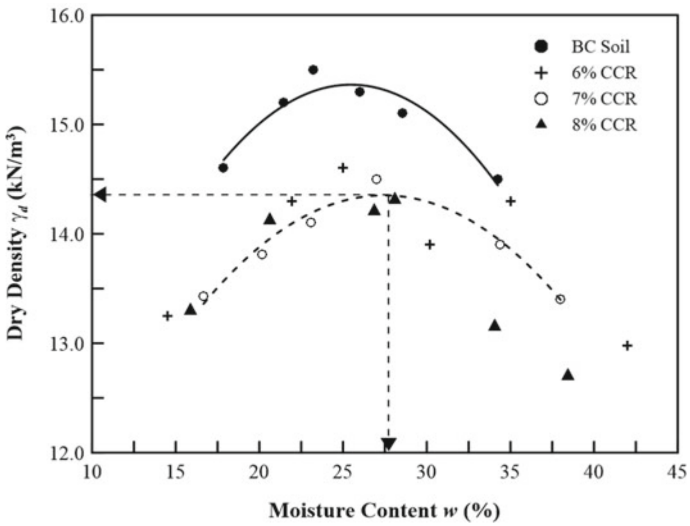


Fig. 5 Relation of OMC and MDD with respect to the addition of different % of CCR

Specimens were tested for the unconfined compression test, and the failure pattern of the sample was studied as shown in Fig. 7. Test carried out on the pure black cotton soil at the 0 curing days is represented by the continuous line in Fig. 7a–c, and it gave the maximum strength of 369.58 kPa. Further, samples prepared with the addition of CCR and tested at the age of 7, 14 and 28 days indicated the increasing trend in the strength. Strength increment noted for the 7% addition of CCR, for 7, 14 and 28, was 1350.36, 1385.57 and 1660.95 kPa. Eight percentage of addition of CCR gives lower initial strength when the samples were cured for 7 days, but it gives higher ultimate strength with the further curing as indicated in Fig. 6. This increase in strength was due to the increase in the cementitious product in soil despite its lower dry unit weight [6, 21]. The effect of the addition of CCR on strength was observed to be maximum between 4 and 6% as shown in Fig. 6. Adding CCR below 4% and above 6%, the effect in strength gain is insignificant (Fig. 7).

4.4 Swelling Pressure of Black Cotton Soil

Black cotton soil undergoes a high degree of volumetric change due to the presence of the montmorillonite clay mineral during wetting and drying process [10]. The volumetric change reflects the swelling in the soil, and such soils are also referred to as expansive soils. Light structures constructed on such soils suffer more damage due to swelling [10]. Hence, the phenomenon of swelling is a key factor to be studied.

In the present study, swelling potential of the soil was determined from the direct method. Indirect method swell force is indicated directly, and its test setup is shown

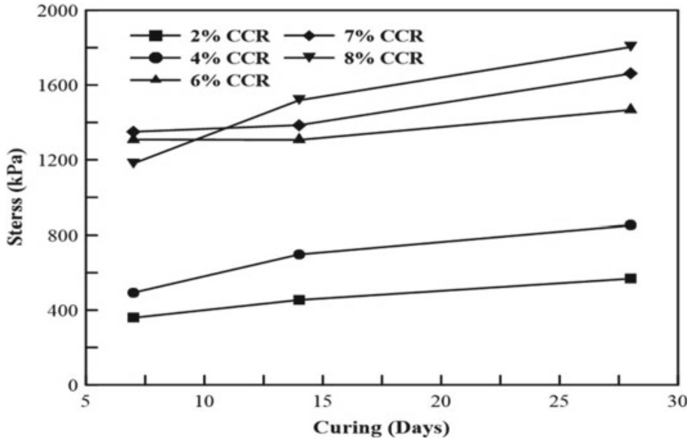


Fig. 6 Effect of addition of CCR on UCS after curing

in Fig. 8. This method is also known as constant volume swell test [10]. The testing assembly includes a mainframe, adapter, load cell, loading pad and water bath for placing the oedometer mold with a soil sample. Porous stones were placed on the top and bottom side of the soil sample for a continuous supply of moisture. A seating load of 10 N was transferred from the loading cap with steel ball arrangement for maintaining a uniform contact between the loading cap and porous stone. Purpose of the loading cap is the uniform transfer of the load over the surface area. Water was filled in the water bath to initiate the test, and water level was maintained constant throughout the test. At a suitable interval of time, swelling force reading was noted. The test was terminated after swell force reading becomes constant. After deducting the seating load from the constant swelling force, swell pressure was determined.

Specimen preparation for swell pressure measurement.

In the present study, swelling of the soil was examined by preparing the indistinguishable soil samples from the soil passing through the 425 μm IS sieve. From the MDD and OMC test results, soil, CCR and water required for filling the oedometer mold were calculated. Distilled water was used in the present study for avoiding any further chemical reactions. For achieving the uniform mix, measured quantities of soil and CCR were mixed in the dry state and later step by step water was added. With the help of a spatula, the soil was mixed in the crucible. The soil was compacted in the oedometer mold on the wet side of the OMC (i.e., OMC + 5%) to avoid any possible losses during mold preparation. After uniform mixing, the mixture was sealed for 24 h in a crucible for uniform distribution of moisture [11]. For achieving the compaction characteristics, mixture obtained after 24 h was statically compacted in the mold in two layers [10]. Using wooden mallet and spacer block, each layer was compacted 25 times in the 75-mm-diameter mold. Before placing the sample in the swelling pressure meter, the upper layer of the compacted soil sample was leveled with a steel spatula for providing a proper base to the porous stone. Test was

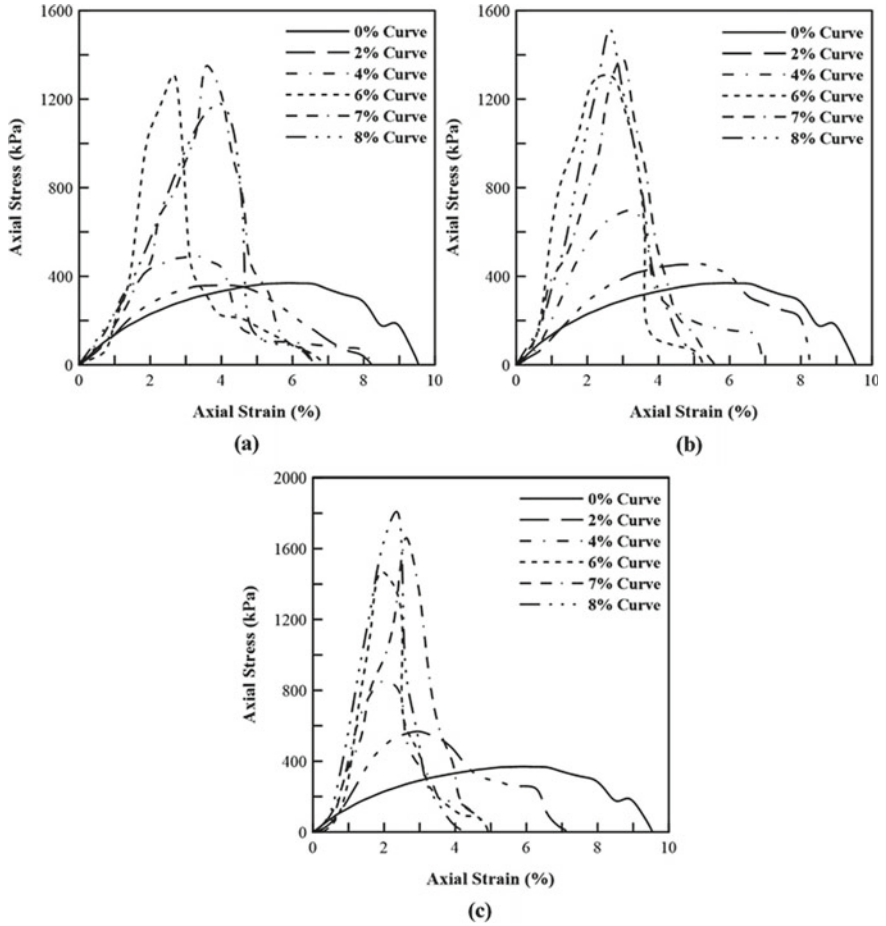


Fig. 7 A stress–strain pattern of the UCS samples during testing, cured for the period of **a** 7 days, **b** 14 days and **c** 28 days

conducted on the the Black Cotton soil and with addition of 2, 4, 6, 7, and 8% CCR. For each mix proportion, three samples were prepared and their average indicates one data point in the graphical representation.

Results of swell pressure.

The swell pressure values of CCR stabilized soil with 7% CCR (optimum dosage) and with variation in its content can be observed in Fig. 9. Swell pressure for pure black cotton soil was observed to be 82.2 kPa, whereas for CCR stabilized soil at 7% CCR was 9.7 kPa. Thus, it can be said that stabilization of soil with 7% CCR reduces its swell pressure by 88% of its initial value. Hence, CCR stabilized soil reduces the uplift pressure exerted by the soil on the loading cap. Mixing CCR with soil significantly reduces the swelling potential of soil, and this reduction in swell is due

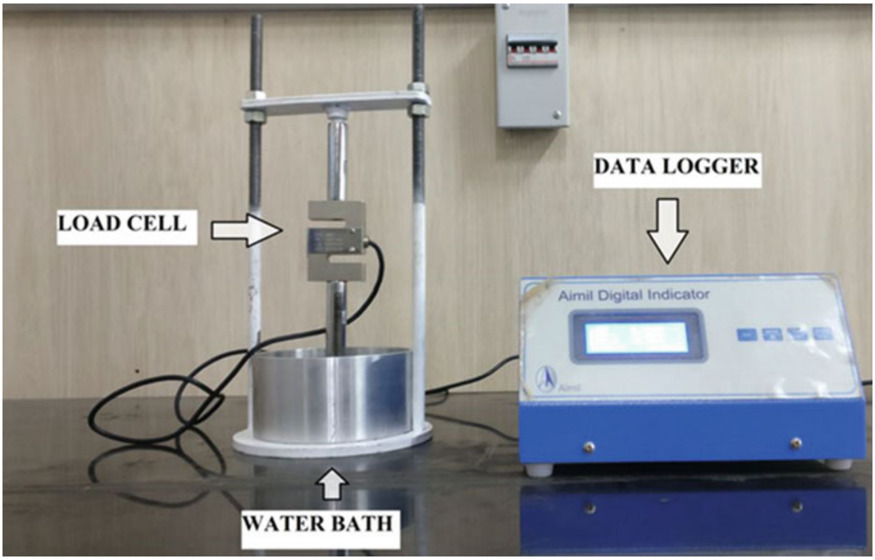
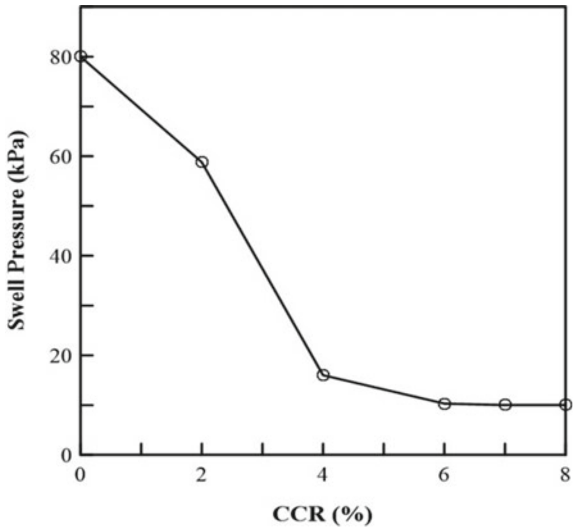


Fig. 8 Swell pressure measuring apparatus

Fig. 9 Variation in swelling pressure with the addition of CCR



to the bonding of the clay particles due to cementitious material [5]. With the increase in CCR content, reduction in swelling pressure was noted as indicated in Fig. 8. Addition of CCR beyond fixation point reduces the vertical swell unsubstantially. But, the effect is more substantial when the addition of CCR is below 4%.

5 Conclusion

Engineering properties of soil stabilized with CCR has been studied, a sustainable stabilizing agent which is a by-product of acetylene gas manufacturing plants has been studied. The index and engineering properties refer to pH, Atterberg's limit, compaction curves, UCS tests and swelling pressure characteristics. The following conclusions can be drawn.

1. Calcium carbide residue an industrial by-product is found to be a suitable soil stabilizing agent because of its high $Ca(OH)_2$ content of about 76%.
2. On the basis of pH test and plasticity index, the optimum dosage of CCR is found to be 7% by weight for black cotton soil stabilized with CCR for Nashik region.
3. CCR is highly alkaline (pH ranging around 12.5), and the pH value of stabilized soil increases with increase in CCR content.
4. As the CCR content in the soil increases, this decrease in plasticity index occurs due to the flocculation of clay particles, due to adsorption of Ca^{+2} ions from the cation exchange process. However, when the CCR content is greater than 7%, the change in the plasticity index is minimal; i.e., there is minimal absorption of Ca^{+2} ions.
5. While conducting the Proctor test, it was observed that the OMC tends to increase with an increase in CCR content whereas the MDD decreases with increase in CCR content. The reduction in MDD is due to the lower specific gravity of CCR and an immediate formation of cementitious products.
6. Unconfined compression test with varying CCR content and for different curing days, it was observed that UCS of the sample increases with increase in CCR content. Moreover, UCS also increases with increase in the duration of curing. For 7% CCR and 28 days curing, the strength observed is 4.49 times the unconfined compressive strength of black cotton soil.
7. Swelling behavior of the black cotton soil reduces considerably with the increase in CCR content. With the addition of 7% CCR, 87.5% reduction in swell pressure was observed when compared with the swell pressure in black cotton soil.

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Influence of Curing Stress and Curing Time on Unconfined Compressive Strength Behaviour of Cemented Clay

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Abstract In recent ground improvement techniques to strengthen the weak soils in order to increase the strength characteristics and stiffness of different types of soils, cement stabilisation has shown to be very effective. In cement stabilisation, curing stress and curing time are two important factors. Since previous researchers have mainly focused on importance of the curing time, the present study emphasises on the combined effect of curing stress and curing time. The aim of present study is to examine the effect of curing time on unconfined compressive strength (UCS) of virgin clayey soil treated with cement under predefined curing stress of 5 kPa. Laboratory test including Atterberg limits, light weight compaction test and UCS test were conducted. The UCS tests were performed on clayey soil (CH) with the different percentages of cement (i.e. 8, 10 and 12%) for the curing period of 0, 3, 7, 14, 28 and 56 days. The curing stress of 5 kPa is being applied along with curing time to check the combined effect of both curing time and curing stress. The results show that the curing time and the curing stress have a significant effect on UCS. The UCS value increases with increasing cement content from 8 to 10% but decreases for further increment in cement content (i.e. from 10 to 12%). The 10% cement content is found to be the optimum. UCS values increases with curing stress and having around 10–15% high value compared to the curing time condition. The increase in UCS is attributed due to the reduced pore spaces and increased confinement due to the curing stress. This study concluded that applying curing stress leads to have incremental impact on the strength of soil and being very helpful for attaining higher strengths in early days.

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Keywords Curing time · Curing stress · Cement stabilisation · UCS

1 Introduction

Soils having low strength is one of the major problems faced by a geotechnical engineer. Different stabilisation techniques are available to overcome this problems. Addition of cement to the soil mixture (CSM) is one of technique to increase the strength of the soil, and it is widely used as a ground improvement technique. CSM has reduced permeability, low compressibility and high strength as compared to the untreated soil [1–3]; It is necessary to identify the behaviour of CSM so as to use in different environmental conditions and according to the requirements. CSM is widely used in various projects throughout the globe to improve the property of the untreated soil. It was first used in Japan (1970) by port and harbour research institute later on it was used to strengthen the embankment of San Francisco's largest potable water reservoir, to stabilise the contaminated sediments in Newark Bay, and to reinforce a slope to maintain its integrity during seismic activities. Typical applications of Cement soil mixture include liquefaction mitigation and foundation stabilisation, vibration reduction and excavation support walls. Constructions for high-speed railway tracks and wind turbines engaged the use of CSM to improve soil [4–6]. Sometimes, we come across such soils at project site which cannot be used as a construction material due to their low bearing capacity and shear strength, e.g. black cotton soil. In that case, we have to improve the soil quality for its further use. This problem leads to development of various ground improvement techniques either using a chemical stabilisation or a mechanical stabilisation [7–9]. Strengthening of weak soil using chemical additives such as cement, lime, bagasse ash, ground granulated blast furnace slag, fly ash has been suggested by many researchers in past [10, 11].

In present study, only one parameter is used to calculate the mechanical properties of the cement-soil mixture, i.e. UCS with considering effect of curing time and a combination of both curing time and curing stress. Soil sample is being collected from locally available site, and a series of laboratory investigation has been conducted to evaluate the performance of CSM with and without curing stress. A series of unconfined compressive strength (UCS), Atterberg limits, light weight compaction test are conducted to analyse the performance of the mixture.

Portland pozzolonic cement is used with varying percentage of 8, 10 and 12% is used in the investigation with special attention given to curing stress and curing time.

A curing stress is being applied along with curing days to check dual effect of both curing time and curing stress. For applying curing stress, a special mould is designed to hold the sample of UCS and curing stress is applied above the sample the design standards of the mould is discussed in this study.

2 Materials Used

2.1 Soil

The soil sample is collected from locally available site of Jaypee university solan, H.P. The physical properties of the soil are discussed in Table 1 and chemical composition of PPC used in the study is given in Table 2.

Table 1 Physical properties of soil used in the study

Property	Value
Specific gravity	2.62
Grain size analysis	
Gravel (%)	0
Sand (%)	15
Silt (%)	20
Clay (%)	65
Consistency limits	
Liquid limit (%)	66
Plastic limit (%)	29
Plasticity index (%)	37
IS classification	CH
Compaction parameters	
OMC of untreated soil (%)	29.42
MDD of untreated soil (g/cc)	1.51
OMC of soil with 8% cement (%)	30.91
OMC of soil with 10% cement (%)	34.06
OMC of soil with 12% cement (%)	34.74
MDD of soil with 8% cement (g/cc)	1.49
MDD of soil with 10% cement (g/cc)	1.47
MDD of soil with 12% cement (g/cc)	1.45
Curing stress (kPa)	5

Table 2 Chemical composition of cement

Oxide	%
CaO	60.99
SiO_2	20.88
Al_2O_3	5.56
Fe_2O_3	3.36
MgO	3.75
SO_3	2.69

3 Experimental Program

To study the improved soil behaviour, the soils was treated with Portland cement. According Woodward (2005), the cement content used for cement-soil mixture in utility is found around 10% of dry soil weight. In present study, it was decided to use (8, 10 and 12%) cement content. The water content that is being used for these 3 different cement content is being calculated by the standard proctor test. The purpose of using different cement content is to find the optimum cement content. To check the effect of curing time 0, 3, 7, 14, 28 and 56 days of curing without applying any curing stress and later with curing stress along with the curing period to analyse their combined effect (Fig. 1).

All the specimens for the UCS test data is being recorded using a strain rate equal to 1% (equalling 1.25 mm/min), a data acquisition arrangement was made to record the applied load and measured deflection. The test proceeded until failure occurred. The samples were tested according to their designated curing times 0, 3, 7, 14, 28 and 56 days.

A curing stress of 5 kPa is being applied along with curing days to check the dual effect of both curing time and curing stress, and test is conducted for 7, 14, 28 and 56 days.

Fig. 1 Unconfined compressive test apparatus

3.1 Mould Preparation for Curing Stress

The apparatus for the curing stress is made with polyvinyl chloride (PVC) pipes which have 55 mm inside diameter and 215 mm height. A base plate of 85 mm is taken which consists of an opening for proper holding of the UCS specimen of 50 mm diameter so that during curing stress it does not move from its place. PVC pipe of 55 mm diameter is fixed from bottom with the base plate and sealed with M-seal to avoid leakage of water and maintain proper curing conditions. Base plate and PVC pipe is shown in Fig. 2. The sample is placed in the base plate mould and covered with PVC pipe.

Fill the pipe with water up to specimen's height. A disc of diameter less than PVC pipe is taken and is kept on specimen so that stress can be applied above it. A curing stress of 5 kPa is applied with the help of measuring weights. After the completion of curing time, the samples were taken out from designed apparatus and were subjected to unconfined compressive tests. The 5 kPa stress is applied with the help of weight that is found out to be around 1 kg.

Calculation of load for the curing stress

Fig. 2 Mould and sample preparation for curing stress

$$\text{STRESS} = \text{FORCE/AREA}$$
$$5\,\text{kPA} = \text{FORCE APPLIED/AREA OF THE SPECIMEN}$$
$$\text{AREA OF SAMPLE OF 50 mm DIAMETER SAMPLE} = 1.96 * 10^{-3}\,\text{m}^2$$
$$\text{FORCE} = \text{STRESS} * \text{AREA}$$
$$\text{FORCE} = 5 * 10^3 * 1.96 * 10^{-3} = 9.81\,\text{N}$$
$$\text{WEIGHT} = \text{MASS} * G$$
$$\text{MASS} = \text{WEIGHT}/G$$
$$\text{MASS} = 9.81/9.81 = 1\,\text{kg}$$

4 Results and Discussions

As explained in previous sections, a number of UCS test with varying percentage of cement content is done with and without curing stress to analyse the impact of curing stress. The results are discussed in sub sections.

4.1 Effect of Cement Content on OMC and MDD of the Soil

The standard proctor test is conducted to find out the optimum moisture content (OMC) and maximum dry density (MDD) of the virgin soil and soil with varying percentage of the cement content. OMC of natural soil is found to be 29.42% and its value increases with cement content due to increase in the demand of water for hydration of the cement [12–14]. Figure 3 shows that with the increase in cement content the OMC of the sample increases and the MDD of the soil sample decreases. The MDD of the untreated soil is 1.51 gm/cm^3. The OMC and MDD corresponding to 8%, 10% and 12% cement content are 30.91%, 33.06% and 34.74% and 1.49 gm/cm^3, 1.47 gm/cm^3, 1.45 gm/cm^3, respectively.

4.2 Stress–Strain Curves

The stress–strain curves are plotted for untreated soil and soil having 8, 10 and 12% of cement content. The stress values of soil increases with increase in curing days, i.e. high values is achieved for 56 days of curing. Figure 4 shows the stress–strain curves for different percentage of cement content. Figure 4a–c corresponds to 8, 10 and 12% cement content stress–strain curves. The stress values increase with time and sample becomes brittle.

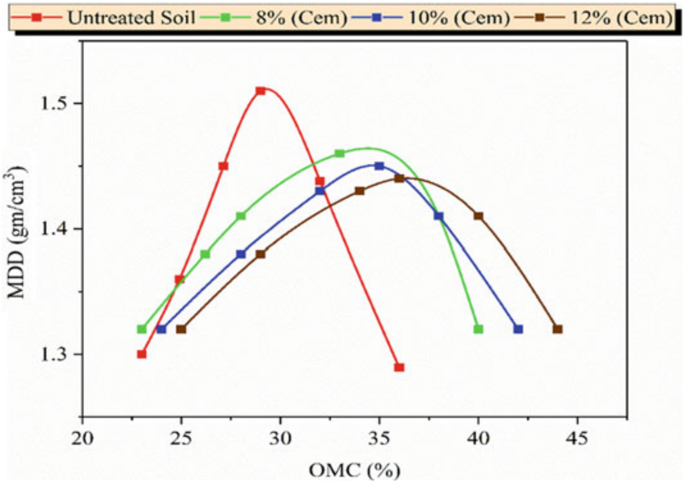

Fig. 3 Variation of OMC and MDD

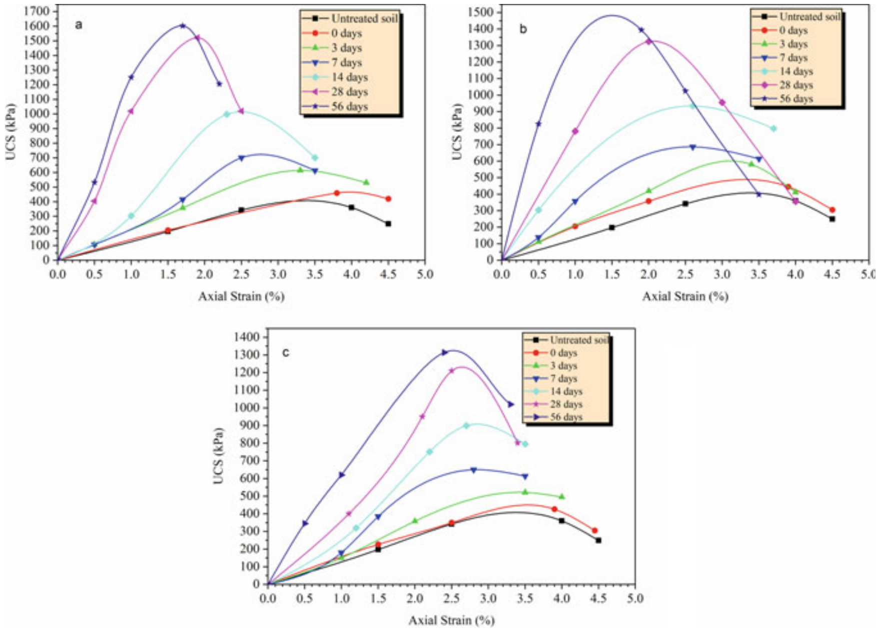

Fig. 4 Stress–strain curves 8%, 10% and 12% cement content for different curing days

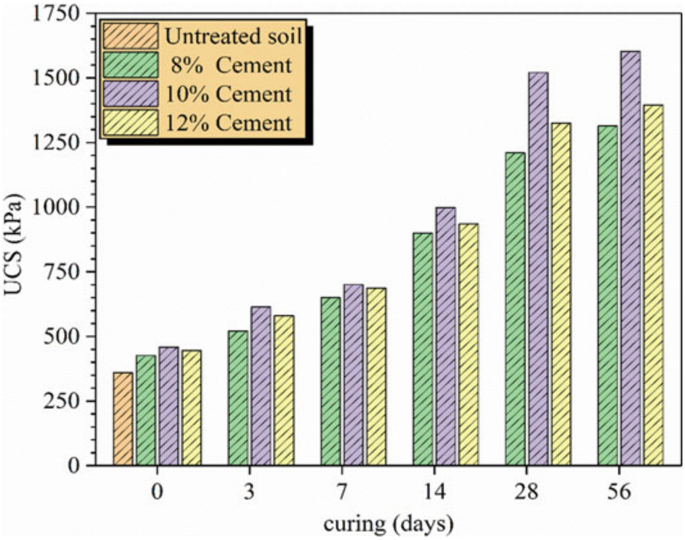

Fig. 5 Variation of UCS with cement content and curing time without curing stress

4.3 *Variation of UCS with Curing Days for Without Curing Stress*

The unconfined compressive strength (UCS) is being conducted on the untreated soil and soil with 8, 10 and 12% cement content. Figure 5 shows the variation of UCS with different cement content and for different curing days. The UCS values increases with curing days and high strength is achieved for 56 days curing. The UCS values increases as cement increase from 8 to 10% and decreases after its increases from 10 to 12%. The UCS strength corresponding to 10% cement is found to be optimum for all curing days. The increase in strength is attributed to the hydration products of cement C-S-H gel formation which reduces pore spaces that decreases the compressibility of the soil and leads to the increase in the strength [1, 15,16]. The high percentage of cement leads to substitution of soil particles by cement which results in lower strength of the soil.

4.4 *Variation of UCS with Curing days and with Curing Stress of 5 kPa*

The unconfined compressive strength UCS is being conducted on the virgin soil and soil with different percentage of cement content. The test is conducted for 7, 14 28 and 56 days of curing only due to the limitations of the mould. 5 kPa curing stress is being applied on the sample. The stress is applied with the help of weights used

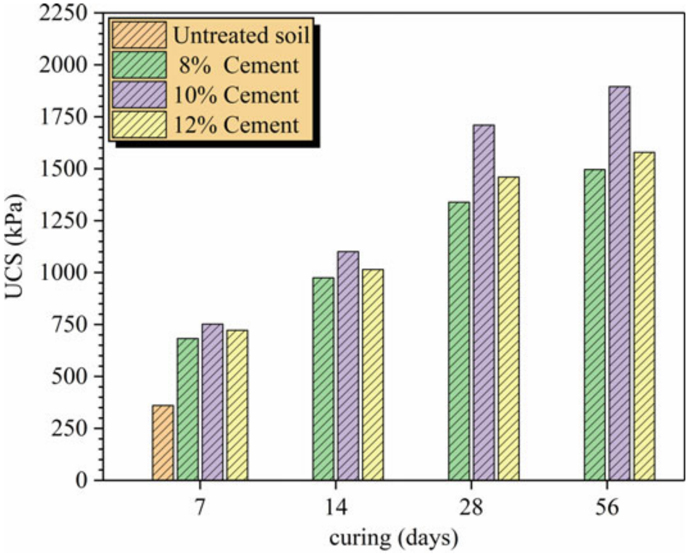

Fig. 6 Variation of UCS with cement content and curing time with curing stress

for standard measurement. Figure 6 shows the variation of UCS with curing days and with curing stress of 5 kPa. The UCS values increases with curing days and higher values of UCS is being achieved with curing stress as compared to the sample that are cured without curing stress. The similar trend is observed here, and the 10% cement has higher strength as compared to 8 and 12%. The increase is strength that is attributed to the increase in confinement pressure from the top that results in decrease in void ratio and porosity of the soil and make the soil more compact.

4.5 Comparison of UCS Values of Soil with and Without Curing Stress

Soil sample is tested for UCS with different percentage of cement content for with and without curing stress conditions. As discussed earlier, higher values of UCS are achieved for 10% cement content as compared to 8 and 12%. The curing stress will increase the UCS values. Figure 7a–d shows the variation of UCS values with curing stress as compared to the without curing stress samples for 7, 14, 28 and 56 days of curing.

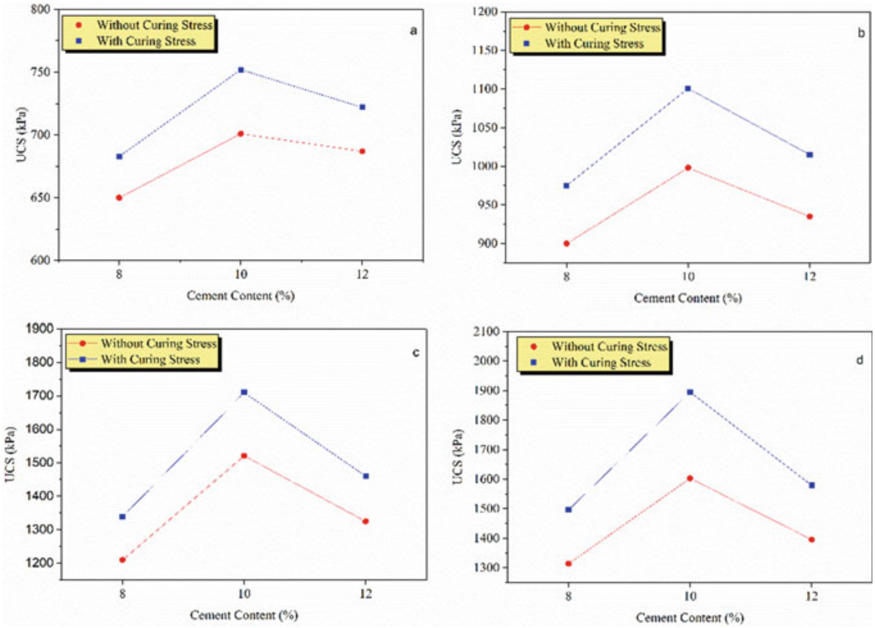

Fig. 7 Variation of UCS for 7, 14, 28 and 56 days of curing for with and without curing stress

5 Conclusions

Based on the laboratory investigations conducted on untreated soil and soil with different percentage of cement content for with and without curing stress, following conclusions can be made.

- Effect of cement content on compaction parameters is not so significant, however, there will be increase in OMC and decreased in MDD of the soil as we increase the cement content.
- The UCS values of soil increases with curing days for same cement content. Higher values of UCS are achieved for 56 days of curing.
- The UCS values increases as we increase the cement content from 8 to 10% but decreases for further increment of the cement content from 10 to 12%.
- The 10% cement content is found to be optimum cement content. The UCS values of the soil increases as we apply the curing stress of 5 kPa, and it is having high values compared to without curing stress samples. The UCS value increases up to 10–15% as compared to without curing stress conditions.
- The curing time and curing stress both have significant effect on the UCS values of the soil.

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Experimental Study on Load-Settlement Behavior of Circular Plate Supported on Small Diameter Timber Piles Under Vertical Loading

Hitesh Kumar and Nayanmoni Chetia

Abstract Locally available wooden piles have been traditionally used in this region for improvement of bearing capacity through decades. Use of reinforced concrete pile is quite effective in improving bearing capacity, but there are sometimes disagreements between efficiency, cost effectiveness, sustainability, etc. Experimental study has been done on model footing (circular plate) supported on small diameter timber piles. Both single and groups of piles of variable length with variable spacing have been tested. Timber piles of 3 (three) different length to diameter ratio with different spacing have been used. The study is conducted on test tank constructed with steel plates and filled with zone-II sand. Load test was carried out on a model circular footing of 0.225 m diameter resting on sand with and without timber piles. The piles were installed in sand bed of 3 (three) different relative densities and subjected to vertical loading. Group efficiencies of pile groups were determined for three length to diameter ratio of 20, 25, and 30 with spacing of 2*D*, 3*D*, 4*D*, and 5*D*. The diameter of piles were taken as 15 mm. Length to diameter ratio, relative density, and spacing between the piles are found to be major factors that influence the performance of the model footing. A regression analysis was done to relate bearing capacity to relative density, spacing of piles, angle of internal friction, unit weight, and length of the pile.

Keywords Timber pile · Pile group efficiency · Relative density · Length to diameter ratio

1 Introduction

Availability of good construction ground are in a declining trend while demand for the same has been increasing exponentially. Foundation on weak soil or on high

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degree variable soil often need proper design, construction, and maintenance. For this, it is required to either modify the structures or to make use of appropriate ground improvement techniques. Objectives of ground improvement techniques are to improve bearing capacity and reduce settlement of loose soil, increase factor of safety against possible slope failure of embankments and dams, reduce susceptibility to liquefaction, reduce shrinkage and swelling of soils, etc. The aims of ground improvement techniques are to increase the bearing capacity of soil, improve the soil properties, and reduce the settlement. Timber piles under consideration are displacement piles and are installed with the driving method or vibratory method. Timber piles are economical, easy to handle, durable, efficient, and safe and can used as a foundation member for most of the structures. Due to installation of timber piles an in-situ reinforced soil system is created which increase the frictional resistance between pile and soil and increases bearing capacity of soil. In 2015, Chetia and Saikia [2] studied improvement of bearing capacity of model footing with structural skirts on soft ground and found that use of structural skirts in conjunction with conventional shallow structural foundations increase in the bearing capacity and reduce settlement. In 2016, Sharma et al. [5] studied the parameters influencing efficiency of micropile groups and found that group efficiency depends upon L/D ratio, spacing of the micropile groups and relative density of sand. In 2016, Ridwan et al. [4] performed experimental Study on bearing capacity of micro grid galam wood pile on soft soil and found that the use of grid modeled wood pile has significant role in increasing bearing capacity of soft soil. It increased approximately 500% of bearing capacity of soft soil. In 2017, Ambak et al. [1] studied the effect of different arrangements such as hexagonal and square arrangement of bamboo piles on rate of settlement on soft kaoline and found that the use of bamboo pile can improve the bearing capacity of soil. In the present study, timber micropiles have been used under model footing plate to study the increase of bearing capacity of the model plate and respective code has been used for determination of bearing capacity.

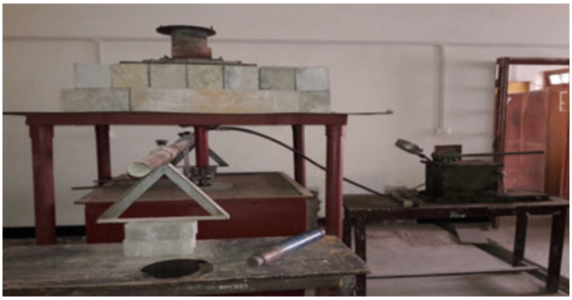
2 Research Methodology

2.1 Test Set-up

To carry out the experimental load tests, a test setup was prepared as per IS: 1888-1982. The test set up consists of a loading frame comprising of the reaction frame which is loaded properly with cement concrete cubes. A mechanically operated inverted hydraulic jack of capacity 20 kN clamped to it, from which load is transferred to the subsoil through the model circular footing. The test tank was constructed with steel plates to prevent them from lateral expansion. Figure 1 shows the prepared test set up for this study.

To measure the magnitude of the applied load, pre-calibrated pressure gauge of capacity 20 kN with 5 kg least count is clamped to a pumping unit. Dial gauges

Fig. 1 Test set up



of accuracy 0.01 mm are placed on the model plate with the help of the horizontal datum bars. These are used to measure the settlements of the model plate due to loading. The datum bars is kept free from any other connection so that any kind of disturbance of the loading frame will not affect the dial gauge system. For this study, cohesionless sand is used. The sand bed is prepared with each layer of 20 cm, and pre-calibrated compacting energy is applied to each layer of sand bed to achieve the required loose, medium, dense relative densities.

2.2 Materials Used

Sand. The sand used for this study is coarse sand collected from Kanaighat (Kalioni River) of Golaghat District, Assam. At first, the sand is air dried and then sieved with IS sieve size of 4.75 mm and portion passing through is taken for the experiment. The grain size distribution curve of the sand is shown in Fig. 2 and the grading characteristics and other properties of sand has been listed in Table 1. The sand was classified as well graded sand based on unified soil classification system (USCS). Direct shear tests are done according to IS: 2720 (Part 13)-1986 to obtain angle of internal friction (φ).

Model footing and model timber piles. Model footing for the tests is made up of steel plate, circular in shape having diameter 0.225 m, and thickness 0.006 m. Model piles are made up of locally available wood, having diameter of 15 mm and *L*/*D* ratio of 20, 25, and 30 with corresponding length of 300, 375, and 450 mm. The photographs of model micro timber piles with roughened surfaces and position of pile in the footing are attached in Fig. 3a, b.

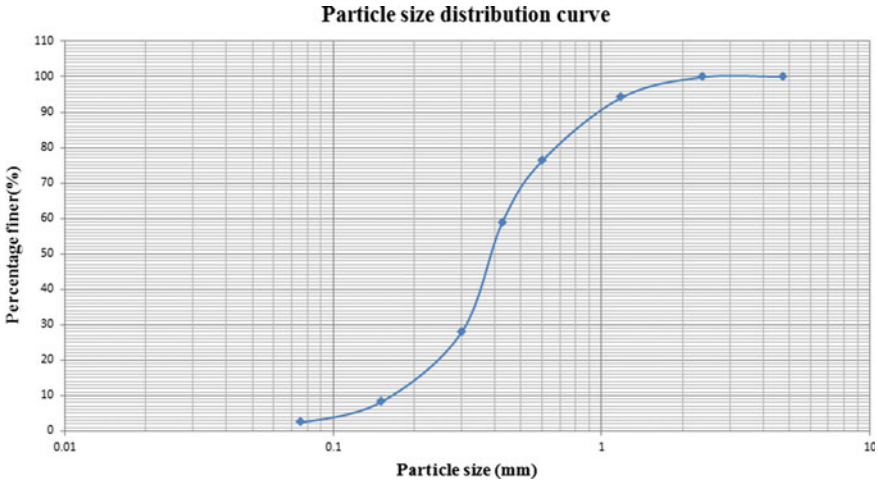


Fig. 2 Particle size distribution curve

Table 1 Properties of sand used in the study

Material	D_{10}	D_{30}	D_{60}	C_u	C_c
Sand	0.17 mm	0.31 mm	0.47 mm	2.76	1.20

3 Methodology for Load Settlement Test

The load-settlement test is performed on the model test tank of dimension 95 cm × 95 cm × 95 cm, and procedure taken for the test is according to plate load test as per IS 1888:1982. After the sand bed obtained the required relative densities, model circular footing is placed centrally, under the hydraulic jack so that the plate, reaction girder and the spindle of the jack are not eccentric. The dial gauges are arranged suitably, and settlement is measured continuously. Load is applied in cumulative equal increments of 10 kg without any impact and eccentricity. Dial gauges readings are recorded for each load increments after the time interval mentioned in IS: 1888-1982. Next higher load is applied to the model circular footing when the settlement gets reduced to 0.02 mm/min, and the process is repeated. The test is continued till a very high settlement value is obtained. For the testing of model timber piles, the piles are inserted centrally into the sand bed and then the model footing plate is placed above it.

(a)

(b)

Fig. 3 **a**, **b** Model micro timber pile and model footing

3.1 Calculation for Ultimate Bearing Capacity of Model Footing

At first, the ultimate bearing capacity of the shallow circular foundation was determined from the load-settlement curves. The load displacement curve of model footing plate in loose, medium, and dense sand with corresponding angle of internal friction as 29°, 33.2°, 35.6° has been presented in Fig. 4. after calculating the experimental value, then ultimate bearing capacities for the same model footings are calculated numerically by bearing capacity equations given by Terzaghi [6] and listed in Table 2. The maximum and minimum dry unit weights are 19.32 and 12.64 kN/m^3. The unit weights of sand beds corresponding to loose, medium, and dense sands are of 14.2, 15.6, and 17.44 kN/m^3 and corresponding relative densities are 31.77, 54.87, and 79.60%. This represents loose, medium, dense conditions.

After comparison, the bearing capacities found from load-settlement graph and Terzaghi's equation shows good proximity hence the test set up can be reasonably used for finding the performance of plates supported by set of timber micropiles.

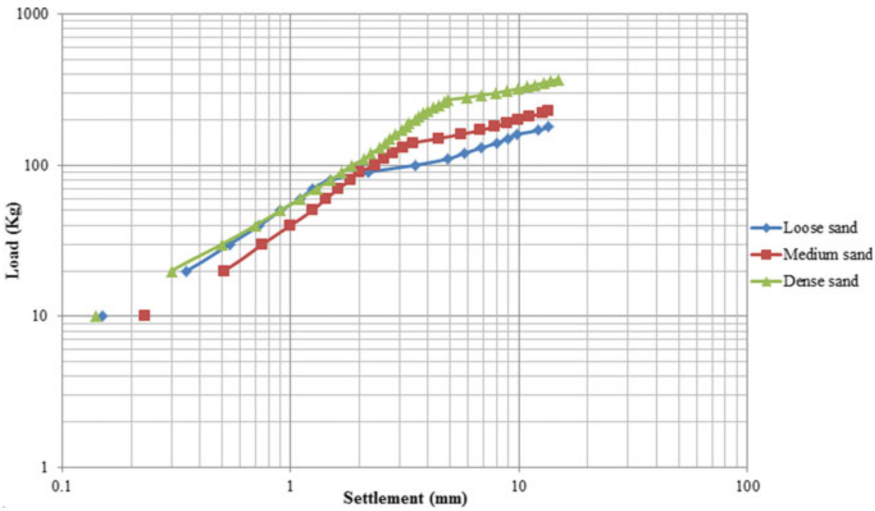


Fig. 4 Load-settlement plot for 0.225 m diameter model footing

Table 2 Comparison of ultimate bearing capacities

Ultimate bearing capacities q_{ult} (kN/m^2)			
	Loose sand	Medium sand	Dense sand
Laboratory experiments	19.72	34.51	66.54
Terzaghi's equation	18.71	32.57	62.93

3.2 Effects of Timber Micropile on Bearing Capacity of Model Footing

The single micro timber pile and pile groups having length to diameter (*L*/*D*) ratio of 20, 25, 30 were inserted in a sand bed of three different relative densities in a square pattern of four piles group, with a spacing of 2*D*, 3*D*, 4*D*, and 5*D* as shown in Fig. 5, where *D* is the diameter of the timber pile. Altogether, 36 timber pile groups and 9 single timber micro piles were taken into consideration while testing for bearing capacity of model footing. The ultimate bearing capacities of plate with single model piles of different *L*/*D* ratios has been listed in Table 3. Load-settlement behavior of circular plate with model micropiles of a particular *L*/*D* ratio with different spacing in loose, medium, and dense sand is presented in Figs. 6, 7, and 8, respectively.

The load-settlement characteristic for the model footing was obtained for different conditions of relative density, length to diameter ratio, and pile spacing. The values are listed in Table 4, and from that values, group efficiency of piles is calculated which is shown in Table 5.

From Table 4, it is seen that in case of loose sand as length of the pile increases bearing capacity also increases, in all case of pile length maximum bearing capacity

(a) (b)

Fig. 5 **a**, **b** Arrangement for inserting timber piles

Table 3 Ultimate bearing capacities of circular plate with single model piles of different *L*/*D* ratios

Pile length (cm)	*L*/*D* ratio	Loose sand	Medium sand	Dense sand
		Ultimate bearing capacity (kN/m^2)	Ultimate bearing capacity (kN/m^2)	Ultimate bearing capacity (kN/m^2)
30	20	29.57	46.83	86.26
37.5	25	32.04	51.76	83.80
45	30	36.97	54.22	78.87

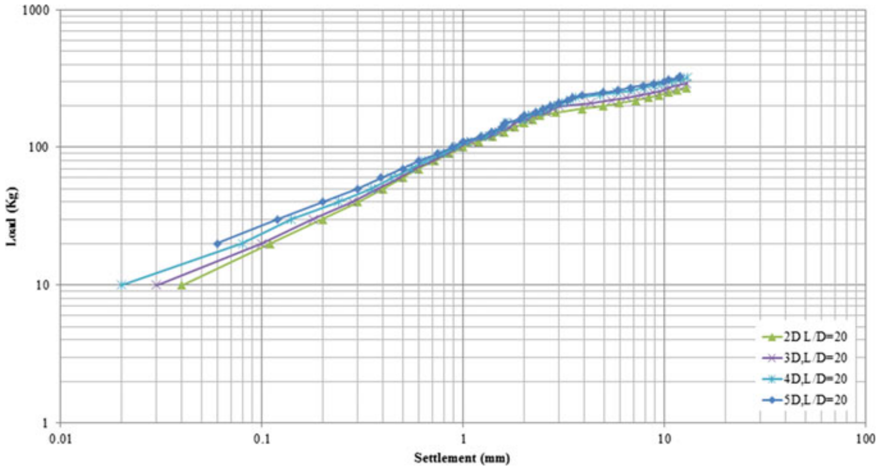


Fig. 6 Load-settlement behavior of circular plate with model piles of *L*/*D* = 20 with variable pile spacing (loose sand)

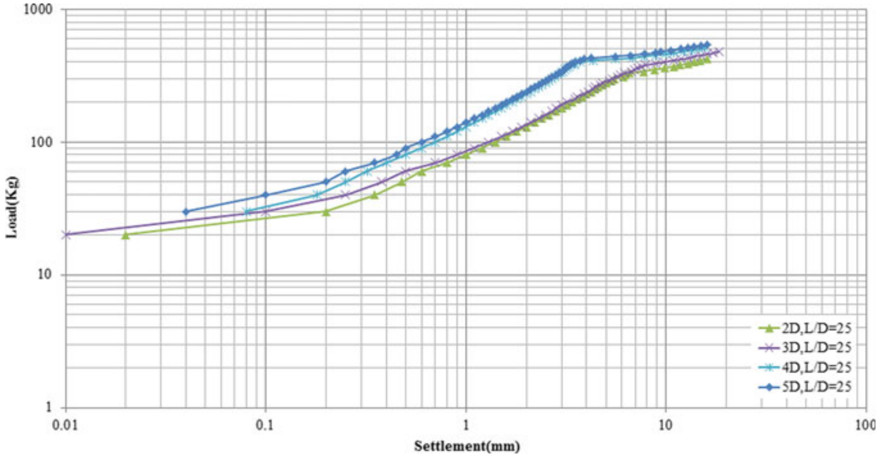


Fig. 7 Load-settlement behavior of circular plate with model piles of $L/D = 25$ with variable pile spacing (medium sand)

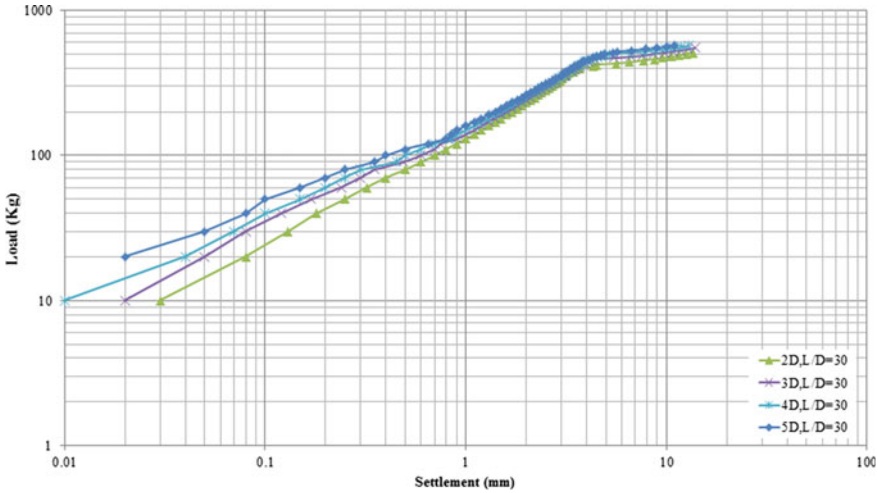


Fig. 8 Load-settlement behavior of circular plate with model piles of $L/D = 30$ with variable pile spacing (dense sand)

observed at pile spacing of 5*D*. For medium sand in case of 30 and 37.5 cm pile maximum bearing capacity observed at pile spacing of 5*D*, in case of 45 cm pile length the maximum bearing capacity observed at pile spacing of 3*D*. For dense sand in case of 30 and 37.5 cm pile, maximum bearing capacity observed at pile spacing of 3*D*, in case of 45 cm pile length, the maximum bearing capacity observed at pile spacing of 5*D*.

Table 4 Ultimate bearing capacities of model footing with model piles of different *L*/*D* ratios, and different spacing's from graph

Pile length (cm)	*L*/*D* ratio	Ultimate load (kN/m^2)			
		2*D*	3*D*	4*D*	5*D*
For loose sand (D_r = 31.77%)					
30	20	44.37	49.29	56.69	59.5
37.5	25	49.29	54.22	61.62	64.08
45	30	59.15	066.54	73.94	76.41
For medium sand (D_r = 54.87%)					
30	20	71.47	81.34	91.19	93.66
37.5	25	81.34	93.66	101.05	105.98
45	30	86.26	115.84	105.98	103.52
For dense sand (D_r = 79.60%)					
30	20	120.77	138.02	133.09	125.70
37.5	25	113.38	128.17	120.77	115.84
45	30	103.52	113.38	123.24	128.16

Table 5 Group efficiency of model micro timber piles

Pile length (cm)	*L*/*D* ratio	Group efficiency			
		2*D*	3*D*	4*D*	5*D*
For loose sand (D_r = 31.77%)					
30	20	0.37	0.41	0.48	0.50
37.5	25	0.38	0.42	0.48	0.50
45	30	0.40	0.45	0.5	0.52
For medium sand (D_r = 54.87%)					
30	20	0.38	0.43	0.48	0.50
37.5	25	0.39	0.45	0.49	0.51
45	30	0.40	0.53	0.49	0.48
For dense sand (D_r = 79.60%)					
30	20	0.35	0.40	0.39	0.36
37.5	25	0.34	0.38	0.36	0.35
45	30	0.33	0.36	0.39	0.41

Variation of efficiency with *L*/*D*ratio. Efficiency versus *L*/*D* ratio were plotted as shown in Fig. 9. It was found that efficiency of the groups increased with the increase in *L*/*D* ratio, in case of loose sand. From the graph, maximum group efficiency obtained for loose, medium, dense sand are 52, 53, 41%.

Variation of efficiency with respect to relative density. The efficiency of the system versus relative density is plotted in Fig. 10. Efficiency was found to increase from

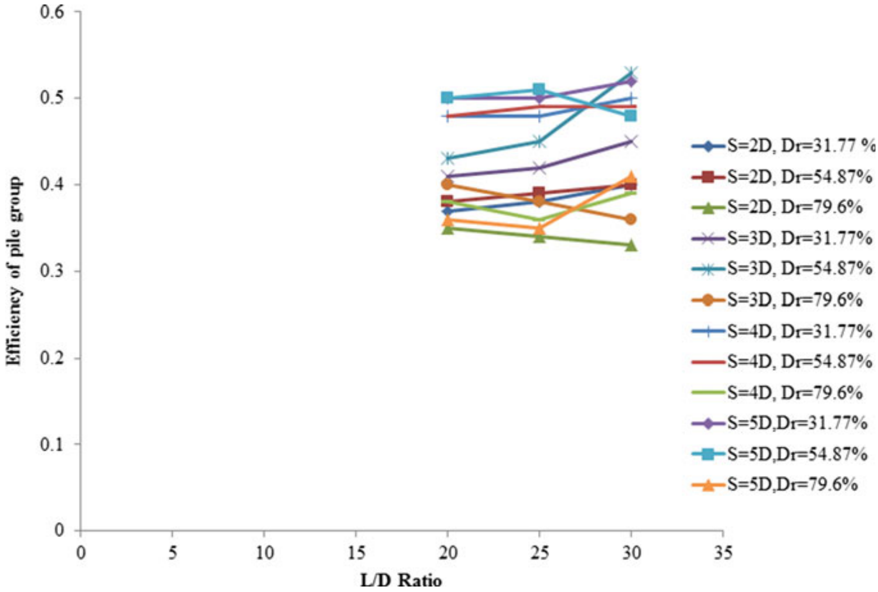


Fig. 9 Efficiency versus *L*/*D* ratio

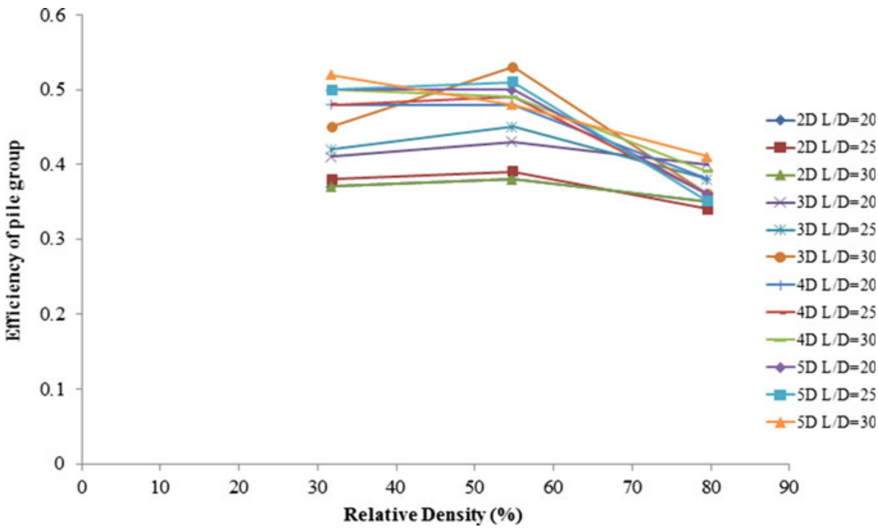


Fig. 10 Efficiency versus relative density plot

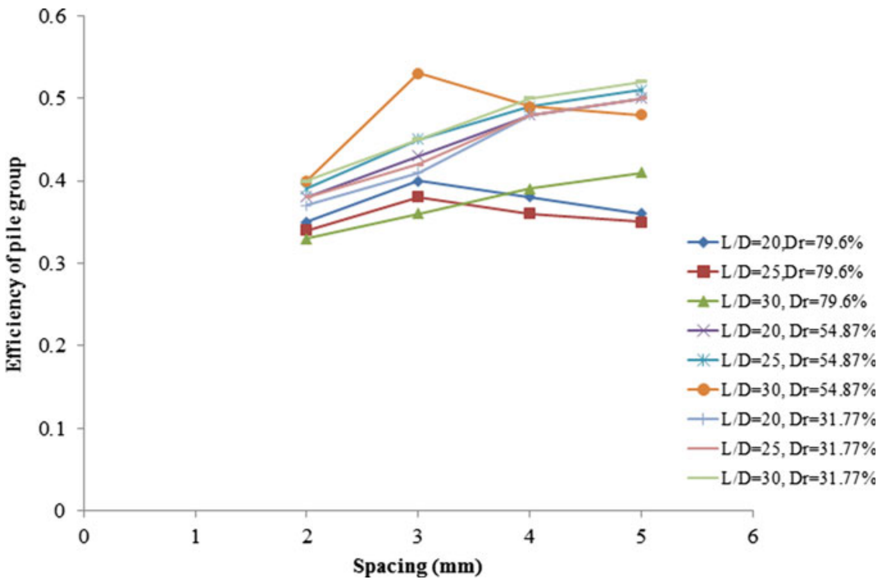


Fig. 11 Efficiency versus spacing plot for footing pile group system

31.77 to 54.87% relative density, and then decrease at 79.60% from the curve, it is observed that medium density is more effective followed by loose and dense.

Variation of efficiency with spacing. Figure 11 shows the plot of efficiency versus spacing of 4-pile groups and model footing system, with spacing of 2*D*, 3*D*, 4*D*, and 5*D*. At low relative density, minimum efficiency is observed at 2*D* spacing and maximum efficiency is observed 5*D* spacing. At medium relative density, minimum efficiency is observed at 2*D* spacing and maximum efficiency is observed at 3*D* spacing of $L/D = 30$ whereas at high relative density, maximum efficiency is observed at 3*D* spacing of $L/D = 20$ and 25 and it gradually reduces at 5*D*, and at $L/D = 30$, maximum efficiency is observed at 5*D* spacing.

4 Regression Analysis

A multiple regression analysis is chosen to develop a model on the basis of the experimental data, wherein the bearing capacity of the circular model footing is taken as the dependent variable, while the rest of the parameters such embedded length of pile (L), spacing of piles (s), relative density of sand (D_r), angle of internal friction of sand (ϕ), unit weight of sand (γ) are considered as independent variables. In this experiment, two SPSS analysis are carried out. For first analysis, a particular length of pile is considered and the number of independent variables are 3. The proposed equations are Eqs. (1), (2), (3), and details are presented in Table 6.

Table 6 SPSS analysis for the model piles of different sand densities

Multiple linear regression			
	$L/D = 20$	$L/D = 25$	$L/D = 30$
Multiple *R*	0.988	0.981	0.963
R square	0.976	0.963	0.928
Adjusted *R* square	0.967	0.949	0.900
Standard error	6.176	6.281	7.290
Observations	12	12	12

The final proposed prediction equation for $L/D = 20$ is found to be

$$q_{\text{predicted}} = -300.865 + 0.031s - 0.957\phi + 25.694\gamma \tag{1}$$

The final proposed prediction equation for $L/D = 25$ is found to be

$$q_{\text{predicted}} = -231.765 + 0.030s + 8.400\phi + 2.104\gamma \tag{2}$$

The final proposed prediction equation for $L/D = 30$ is found to be

$$q_{\text{predicted}} = -163.936 + 0.041s + 9.733\phi - 4.991\gamma \tag{3}$$

SPSS analysis was performed with all four independent variables, viz. (L), (s), (ϕ), (γ) and the generalized formula proposed is presented in Eq. (4). Corresponding analysis is presented Table 7, and normal P-P plot for combined SPSS analysis are shown in Fig. 12.

Table 7 Combined SPSS analysis for the model piles $L/D = 20, 25, 30$

Multiple *R*	0.956
R square	0.914
Adjusted *R* square	0.903
Standard error	8.689
Observations	36

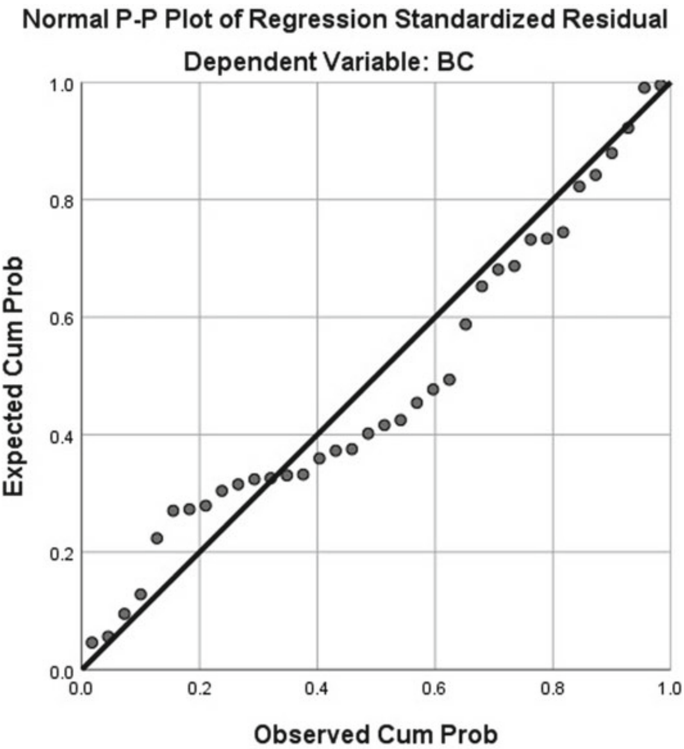


Fig. 12 Normal P-P plot for $L/D = 20, 25, 30$

The final proposed prediction equation is found to be

$$q_{\text{predicted}} = -251.116 + 0.050L + 0.034s + 5.725\phi + 7.602\gamma \tag{4}$$

5 Conclusions

Based on the results of the present experimental investigation, the following conclusion can be drawn

- The bearing capacity of model footing increases with the increase in the length of pile in case of loose sand.
- The maximum group efficiency obtained is 53% for pile with $L/D = 30$ with spacing $3D$ in case of medium density sand.
- The percentage increase in bearing capacity of soil is higher for lower relative density of sand followed by medium and dense.

- Provision of small diameter timber piles for increase of bearing capacity of shallow foundation can be one of the most optimized and sustainable solutions, particularly for loose and medium sand.

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Stabilization of Expansive Soil Using Terrazyme

Abhishek Tiwari, J. K. Sharma, and V. Garg

Abstract The black cotton soil is an expansive soil having a high potential for shrinking and swelling with seasonal moisture variations due to which black cotton soil fails to bear imposed load. Thus, stabilization of black cotton soil is one of the challenging tasks for engineers. The stabilizers used must be economical, easy to implement, and eco-friendly but the methods used conventionally were time-consuming and uneconomical. It is better to use terrazyme to stabilize a soil as it is natural, non-toxic, inflammable, non-corrosive, biodegradable liquid enzyme formulated from fermented vegetable extracts and it shows a permanent effect. This paper deals with a series of tests conducted on virgin soil as well as the tests conducted on the terrazyme treated soil with different dosages such as 0.5/075/1/1.5 cubic meters of soil per 200 ml terrazyme. Adding terrazyme shows significant improvement in engineering properties of soil such as specific gravity, consistency limits, optimum moisture content, maximum dry density, unconfined compressive strength, swelling index, and California Bearing Ratio (CBR) (soaked). On the basis of these results, the optimum dosage is obtained. Apart from this, the soil which we are used must have at least 10% of clay content for the effect of terrazyme.

Keywords Stabilization · Expansive soil · Terrazyme · Biodegradable · Non-toxic

1 Introduction

Black cotton soil is highly expansive in nature and shows high potential for swelling and shrinking with seasonal moisture variation due to which, when the water content

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of the soil increases and decreases during the seasonal variation, soil gets swollen and shrinks, respectively. Particles of soil get rearranged by which cracks in the structure have developed and differential settlement occurs. So the black cotton soil is not able to take an imposed load during or after completion of construction projects without treatment. That is why the treatment of black cotton soil is necessary.

Soil stabilization is an approach to improve the geotechnical properties of soil by using different stabilizers such as traditional (hydrated lime, Portland cement, and fly ash), by-product (kiln dust of lime, cement, etc.), and non-traditional (polymers, enzymes, potassium chloride, etc.). Nowadays, bio-enzyme is a newly emerging technique used as a stabilizer. Bio-enzymes are chemicals which are used as an organic substance and available in liquid concentration. It is effective, easy to use, eco-friendly, and also shows good improvement in geotechnical properties. This study deals with terrazyme as a stabilizer.

1.1 Terrazyme as Stabilizer

Terrazyme is natural, non-toxic, inflammable, non-corrosive, biodegradable liquid enzyme formulated from fermented vegetable and fruits extracts. It improves geotechnical properties of soil such as increase CBR, MDD, and UCS, and decreases the consistency limits, swelling pressure, OMC. Its effect is permanent.

Stabilization mechanism of terrazyme

The mixture of clay and water contains cations (positively charged ions) around a clay particle which creates water film around clay particles and always adsorbed at the surface of clay that adsorbed water called a double layer of clay. When the soil is swelling, the size of the double layer is increased. Terrazyme reacts with a double layer of adsorbed water of clay and reduces its thickness around clay particle due to which reduction in voids of the particle occurs. Ultimately reduction in the swelling capacity of soil (Table 1).

Table 1 Geotechnical properties of terrazyme

Sr. No	Geotechnical properties	Value
1	Physical state	Liquid
2	Color	Dark brown
3	Odor	Smell like molasses
4	Boiling point	212 °F
5	Specific gravity	1–1.09
6	pH value	4.30–4.60
7	Evaporation rate	Same as water
8	Extracted from	Molasses

2 Literature Review

Many researchers worldwide work with different bio-enzyme and terrazyme in different dosages with different type of soil for stabilization of soil and enhancing properties of expansive soil. Shukla et al. [1] presented "bio-enzyme for stabilization of soil in road construction a cost-effective approach," in which they conducted test on five different soils in increasing order of clay content Tests were conducted on untreated and treated soil samples by bio-enzymes to determine different engineering properties, such as consistency limit, UCS, and CBR at different curing period in a laboratory. Low to high improvement is seen in the physical properties of soil with bio-enzyme. Sandy to silty-type soil showed improvement in the UCS and CBR. It was found that pavement thickness is reduced by using of bio-enzyme from 24 to 48%. Ravi Shankar et al. [2] discussed about "bio-enzyme stabilized lateritic soil as a highway material" and they investigated the modified geotechnical properties of the lateritic soils by stabilizing with an enzyme. They found that by adding 200 ml terrazyme per 2, 2.5, 3, 3.5 m^3 of soil, CBR and UCS value increases up to 400% and 450%, respectively. And permeability reduced up to 42%. Agarwal and Kaur [3] studied "effect of bio-enzyme stabilization on unconfined compressive strength of expansive soil" in which they mixed black cotton soil with terrazyme in different dosage (0.0, 0.25 ml, 0.5 ml, 0.75 ml, 1.0 ml, 2.0 ml, 3.0 ml, and 4.0 ml/per 5 kg of soil) and cured for 1 and 7 days. They conducted unconfined compressive strength resulted in a significant increase in the unconfined compressive strength of the black cotton soil up to 200%. Saini and Vaishnava [4] presented "soil stabilization by using terrazyme" in which they mixed local soil with terrazyme in different dosage 3.0/2.5/2.0/1.5 m^3 per 200 ml of terrazyme and conducted different tests such as Atterberg's limit, standard proctor test, and California bearing ratio (soaked and un-soaked). With 2-week curing period, result shows improvement. The (0.75 m^3/200 ml) dosage become optimum because consistency limits decreased and CBR is increased after 4-week curing. Venkatasubramanian and Dhinakaran [5] investigated "effect of bio-enzymatic soil stabilization on unconfined compressive strength and California Bearing Ratio" and performed a test on three different soils with different properties. These soils were tested with different dosage of terrazyme and they found that increments in CBR and UCS are 157–673% and 152–200%, respectively. Bergmann [6] studied on "soil stabilizers on universally accessible trails" and concluded from his study on bio-enzyme that for imparting strength to the soil, bio-enzyme requires some clay content. They found that for proper stabilization of soil, minimum 2% clay content is necessary and 10–15% of clay content gives good results. Rajoria and Kaur [7] carried out a theoretical evaluation of enzyme. They found that the total reduction of road construction cost is about 18–26% in road construction under Maharashtra public works departments.

3 Material Used

3.1 Terrazyme

The terrazyme used for this study which is in concentrated liquid form. It is obtained from Avijeet Agencies Pvt. Limited, Anna Nagar Chennai, India.

Terrazyme dosages

The enzyme dosage varies from 200 ml/1.5 m^3 to 200 ml/0.5 m^3 of the soil, and it depends upon soil properties. In this experimental investigation, the enzyme dosage assumed for expansive clayey soil was 200 ml for bulk volume 1.5–0.5 m^3 of soil. Bulk density of BC soil = 1.49 g/cc

Bulk density = Weight/Volume

Weight = Bulk Density × Volume

For Dosage 1

200 ml for 1.5 m^3 of soil = 1.49 × 1.5 × 1000 = 2235 kg of soil

For 1 kg = 0.089 ml of enzyme.

For Dosage 2

200 ml for 1.0 m^3 of soil = 1.49 × 1.0 × 1000 = 1490 kg of soil

For 1 kg = 0.134 ml of enzyme.

For Dosage 3

200 ml for 0.75 m^3 of soil = 1.49 × 0.75 × 1000 = 1117.5 kg of soil

For 1 kg = 0.178 ml of enzyme.

For Dosage 4

200 ml for 0.5 m^3 of soil = 1.49 × 0.5 × 1000 = 745 kg of soil

For 1 kg = 0.268 ml of enzyme.

3.2 Black Cotton Soil

It is collected from Borkheda area near Allen building construction site, 12 km from Rajasthan Technical University, District Kota Rajasthan for this study.

4 Experimental Program and Results According to (IS Codes)

4.1 Objective

To study the behavior of expansive soil using terrazyme stabilization technique.

4.2 Experimental Details, Result, and Discussion

The following geotechnical properties are studies with different dosage of terrazyme given below

- Consistency limits (liquid limit, plastic limit, plasticity index) with 0, 7, 14 days of curing
- Optimum moisture content, maximum dry density, differential free swell, specific gravity
- CBR (soaked) and UCS with 0, 7, 14 days curing (Table 2).

Table 2 Characteristics of black cotton soil

Sr. No	Engineering properties	Value
1	Specific gravity	2.53
2	*Grain size distribution*	
	Gravel (%)	0.37
	Sand (%)	12.04
	Silt (%)	22.19
	Clay (%)	65.4
3	*Consistency limits*	
	Liquid limit (%)	49.38
	Plastic limit (%)	25.69
	Plasticity index (%)	23.68
4	I.S. soil classification	CI
5	*Engineering properties*	
	Max. dry density (g/cc)	1.60
	OMC (%)	22.10
6	CBR (%) (soaked)	1.82
7	Unconfined compressive strength (kN/m^2)	152.80
8	Differential free swell (%)	58

Table 3 Specific gravity of specimen

Particulars	Sp. gravity
BCS	2.53
1.5 m^3/200 ml	2.55
1.0 m^3/200 ml	2.58
0.75 m^3/200 ml	**2.60**
0.5 m^3/200 ml	2.58

Specific Gravity (IS 2720 Part III, 1980)

The specific gravity is determined by pycnometer test in a laboratory. The specific gravity of black cotton soil and mix specimen is calculated and result given in Table 3.

The specific gravity of black cotton soil is 2.53; after adding terrazyme, specific gravity increases to 2.61 at dosage of 0.75 m^3/200 ml.

Consistency Limits IS 2720 (Part 5, 1980 and Part 6, 1972)

Consistency limits are boundary condition at which the soil changes its state from one to another liquid limit, plastic limit, and plasticity index are falls under consistency limits. The value of the liquid limit is 49.38%, the plastic limit is 25.69%, and plasticity index is 23.68%. Table 4 shows the value of consistency limits with terrazyme and curing period of 0, 7, 14 days curing. And graphical results are shown in Figs. 1, 2, and 3.

With increase the amount of terrazyme and curing period, consistency limits are decreased and maximum decrement occurred at 0.75 m^3/200 ml dosage.

Table 4 Consistency limits of specimen

Dosages	Consistency limits	0 days curing	7 days curing	14 days curing
1.5 m^3/200 ml	Liquid limit	47.52	44.41	41.11
	Plastic limit	24.35	24.35	21.32
	Plasticity index	23.16	20.06	20.78
1.0 m^3/200 ml	Liquid limit	44.80	42.4	39.19
	Plastic limit	22.5	21.97	19.40
	Plasticity index	22.3	20.4	19.79
0.75 m^3/200 ml	Liquid limit	39.65	34.21	29.21
	Plastic limit	20.83	19.46	17.14
	Plasticity index	18.82	14.75	12.06
0.5 m^3/200 ml	Liquid limit	39.98	34.53	30.88
	Plastic limit	20.94	20.83	18.45
	Plasticity index	19.04	11.15	12.43

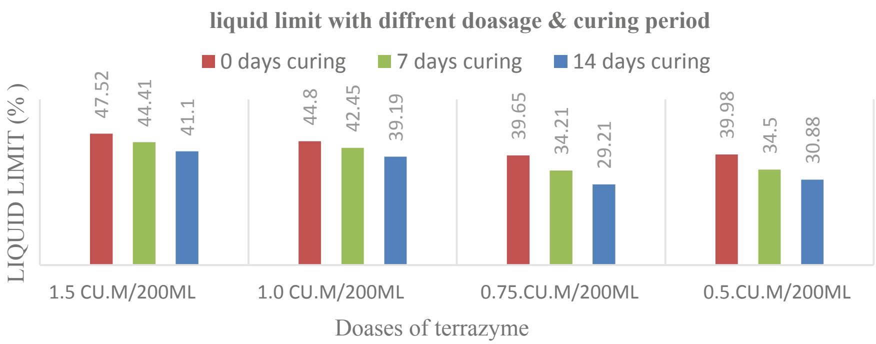

Fig. 1 Liquid limit with diff. dosage and curing

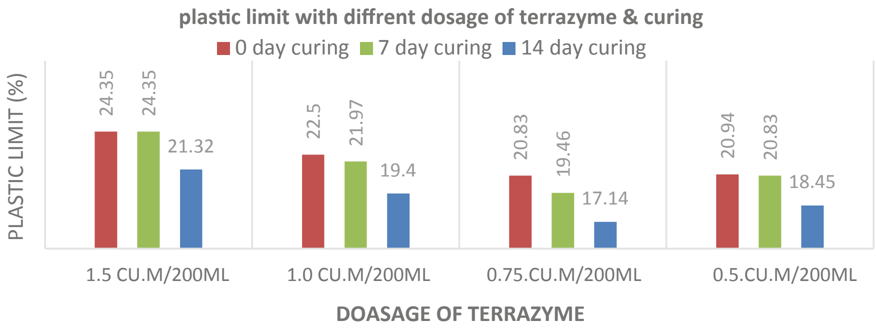

Fig. 2 Plastic limit with different dosage and curing

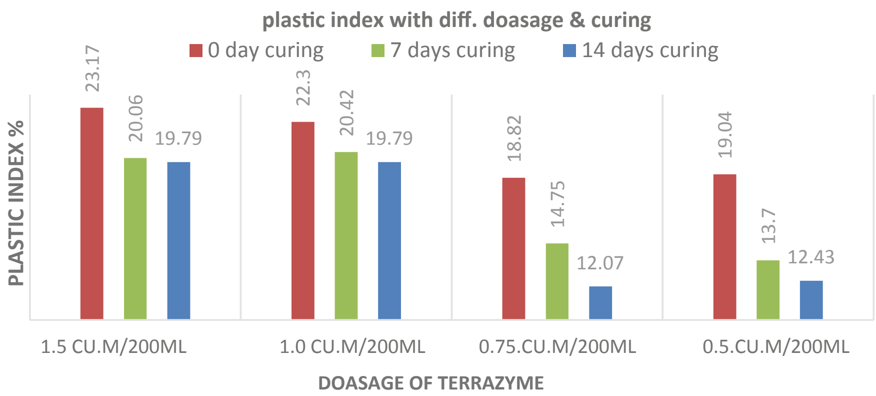

Fig. 3 Plasticity index with different dosage and curing

Table 5 Standard proctor test results for BCS with different dosage of terrazyme

Specimen	MDD (g/cm^3)	OMC (%)
BCS	1.60	22.10
1.5 m^3/200 ml	1.64	21.05
1.0 m^3/200 ml	1.67	19.6
0.75 m^3/200 ml	1.71	17.3
0.5 m^3/200 ml	1.726	17.01

Standard Proctor Test IS 2720 (Part 7, 1980)

For determining the optimum moisture content and maximum dry density, standard proctor test is done with different dosages of terrazyme OMC and MDD is calculated. The OMC and MDD of untreated soil are 22.10% and 1.60 g/cm^3, respectively. The result of the mixed specimen is shown in Table 5.

The maximum dry density (MDD) is increased by 7.8% with terrazyme dosage of 0.5 m^3/200 ml. The graphical representation of the proctor test and mix specimen is shown in Fig. 4.

Unconfined Compressive Strength IS 2720 (Part 10, 1973)

UCS of black cotton soil was evaluated with different dosages of terrazyme with curing period 0, 7, and 14 days of curing. The specimen is prepared and placed into desiccator for retain its moisture. The test result is summarized in Table 6. And graphical representation is shown in Figs. 5, 6, and 7 with 0, 7, and 14 days of curing.

Dosage (0.75 m^3/200 ml) shows the maximum increment of 112% at 14 days of curing.

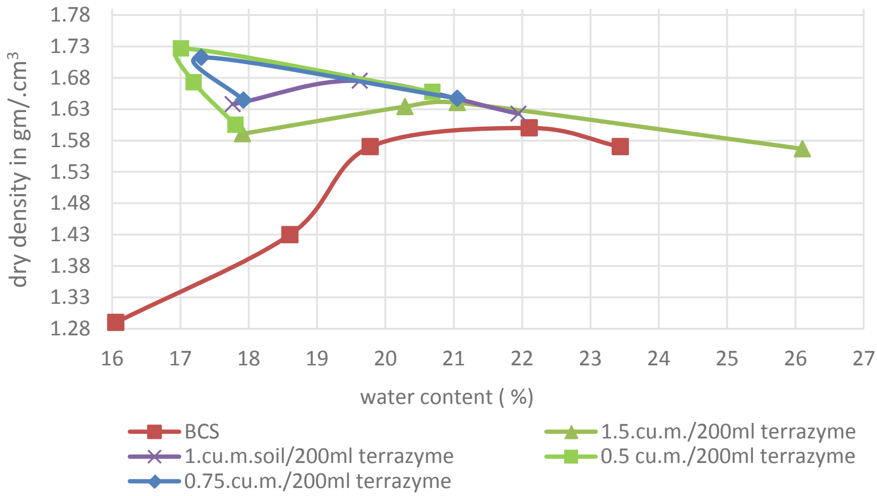

Fig. 4 Standard proctor test for BCS with different dosages of terrazyme

Table 6 UCS test results obtained for mix specimen with 0, 7, and 14 days of curing

Test specimen	Curing period	Unconfined compressive strength, q_u (kN/m^2)	% Variation of compressive strength
BCS	–	152.80	
1.5 m^3/200 ml	0	179.20	17.2
	7	215.66	41.14
	14	259.91	70.1
1.0 m^3/200 ml	0	215.66	41.14
	7	270.96	77.33
	14	299.69	96.13
0.75 m^3/200 ml	0	237.75	55.59
	7	279.81	83.12
	14	324.07	112.08
0.5 m^3/200 ml	0	232.25	51.99
	7	277.60	81.67
	14	310.80	103.40

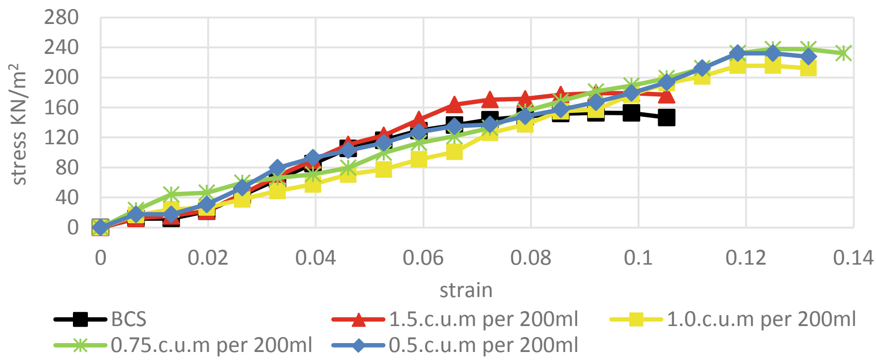

Fig. 5 UCS for mix specimen (0-day curing)

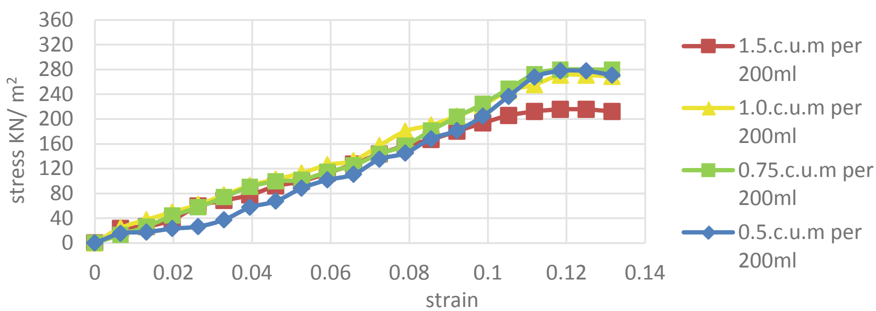

Fig. 6 UCS for mix specimen (7-day curing)

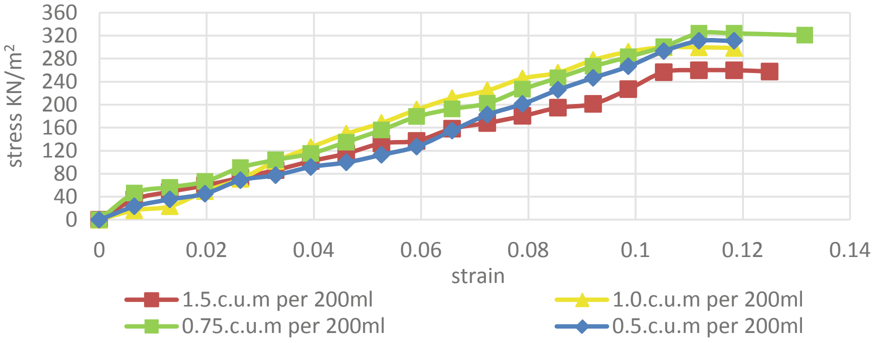

Fig. 7 UCS for mix specimen (14-day curing)

California Bearing Ratio (IS 2720 Part 16, 1979)

California Bearing Ratio Test is conducted to determine the CBR value of soil. Black cotton soil was treated with four dosages of an enzyme (terrazyme) at optimum moisture content (OMC), and molds are prepared by standard proctor method and then covering molds by plastic bags for 0, 7, and 14 days later placed molds for four days in soaked condition. The CBR results after 0, 7, and 14 days of curing are shown in Table 7.

From Table 7, it is observed that the CBR value of black cotton soil is 1.82 in soaked condition. When terrazyme is mixed with a black cotton soil, CBR value of soil is increased up to 17.03% at dosage of 200 ml/0.5 m^3. After curing of 7 and 14 days, CBR value increases 129.67% and 164.83%, respectively, at dosage of 0.75 m^3/200 ml.

Table 7 CBR test results obtained for mix specimen with curing

Test specimen	Curing period	CBR%	% Increase
BCS	–	1.82	–
1.5 m^3/200 ml	0	1.94	6.18
	7	3.19	75.27
	14	3.49	91.75
1.0 m^3/200 ml	0	2.01	10.43
	7	3.64	100
	14	4.02	120.87
0.75 m^3/200 ml	0	2.09	14.83
	7	4.18	129.67
	14	4.82	164.83
0.5 m^3/200 ml	0	2.13	17.03
	7	4.1	125.27
	14	4.44	143.95

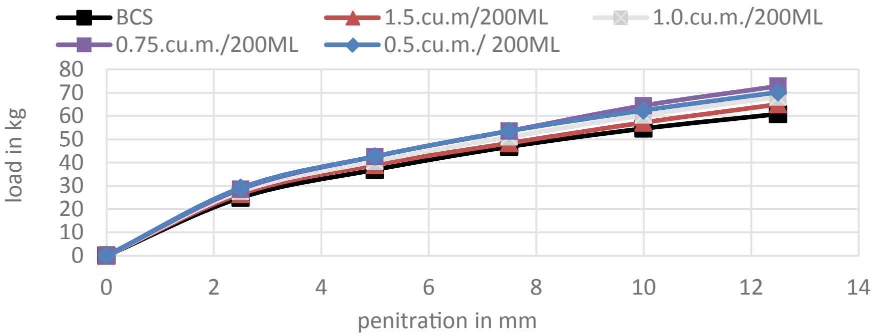

Fig. 8 Soaked CBR with (0-day curing)

And graphical representation is shown in Figs. 8, 9, and 10 with 0, 7, and 14 days of curing. And percentage increments are shown in Fig. 11.

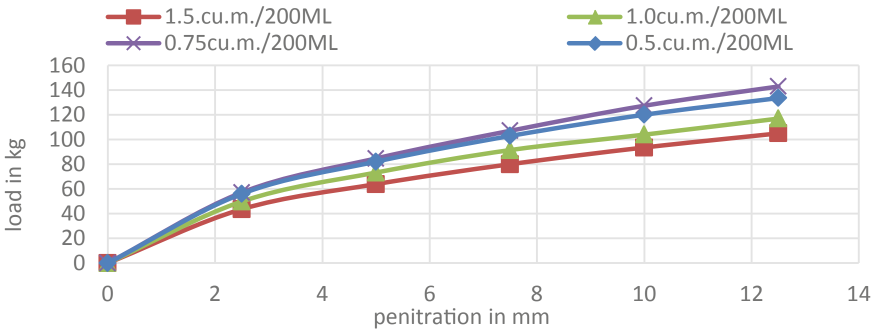

Fig. 9 Soaked CBR with (7-day curing)

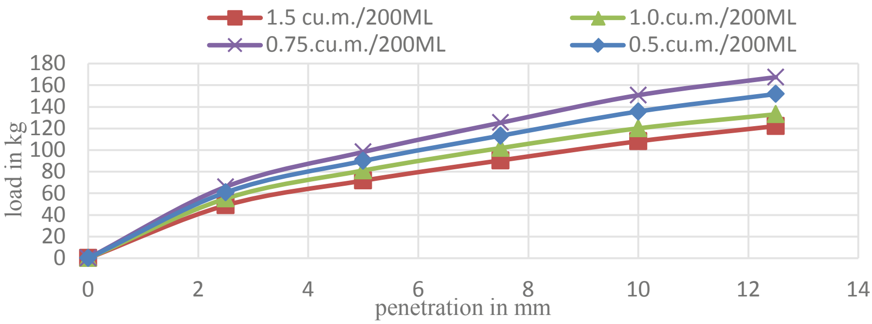

Fig. 10 Soaked CBR with (14-day curing)

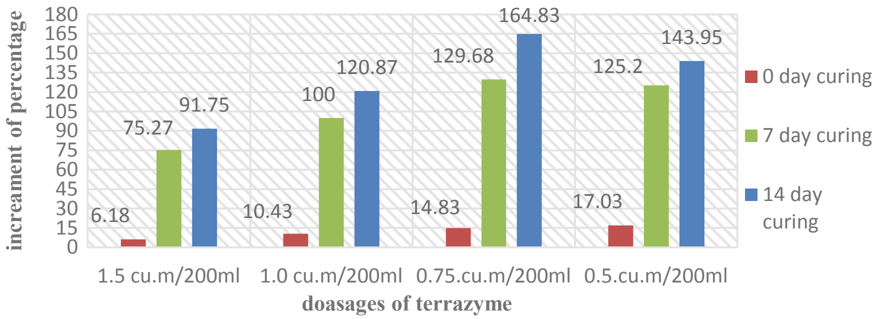

Fig. 11 Percentage variation of CBR value

Table 8 DFS of mix specimen

Particulars	Swell index	Percent decrease
BCS	58	–
1.5 m^3/200 ml	22	62
1.0 m^3/200 ml	11	81
0.75 m^3/200 ml	0	100
0.5 m^3/200 ml	0	100

Differential Free Swell IS 2720 (Part 40, 1977)

The DFS is basically performed to check out the swelling percentage of black cotton soil in case of terrazyme mix specimen DFS is reduced to zero at dosage 0.75 m^3/200 ml (Table 8).

5 Conclusion

On the basis of the above investigations and studies, the conclusion is established.

1. It is observed that the (0.75 m^3/200 ml) is an optimum dosage. Because at this dosage maximum decrement occurred, consistency limits and increment occurred in soaked CBR and UCS.
2. The specific gravity increases with increment in a dosage of terrazyme from 2.53 to 2.60 at dosage of (0.75 m^3/200 ml) and then decreases at dosage of (0.5 m^3/200 ml).
3. The liquid limit of BCS is 49.38% which decreases up to 39.65%, 34.21%, and 29.21% during curing period of 0, 7, and 14 days, respectively, at dosage of (0.75 m^3/200 ml).
4. The plastic limit of BCS is 25.69% which decreases up to 20.83%, 19.46%, and 17.14% during curing period of 0, 7, and 14 days, respectively, at dosage of (0.75 m^3/200 ml).

5. The plasticity index of BCS is 23.68% which decreases up to 18.82%, 14.75%, and 12.06% during curing period of 0, 7, and 14 days, respectively, at dosage of (0.75 m^3/200 ml).
6. The MDD and OMC of black cotton soil are observed 1.60 g/cc and 22.10%, respectively. MDD is increased from 1.60 to 1.72 g/cm^3 and OMC is decreased from 22.10 to 17.10% at dosage of (0.5 m^3/200 ml).
7. The unconfined compressive strength of black cotton soil is 152.8 kN/m^2 which maximum increases up to 55.59%, 83.12%, and 112.08 with a curing period of 0,7, and 14 days, respectively, at dosage of (0.75 m^3/200 ml).
8. The CBR value of black cotton soil is 1.82%. The maximum percentage increment of CBR value with a curing period of 0, 7, and 14 days is 14.83%, 129.67%, and 164.83%, respectively, at dosage of (0.75 m^3/200 ml).
9. The DFS of black cotton soil is 58% and decreases to zero at dosage of (0.75 m^3/200 ml).

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Stabilization of Expansive Black Cotton Soil Using Alkali-Activated Binder with Glass and Polypropylene Fiber

Mazhar Syed, Sagar Agarwal, and Anasua GuhaRay

Abstract Expansive soil exhibits high swelling and shrinkage behavior when moisture fluctuation occurs; this volumetric variation in black cotton soil (BCS) renders unsuitable for use in geotechnical applications. The present paper emphasizes an experimental investigation on the effect of discrete polypropylene (PP) and glass fiber (GF) with alkali-activated binder (AAB) on geomechanical properties of BCS. Hence, the present study aims to compare the geoengineering and microstructural characteristics between PP and glass fiber on AAB treated BCS. PP and GF were varied from 0 to 0.4% with 5% AAB in the BCS. AAB is produced by the reaction between alkali-activator solution (sodium hydroxide and sodium silicate) and aluminosilicate precursor (Class-F fly ash/slag). Microstructural analysis for AAB treated BCS reinforced with both PP and GF is performed through a stereomicroscope, X-ray diffraction (XRD), Fourier transform infrared spectroscopy (FTIR), scanning electron microscope (SEM) and energy dispersive X-ray spectroscopy (EDS). The unconfined compressive strength (UCS), indirect tensile strength (ITS) California Bearing Ratio (CBR) and linear shrinkage tests for glass fiber-AAB and PP fiber-AAB treated BCS are carried out. The influences of varying percentages of different fiber with AAB content in the BCS show a significant improvement in geoengineering properties, especially tensile strength. It is observed that the addition of 5% AAB with 0.3% of glass fiber and PP fiber reduces the linear shrinkage by 12–15% while CBR and ITS values are increased by 20–30%. From the results, it is observed that the PP fiber-AAB treated BCS achieves maximum mechanical strength when compared to glass fiber-AAB treated BCS.

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Keywords Black cotton soil · Alkali-activated binder · Polypropylene and glass fibers · Geoengineering characteristics · Microstructural analysis

1 Introduction

Expansive black cotton soil (BCS) has a low volumetric stability due to a high concentration of montmorillonite and smectite group, which increase the swelling and shrinkage characteristics during wetting and drying season [1, 2]. Chemical soil stabilization by lime and cement is an effective method to upgrade the geomechanical characteristics of these expansive soils by altering physico-mechanical behavior [3, 4]. However, the production of this calcium-based binder has shown a great impact on the environment by releasing carbon dioxide and greenhouse gases [5, 6]. Hence, it is always encouraged to utilize the industrial by-products such as fly ash [7, 8], agro-waste like palm oil fuel ash, rice husk ash, cement kiln dust, solid wastes like Ground Granulated Blast Furnace Slag (GGBS), bagasse and volcanic ash [9, 10], as full or partial replacement with conventional cementitious binders along with inclusion of geotextiles, fibers [11, 12]. Disposal of these industrial by-products is a serious issue. In India, power plant produces nearly 95 million tons of fly ash per annum as per IRC-SP-20-2002 and almost 85% of ash produced is very fine in nature. Therefore, the utilization fly ash/slag with cementitious binder aids to serve the dual benefits of improving the soil bearing capacity and preventing the disposal into dumping yard [8–10]. Multiple researches are carried out to improve the geomechanical behavior in terms of tensile and shear strength of soil with a combination of cementitious binder and fiber [11, 12]. However, limited studies are reported on the comparison of tensile strength using AAB as an additive to soil, reinforced with glass and polypropylene fiber.

The present study aims to utilize the industrial by-product precursors with low carbon emission binder as a cementitious additive for ground improvement. The study proposes a method of geopolymerization of expansive black cotton soil by blending Class-F fly ash with a solution of sodium silicate and sodium hydroxide. This mixture combination leads to the formation of alkali-activated binders (AABs) which possess similar mechanical characteristics as hardened cement binder [13, 14]. As a result, their geomechanical properties are also expected to be similar to Portland cement (PC) binder influence. The primary objective of this paper is to improve and compare the tensile strength and ductility behavior of BCS by adding AAB with discrete glass and polypropylene fibers. A series of geotechnical characterization tests are conducted using different percentages of glass and polypropylene fiber with AAB. In addition, microstructural characterizations are also carried out to understand the interfacial mechanism of fiber reinforced AAB treated soil.

2 Material Characteristics

2.1 *Expansive Black Cotton Soil (BCS)*

BCS used in the present study is collected from Nalgonda district of Telangana state. The soil is dark brown in color and is excavated at 30 cm depth from natural ground surfaces to avoid grabbing of roots. BCS is classified as high plasticity clay (CH) as per Unified Soil Classification System (USCS). The different physical and engineering properties of raw BCS are provided in Table 1. The particle size distribution curves of BCS, along with fly ash, are shown in Fig. 1. The BCS is sun-dried and ground prior to all the tests.

Table 1 Engineering properties of raw BCS

Soil properties	Values
Specific gravity	2.59
Optimum moisture content, OMC (%)	24.5
Maximum dry density, MDD (g/cc))	1.65
Liquid limit, LL (%)	62.0
Plasticity index, PI (%)	38.0
Free swelling index, FSI (%)	86.0
Indirect tensile strength, ITS (kPa)	12.54
Unconfined compressive strength, UCS (kPa)	185
California bearing ratio, CBR (%) soaked	1.96

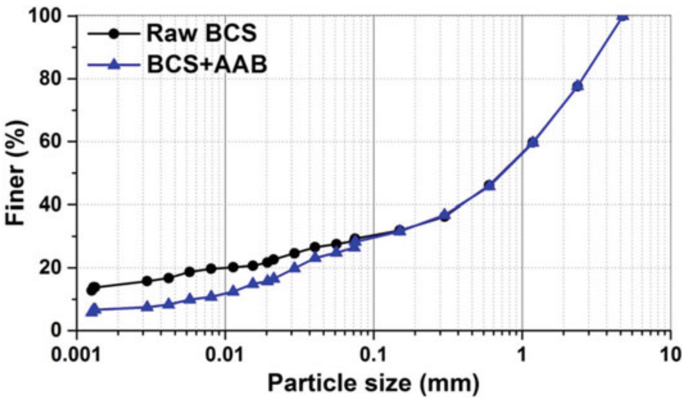

Fig. 1 Particle size distribution curve of raw BCS and AAB treated BCS

Fig. 2 Image showing discrete polypropylene and glass fiber

2.2 Polypropylene (PP) and Glass Fiber (GF)

Polypropylene and glass fiber used in this study are acquired by Kankadurga Industries Pvt. Ltd., Hyderabad. Both fibers length of 12 mm is adopted for all the tests. Figure 2 shows the physical appearance of both polypropylene and glass fiber.

2.3 Alkali-Activated Binder (AAB)

AAB is prepared by mixing sodium silicate solution with crushed sodium hydroxide pellets and water with fly ash, by maintaining the mass ratio of sodium hydroxide to sodium silicate to fly ash as 10.57:129.43:400 [12]. The sodium silicate solution and sodium hydroxide pellets are obtained from HYCHEM Chemical Laboratories. Fly ash is obtained from NTPC thermal power plant, Ramagundam.

2.4 Preparation of Soil Sample

BCS is uniformly mixed with 5% of AAB paste (total mass of soil) by maintaining 0.4 *w*/*s* ratio in the AAB compound. AAB mixed soil is compacted in three layers with a 9 kg steel rammer falling freely from a height of 310 mm. The compacted soil is covered with moist jute bags for 24 h, prior to random mixing of different percentages (0, 0.1, 0.2, 0.3 and 0.4% mass of BCS) of GF and PPF in the AAB soil.

3 Results and Discussion

3.1 Microstructural Characterization

X-ray diffraction (XRD). Powder X-ray diffraction analyses are performed to identify the mineral crystallinity present in untreated BCS, glass and polypropylene fiber reinforced AAB treated BCS using a Rigaku Ultima IV diffractometer with CuKα rays generated at 40 mA and 40 kV by operating 2θ ranges from 00 to 800 with a step 0.020 for 2θ values at 2 s per step. Figure 3 shows the XRD pattern of BCS. Raw BCS reveals the presence of clay minerals such as montmorillonite (M), quartz, (Q) and muscovite (Ms) [12]. Further addition of fly ash-based sodium aluminosilicate binder in BCS with fibers shows the additional peaks corresponding to mullite (Mu) and augite (Au), which are characteristic of hardened AAB paste. Moreover, the XRD pattern of both glass and polypropylene fiber reinforced AAB soil shows the negligible changes between the peak intensities, as is evident from the diffractograms. The formation of new minerals in the BCS may be attributed to the geopolymerization reaction of AAB. Hence, it can be concluded that both the fibers in AAB will remain in conjunction with BCS after forming.

Fourier transfer infrared (FTIR) spectroscopy. Fourier transform infrared (FTIR) spectroscopy is conducted to analyze the molecular bonding vibration present in untreated BCS, glass and polypropylene fiber reinforced AAB treated BCS using a JASCO FTIR 4200 setup through KBr pellet arrangement between 4000 and 500 cm^{-1} infrared spectroscopy. Figure 4 shows the transmittance spectrum curves of untreated and fiber-AAB treated BCS. Raw BCS shows O–H stretching vibrations around 3615 cm^{-1}, which is the general characteristic of montmorillonite. AAB treated BCS shows slight reduction in the intensity of chemical bonds corresponding to montmorillonite. Furthermore, the broadband is found at 3450 cm^{-1} corresponding to O–H stretching of the hydroxyl group. C=O carbonyl bond is also detected at 1730 cm^{-1} for all samples. Another main peak at 1033, 785 and 527 cm^{-1}

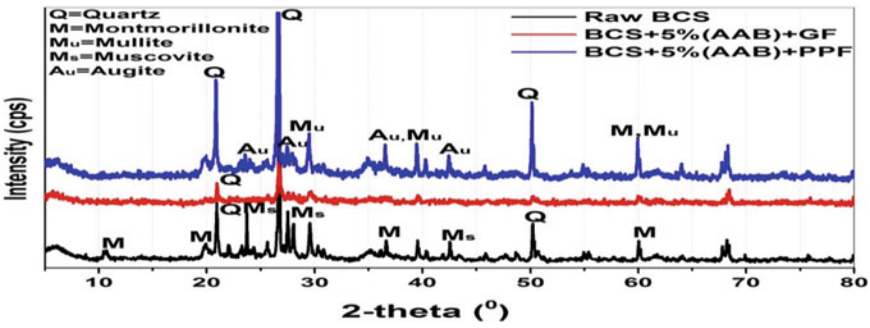

Fig. 3 XRD pattern of untreated BCS and fiber-AAB treated BCS

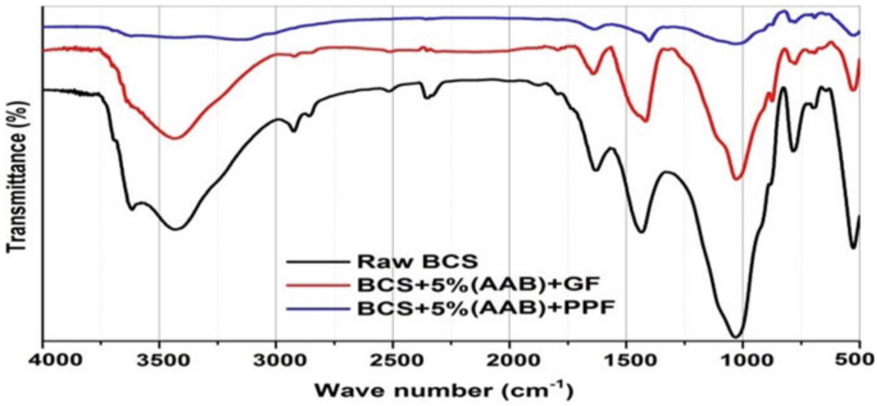

Fig. 4 FTIR spectroscopy of untreated BCS and fiber-AAB treated BCS

is attributed to Si–O–Si, Al–O. Si–O–Al bonds are visible in raw and fiber-AAB BCS samples, but most of them show a chemical shift of about 10 cm^{-1}.

Scanning electron microscope (SEM) and energy dispersive X-ray spectroscopy (EDX). Soil surface micrographs and elemental compositions of untreated BCS, glass and polypropylene fiber reinforced AAB treated BCS are examined using a Thermo Scientific Apreo SEM provided by Field Electron and Ion Company (FEI) at different magnifications and spot regions. Energy dispersive X-ray spectra (EDS) are recorded using Aztec analyzer system provided by Oxford instruments with a probe current of 65 μA at a working distance of 10 mm. Figure 5a shows the morphology of untreated BCS, which reveals the flocculated flaky microstructure. In addition, oxygen (O) and silica (Si) are found to be the major components. Figure 5b, c shows the micrographs of discrete polypropylene and glass fiber matrix incorporated with fly ash-based AAB paste around the aggregated clayey surfaces. This fiber-AAB mixture acts as spatial thread groove network by interlocking the clayey particles through bonding. Moreover, the peak intensities of silica (Si) and calcium (Ca) become relatively stronger in AAB treated BCS.

Stereomicroscopic images. Stereomicroscopic images are used to visualize the surface images of untreated BCS, glass and polypropylene fiber reinforced AAB treated BCS using an Olympus SXZ7 setup with the least dimension of 20 μm. Figure 6a shows the surface images of untreated BCS, which consist of light red colour particles with some brown regions, indicating the presence of iron, illite and smectite group [4, 12]. Figure 6b shows the deposition of hardened AAB paste with polypropylene fiber around the BCS. Figure 6c shows the discrete glass fiber matrix embedded in AAB treated BCS, which act as bonding bridge network between the particles.

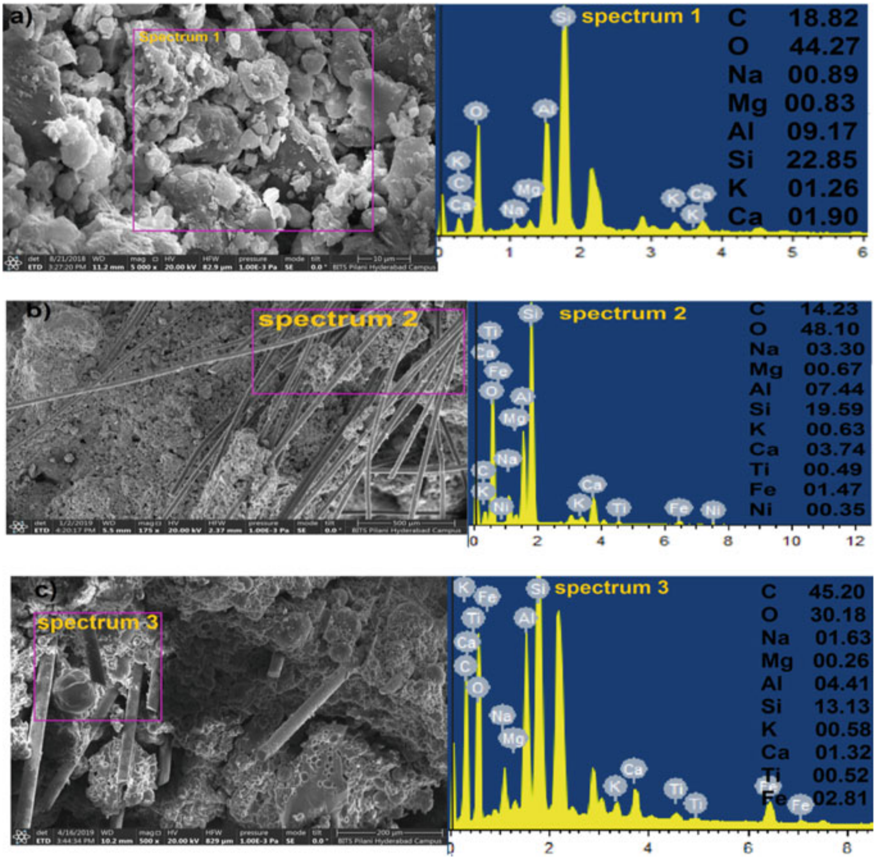

Fig. 5 SEM/EDS images of **a** untreated BCS, **b** polypropylene fiber reinforced AAB treated BCS, **c** glass fiber reinforced AAB treated BCS

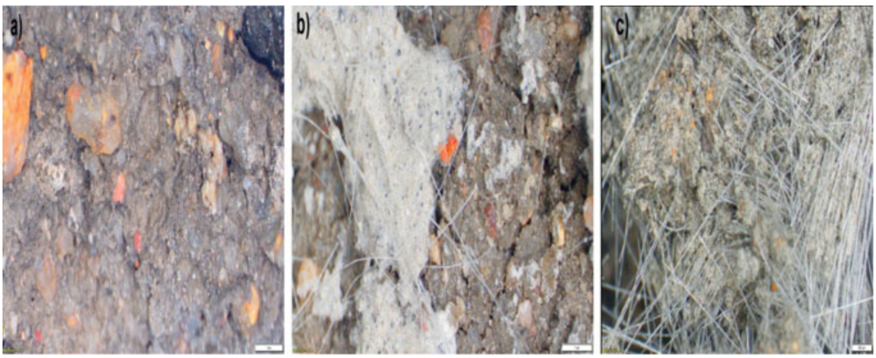

Fig. 6 Stereomicroscopy images of **a** untreated BCS, **b** polypropylene fiber reinforced AAB treated BCS, **c** glass fiber reinforced AAB treated BCS

3.2 Geotechnical Characterization

Detailed geoengineering characterization is performed using compaction, shear, tensile resistance, consolidation and California bearing ratio tests for both untreated BCS and fiber reinforced AAB treated BCS. All the soil specimens are prepared with respect to their MDD and OMC values. These experimental results are used to assess the effectiveness of binder and mechanical behavior of fiber reinforced-AAB-treated BCS. The details of the tests and the discussion of test results are given in the following sections.

Compaction. A series of standard Proctor compaction tests are performed as per ASTM D-698 standard for untreated BCS, glass and polypropylene fiber reinforced AAB treated BCS (Fig. 7). MDD and OMC of raw BCS are found 1.65 g/cc and 24.5%. The intrusion of fly ash-based sodium aluminosilicate binder increases the MDD value from 1.65 to 1.82 g/cc. mainly fiber-AAB attributes toward the filling of voids between the clayey particles and reduction of specific surface area in the soil [3]. It is also interesting to note that the optimum moisture content of AAB modified BCS reduces from 24.5 to 18.6%. This reduction may be because of induced pozzolanic reaction and flocculation mechanism due to encapsulation of clayey particles by deposition of hardened AAB paste with fiber matrix. The increase in MDD and decrease in OMC is an indicator of improving the shear strength properties of soil.

Linear shrinkage. The linear shrinkage tests are carried out as per the Australian AS1289-C4 standard code for both untreated BCS and fiber reinforced AAB treated BCS. Figure 8 shows the variation of shrinkage value with respect to fiber content. The shrinkage limit and linear shrinkage of raw BC are 15.3% and 26.8%, respectively. The addition of AAB in BCS leads to reduce the shrinkage properties. Further addition of fiber in the AAB modified BCS shows a marginal effect on linear shrinkage and controls the cracks effectively. This reduction may be attributed toward the formation of pozzolanic phenomena around the clay surfaces [4, 15].

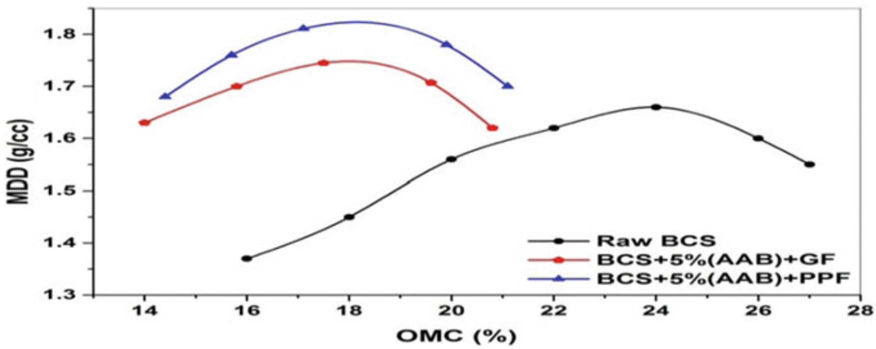

Fig. 7 Variation of MDD and OMC values of untreated BCS and fiber-AAB treated BCS

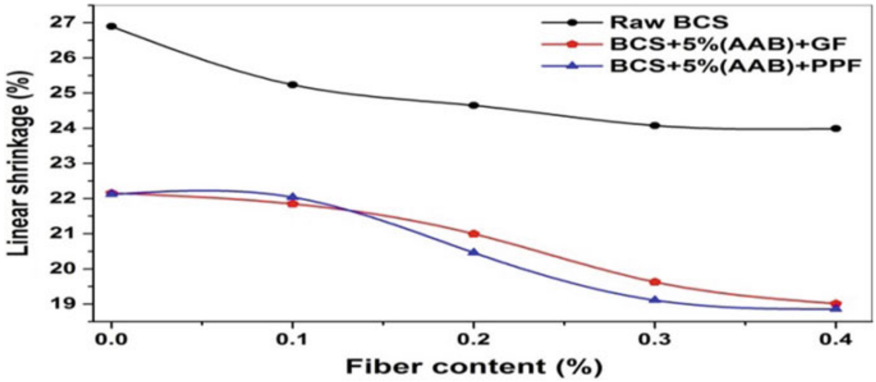

Fig. 8 Variations of linear shrinkage values of untreated BCS and fiber-AAB treated BCS

Consolidation and swelling pressure. Consolidation and swelling pressure tests are conducted in an Oedometer according to ASTM D-2435 and D-4546 standards. Both untreated BCS and fiber-AAB treated BCS samples are statically compacted at MDD and OMC in a consolidation ring of 20 mm height and 60 mm diameter. Figure 9a shows the slope of compressibility curve of treated and untreated BCS. In the *e*-log *p* curve of consolidation, raw BCS attains the highest equilibrium void ratio and the polypropylene fiber-AAB treated BCS attains least void ratio on saturation. However, the inclusion of glass fiber in the AAB treated BCS does not significantly alter the void ratio when compared to polypropylene fiber-AAB treated BCS. Figure 9b shows the time swell curve of untreated BCS, glass and polypropylene fiber reinforced AAB treated BCS. Raw BCS has the maximum swelling pressure of 78 kPa; both fiber-AAB mixtures blended soil attains maximum swelling pressure around 39 kPa. This reduction of swelling may be due to the formation of new mineralogy by dissolution of clay particles with pozzolanic additives [16, 17].

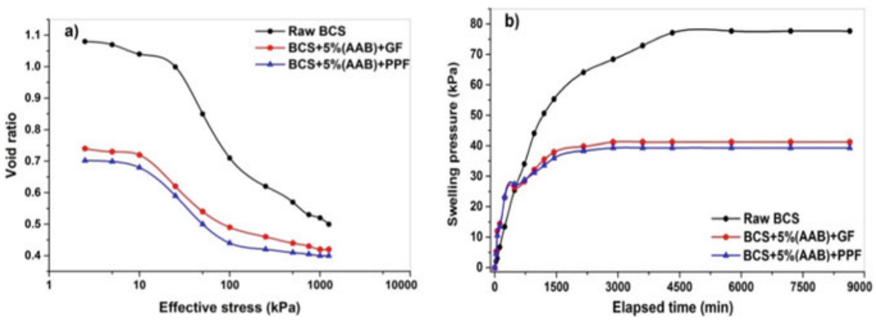

Fig. 9 Variation of **a** *e*-log (*p*) curves, **b** swelling pressure of untreated BCS and fiber-AAB treated BCS

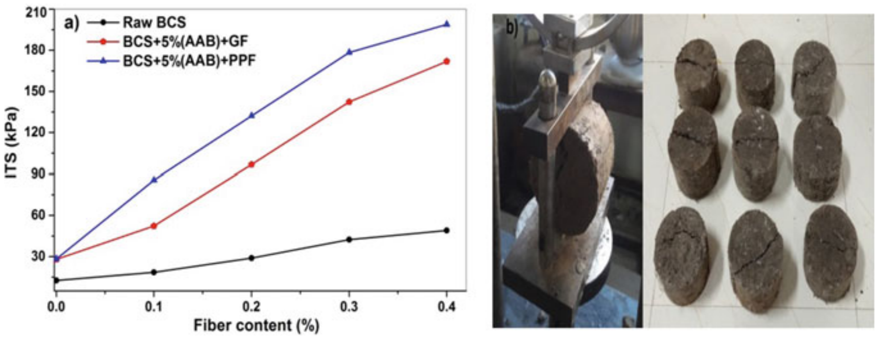

Fig. 10 **a** Variation of ITS values of untreated BCS and fiber-AAB treated BCS, **b** typical arrangement of ITS test

Indirect tensile strength (ITS). Indirect tensile tests are conducted as per ASTM D4123-1995 standard on both untreated and AAB treated BCS with different fibers. The soil specimens are prepared by maintaining 80 mm height and 100 mm diameter with a loading strip of 12.5 mm at a constant strain rate of 50.5 mm/min in a Marshall Stability machine. The samples are preserved for 24 h in the humidity chamber before testing. Figure 10a shows the tensile strength of soil with respect to fiber. Addition of glass and polypropylene fibers in the AAB treated BCS increases the tensile resistance strength from 24 to 190 kPa. Thus, the enhancement in ductile behavior of BCS may be because of the fiber surface morphology and pozzolanic reaction induced in the clay bonding. Figure 10b shows the typical arrangement of indirect tensile strength test of soil under the loading strip frame.

Unconfined compressive strength (UCS). Unconfined compressive strength tests are performed for untreated BCS, glass and polypropylene fiber reinforced AAB treated BCS as per ASTM D-2166 standard. Soil samples are molded in 38 mm diameter and 76 mm molds at MDD and OMC values under a fixed strain rate of 1.25 mm/min. Figure 11a shows the shear strength values of soil with respect to fiber. Combined addition of glass or polypropylene fibers in the treated BCS shows relatively higher shear resistance. This abrupt enhancement in shear strength may be attributed to the formation of interfacial friction and confinement bonding between the fiber and clay particles. Figure 11b shows the stress–strain curve of treated and untreated BCS. Thus, the inclusion of polypropylene fiber in AAB modified soil attains maximum compressive strength when compared to glass fiber-AAB treated soil.

California bearing ratio (CBR). Soaked and unsoaked CBR tests are performed for untreated BCS, glass and polypropylene fiber reinforced AAB treated BCS, as per ASTM D-1883 standard. Figure 12 shows the variations of soaked and unsoaked CBR values at 2.5 mm. The soaked CBR value of raw BCS is 1.96, indicating low strength. As seen from the graph, the inclusion of fly ash-based sodium aluminosilicate binder with discrete fiber matrix increases the soaked CBR value from 1.96 to 5.59%.

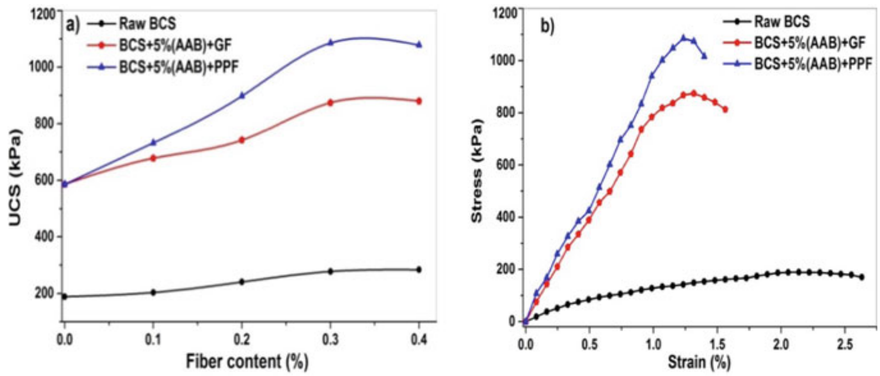

Fig. 11 **a** Variation of UCS with fiber content, **b** stress–strain curves of untreated BCS and fiber-AAB treated BCS

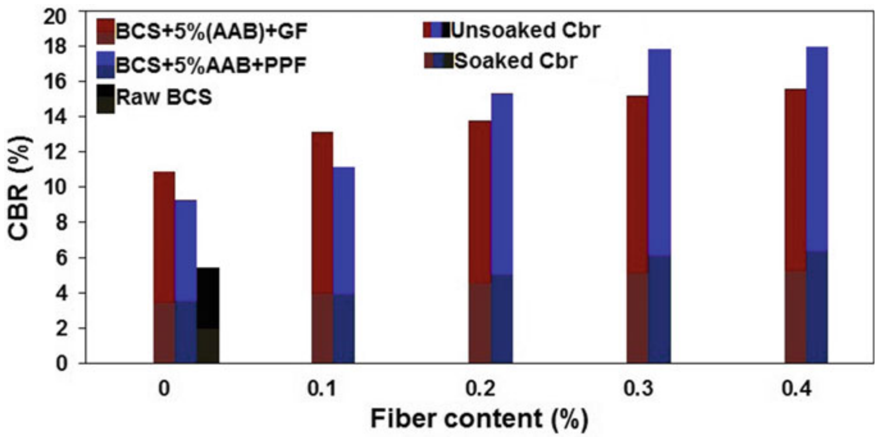

Fig. 12 Variation of soaked and unsoaked CBR values of untreated BCS and fiber-AAB treated BCS

This improvement in strength bearing ratio may be because of fiber bonding and pozzolanic reactions.

4 Conclusions

The present study compares the geotechnical and microstructure properties of untreated BCS, glass, polypropylene fiber reinforced with envirosafe alkali-activated binders (AAB). Flay ash-based sodium aluminosilicate binder serves a dual benefit of reducing traditional-based cementitious binders and preventing the disposal of industrial by-product through maintaining a green sustainable environment. The main conclusions that can be drawn from this study are as follows.

- The polypropylene fiber reinforced AAB treated BCS attains higher strength in terms of tensile, bearing capacity and shear resistance when compared to glass fiber-AAB treated BCS.
- Liner shrinkage, void ratio and swelling pressure of fiber-AAB treated BCS is significantly reduced by around 30%. Blending of 0.3% polypropylene and/or glass fiber in 5% AAB mixture aids to control the tension cracks significantly.
- The tensile strength and CBR of fiber-AAB treated BCS increase by around 63% and 45%, respectively, as the fiber content increases from 0 to 0.4%. Fiber-AAB treated BCS micrograph shows a strong interfacial surface interaction between the fiber and soil matrix.
- Microanalysis results confirm the formation of new crystalline phases and molecular vibrations in fiber-AAB treated BCS. In addition, stereomicroscopic images show that the fiber acts as a bridge network between the fiber and clay particles and effectively modifies the brittleness characteristic through friction and bonding.

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Strength Enhancement of Clayey Soil Using Cement Kiln Dust and Recron Fiber

Amrendra Kumar, Sunita Kumari, and Ravi Kumar Sharma

Abstract Due to the fast growth of economy, construction work is very rapid in India. The removal of poor layer of ground is uneconomical. Scarcity of land forces to search new material so that bearing capacity of poor ground can be improved. Therefore, alternative method is used for the stabilization of soil. Cement kiln dust (CKD) is a waste material obtained from the cement industries. This paper presents the stabilization soil having low bearing capacity by mixing CKD and Recron fiber. CKD has pozzolanic capacities which may improve the engineering behavior of clayey soil. An experimental study was carried out by mixing CKD with soil in the ratio of 96:04, 92:08, 88:12, 84:16, and 80:20. It is found that the un-soaked California bearing ratio (CBR) increases from 6.7 to 12%. The Recron fiber also improves the soil properties of loose soil due to the reinforcing action. The inclusion of fibers in the optimum mixture of soil and CKD is in the ratio of 0.1, 0.2, 0.4, and 0.8% by weight of mixture. The CBR value increases maximum for 0.4% of Recron fiber from 12.35 to 15.67%. The stabilized soil may be used as subgrade for construction of road.

Keywords Clayey soil · Cement kiln dust (CKD) · Recron fiber · California bearing ratio (CBR)

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1 Introduction

Clayey soils with low bearing capacity are extensively available. Therefore, it is impossible to avoid these sites for construction works. Many agencies use industrial waste material for the improvement of engineering properties of soil along with the feasibility of the project and its environmental effect. The productions of the waste materials have been increased tremendously in recent years in developing country. Cement kiln dust (CKD) is a fine, powdery material which contains reactive calcium oxide. The reactive behavior of CKD depends upon the location of dust collection system, the type of operation and the type of fuel used. When the Recron fibers mixed with soil, CKD mixture shows improvement in engineering properties. Fiber increases the penetration and tensile resistance which keeps subgrade intact and avoids the problem like potholes, cracking, and failure. Optimum moisture content of improve soil increases but cohesion decreases with addition of fibers content. The California bearing capacity, unconfined compressive strength and bearing capacity increase with the addition of fiber. Nicholson [8, 9] conducted a series of laboratory examinations on CKD and fly ash mixtures for stabilizing the sub-base materials with different aggregates and patent them. Napeierala [7] reported that CKD can be used as stabilizing material for sandy soils in subgrade applications. Baghdadi and Rahman [2] deliberate the application of CKD for stabilizing siliceous dune sand in highway construction. For the base materials, mix proportion of 30% CKD and 70% sand gave best results. Baghdadi et al. [3] reported a range from 12 to 50% of CKD was satisfactory to stabilize dune sand. Cement kiln dust (CKD) enhancing the mechanical as well as the hydraulic properties of soils in arid lands evaluated the potential use of Mohamed [6]. Experimental results show good improvement in compressive strength of the soil stabilized with CKD [10]. Amadi [1] assessed the durability by immersion tests on mixtures of black cotton soil 10% quarry fine and different percentage of CKD.

The Recron fiber reinforcement improves stress–strain patterns and progressive failure in place of quick post-peak failure of plain samples. The unconfined compressive strength of soil is increased by 7 times with CKD and 9 times for CKD with fiber modification [11]. Husain and Aggarwal [5] reported that CBR value and unconfined compressive strength of soil increase with the addition of Recron fiber. Sharma [12] conducted different laboratory tests on different percentages of cement kiln dust without fiber and with fiber content and concluded that improvement in engineering properties. Das and Singh [4] conducted a series of laboratory tests compaction test, unconfined compressive strength and California bearing ratio on soil reinforced with areca nut coir, water hyacinth stem and Recron fiber. Results show the improvement in the geotechnical properties of soil.

In this paper, the California bearing ratio test is carried out for strength analysis of natural soil and soil modified with the CKD along with Recron fibers.

2 Materials and Method

The sample of soil is obtained from under construction site of ISBT Bariya Patna. From the Indian standard (IS 1498-1970) system of classification, the soil from the test classified as medium plasticity clay and Table 1 shows the properties of clayey soil sample. Cement kiln dust from the laboratory of NIT Patna and Recron fiber from the market suppliers.

Laboratory tests on various materials are performed with according to IS code of 2720 different parts. The specific gravity tests (IS 2720-3-1-1980), consistency limit tests (IS 2720-5 1985), and the standard Proctor tests (IS 2720-7-1980) are performing in laboratory. The particle size distribution of clay is given in Fig. 1. The IS 2720-16 (1979) is used for California bearing ratio tests. The mold size is of 150 mm diameter and 125 mm height. The different type of sample is compacted in CBR mold of standard size at maximum dry density and optimum moisture content for un-soaked test. A surcharge of 50 N weight is used for testing. A 50 mm diameter

Table 1 Engineering properties of collected soils

Property	Soil
Specific gravity	2.612
Atterberg limits	
Liquid limits (%)	46.50
Plastic limits (%)	23.8
Plasticity index (%)	22.7
Soil classification	CI
Compaction parameters	
Optimum moisture content (%)	13.5
Maximum dry unit weight (g/cm^3)	1.767
CBR (%)	7.6

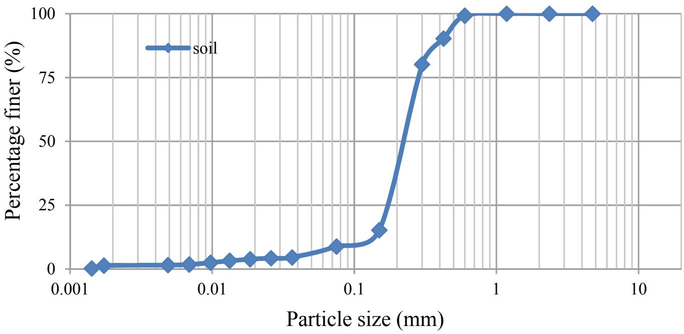

Fig. 1 Particle size distribution of clayey soil

and 100-mm-long-metal plunger is used for penetration of sample at the rate of 1.25 mm/min using in CBR testing machine.

3 Results and Discussion

3.1 Compaction Tests

The proctor test behavior of clayey soil modified with the different percentage of CKD is shown in Fig. 2. The CKD mixed in the soil in the proportion of 4, 8, 12, 16, and 20%. The maximum dry density of soil increase with the addition of CKD up to 12% then reduction is observed. The increment in the maximum dry density may be due the pozzolanic action and the presence of calcium ions in the CKD. The maximum increment for maximum dry density is observed at 12% addition of CKD. Figure 3 shows that optimum moisture content of the mixture. The optimum moisture content initially decreases with the addition of CKD up to 12% then increases. The

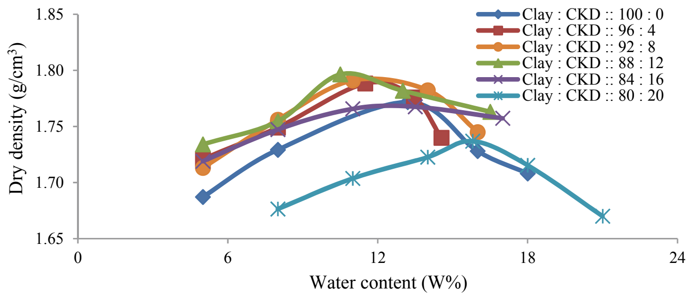

Fig. 2 Variation of dry density of different percentage of clay and CKD

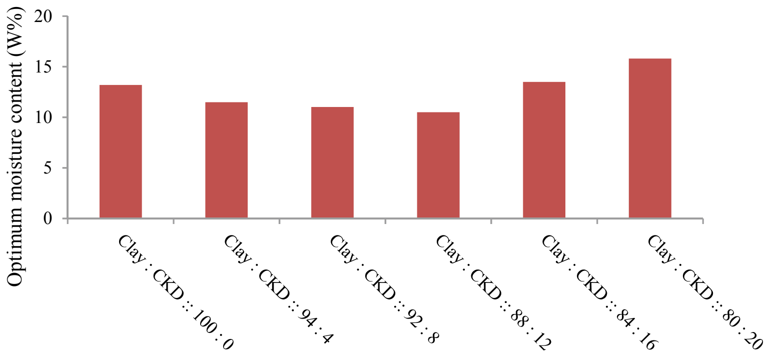

Fig. 3 Variation of optimum moisture content of different percentage of clay and CKD

reason behind this behavior is due to initial proper mixing of soil and CKD and reacts with each other. After optimum percentage of CKD, proper mixing is less.

Figure 4 shows the variation of dry density with water content for clay and CKD optimum mix and Recron-3S fiber. The results show that with the addition of fiber maximum dry density reduces this is due to the light weight and less specific gravity. Figure 5 shows the variation of optimum moisture content with the variation of fiber. It is also observed that with the addition of fiber, the moisture increases little due to the surface roughness of the fiber.

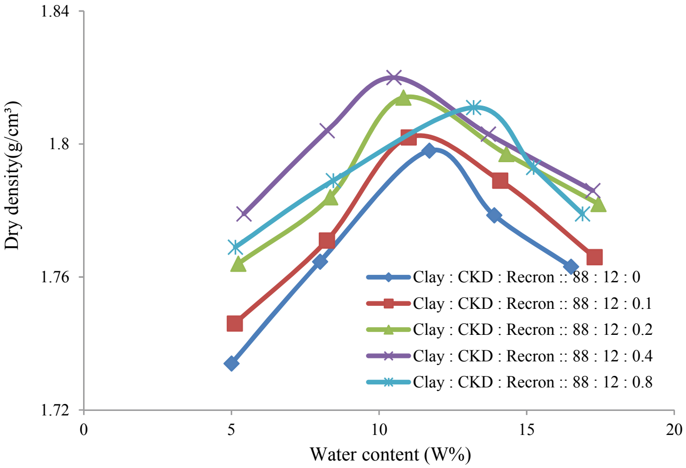

Fig. 4 Variation of dry density of different percentage of clay and CKD

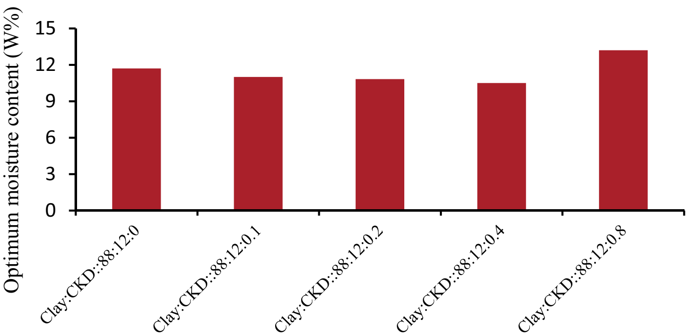

Fig. 5 Variation of optimum moisture content of variation of fiber

3.2 California Bearing Test

Figure 6 shows the variation of California bearing ratio value of clay and clay with different percentage of CKD. The CBR value increases up to optimum mixture of 88% clay and 12% CKD then decreases little bit which is not significant.

Figure 7 shows the variation of optimum mix of clay and CKD with the addition of fiber. The results show that with the addition of fiber the CBR value increases. The Recron fiber also improves the soil properties of loose soil by reinforcing action. The inclusion of fibers in the optimum mixture of soil and CKD is in the ratio of 0.1, 0.2, 0.4, and 0.8% of weight of mixture. The CBR value increases maximum for 0.4% of Recron fiber from 12.35 to 15.67%. The stabilized soil may be used as subgrade for construction of road. The addition fiber increases the penetration resistance due to better combination of bonding between the soil–CKD–fiber. After a certain limit,

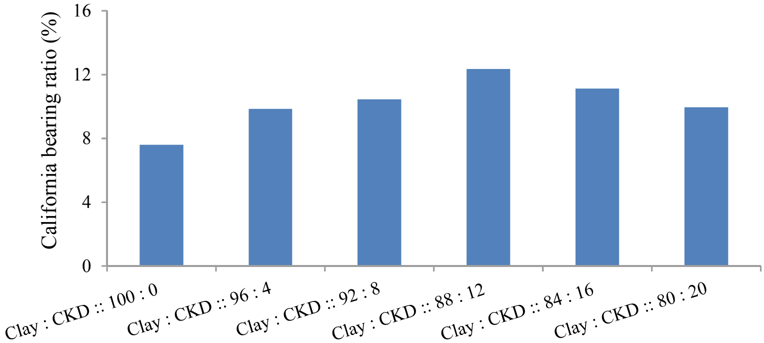

Fig. 6 Variation of California bearing ratio of different percentage of clay and CKD

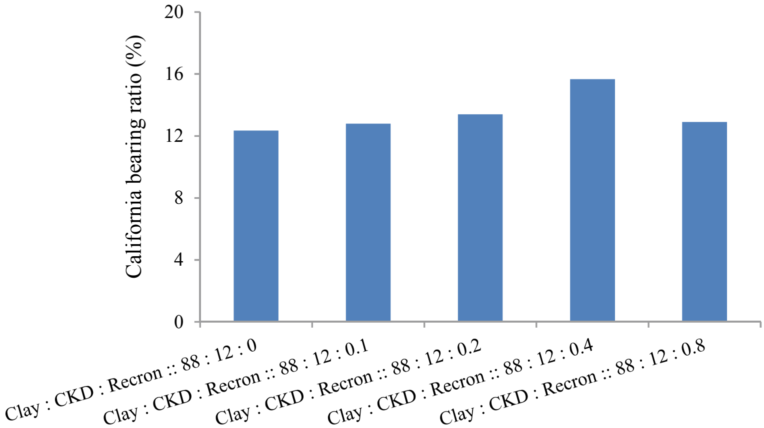

Fig. 7 Variation of California bearing ratio of different percentage of fiber with optimum percentage of clay and CKD mix

higher percentage of fiber not combined properly segregate the soil and lower value of CBR is obtained for 8% of fiber addition.

4 Conclusion

On the basis of various laboratories test and its results, following conclusions are drawn:

1. Compaction test results show the addition of CKD to the soil the maximum dry density increase up to 12% addition of CKD then after decreases. At higher percentage, the pozzolanic action decreases due to unavailability of reactive site.
2. Addition of Recron fiber the maximum dry density decreases due to lightweight nature of fiber.
3. California bearing ratio test shows optimum mix has maximum (88% soil and 12% CKD) value due to higher maximum dry density and lower moisture content.
4. Strength of soil can be increased to the certain extent by using Recron fibers in soil and CKD optimum mix.
5. Due to the addition of fiber the tensile, abrasion, and penetration properties of subgrade increases which keeps road surface intact and free from cracking and failure.

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Effect of Rice Husk Ash on the Behaviour of Highly Compressible Clay

E. R. Sujatha, M. Abijayan, M. Vignesh, and V. Shriram

Abstract This study explores the possibility of using rice husk ash blends for soil improvement. The results of the study show that rice husk ash renders the soil less plastic. The liquid limit, plasticity index and flow index of the rice husk ash–clay blends show a significant decrease with the increase in ash content. The compaction curves tend to shift to the right. The optimum moisture content increases, and dry density decreases with the increase in ash content. The addition of rice husk ash not only increases the unconfined compressive strength of the clay soil but also the post peak strength of the blends. An increase of approximately 100% is observed at 5% ash addition. The California bearing ratio (CBR) of the clay–rice husk ash blend is 37% more than the untreated soil. The results of the investigation emphasize the benefit of rice husk ash replacement with in-situ soil for improving the geotechnical properties.

Keywords Rice husk ash · Plasticity index · Unconfined compressive strength · California bearing ratio

1 Introduction

A good foundation is necessary is for a stable structure. If the selected site is not suitable for construction by virtue of the poor geotechnical properties of the soil (e.g. soft clay), and if that site cannot be avoided, ground improvement techniques are implemented. The various materials used for ground improvement are cement, lime, fly ash, bitumen, inorganic ashes of industrial and agricultural products like the ground granulated blast furnace slag [1] and ground nut shell ash [2]. Cementitious properties, possessed by the materials, help the soil to gain adequate bearing capacity. The CBR of the soil is from 4 to 10 times higher than that of untreated soil, when replaced with 6% lime and treated for two hours [3]. Also, an increase in unconfined

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compressive strength of 47% is observed when 12% of soil is replaced with rice husk ash [4].

In this study, the soil is replaced with various percentages of rice husk ash (RHA) and the geotechnical properties of soil–RHA blends are investigated. The primary objective of using rice husk ash is due to its abundant availability and immediate need to be disposed with economic value. According to a data given by Ministry of Environment, Forest and Climate Change, 4.65–5.68 million tonnes of RHA (15–18% of rice husk) is generated by India in 2014. Hence, the use of RHA for ground improvement will result in the utilization of large volume of RHA and will enable it to be properly disposed. It is also an economic alternative to traditional admixtures like lime, cement and bitumen [4–7]. Rice husk ash is not hazardous in nature and is also easier to handle.

2 Materials and Methods

2.1 Materials

Soil. The soil used for the experimental investigations was clay soil extracted from Thirumalaisamudram village, Thanjavur District, Tamil Nadu. The geotechnical properties of the soil are given in Table 1.

The soil is classified as highly compressible clay according to Indian Standard soil classification system. The unconfined compressive strength of the soil shows that it falls in the moderately stiff consistency when moulded at its optimum moisture content.

Table 1 Geotechnical properties of soil

Geotechnical property	Value
Specific gravity (G)	2
Liquid limit (%)	60
Plastic limit (%)	20.64
Plasticity index (%)	39.36
Shrinkage limit (%)	8.27
Shrinkage index (%)	12.37
Shrinkage ratio	2.05
OMC (%)	15
Maximum dry density (kN/m^3)	13.05
Unconfined compressive strength (kPa)	90.59
Undrained cohesion (kPa)	45.29
CBR	3.69

Table 2 Percentage of elements (in oxide form) of RHA

Formula	SiO_2	K_2O	P_2O_5	CaO	Na_2O	SO_3	Cl	Fe_2O_3	Al_2O_3	PbO
Concentration (%)	95.6	1.29	0.85	0.65	0.26	0.14	0.14	0.13	0.12	0.09

Rice Husk Ash (RHA). The rice husk ash is brought from Mannargudi District, Tamil Nadu. The ash is greyish white in colour, and its incineration temperature is about 600 °C. It has significant silica content, making the ash a good pozzolanic material. The percentage of elements (in oxide form) present in rice husk ash is given in Table 2.

2.2 *Methods*

The obtained soil sample is air-dried for 24 h, followed by oven drying for 24 h, to prevent higher water absorption. Then, soil samples are blended with rice husk ask in percentages of 2.5, 5, 7.5, 10 and 12.5%. All samples were mixed with the required water content and were sealed in airtight packs for 24 h to allow the soil reach equilibrium water content. The experiments were carried as per IS 2720 specifications.

Consistency Limits

Liquid and plastic limit. The tests are done in accordance with IS 2720 (Part V)-1985. Samples passing through 425 μm are used.

Shrinkage limit. This test is done as prescribed in IS 2720 (Part VI) 1972-78. The sample is mixed with a water content slightly above liquid limit, and oven dried for 24 h.

Compaction. Standard Proctor Test is done in accordance with IS 2720 (Part VII)-1965. The sample is 100 mm in diameter and 127.3 mm in height. The results obtained in this experiment are vital, as it directly influences CBR value, compressibility, compressive strength, etc.

Unconfined Compressive Strength. The test is done according to the procedure stipulated in IS 2720 (part X)-1991. 300 g of sample are mixed at OMC and then kept in an airtight container for 24 h, for the soil to attain equilibrium moisture content. Then, the soil is moulded to 40 mm in diameter and 80 mm in height.

California Bearing Ratio. The test is done in accordance to IS 2720 (Part XVI)-1987. Five kilograms of sample is mixed at OMC and then kept in an airtight container for 24 h, for the soil to attain equilibrium moisture content. Then, the soil is moulded to cyclinders, 150 mm in diameter and 175 mm in height.

3 Results and Discussion

3.1 Consistency Indices

The addition of rice husk ash to soil decreases the liquid limit to considerable amounts, but increases the plastic limit of the blends (Fig. 1). Decrease in liquid limit indicates a decrease in the compressibility of the soil. Plasticity index shows a significant reduction indicating the change in plastic behaviour of the soil. Reduction in plasticity index of the soils indicates a better bearing material. The reasons for this trend can be due to the non-plastic nature of rice husk ash and its tendency to form flocs that leads to increase in particle size [8], decrease in plasticity and high shear strength [8] as a result of the physio-chemical interaction between the charged clay surface and RHA particles [8]. These changes lead to a shift in the classification of these soil–RHA blends from high compressible clay to silt (blend) of low compressibility (Fig. 2).

The shrinkage limit of the soil increases gradually up to 13.82% at 12.5% RHA addition (Fig. 1) indicating that RHA controls the volume change susceptibility of the clay soil. Shrinkage limit of RHA blend shows an increase of nearly 67% at 12.5% addition. This again indicates that RHA addition improves the soil as a better bearing medium.

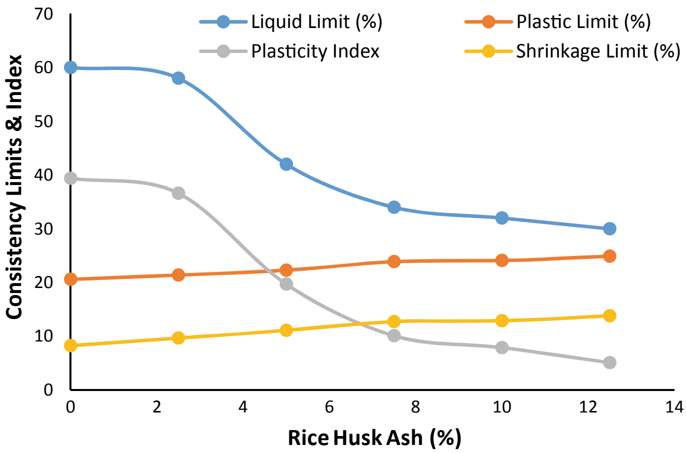

Fig. 1 Effect of rice husk ash on consistency limits and index

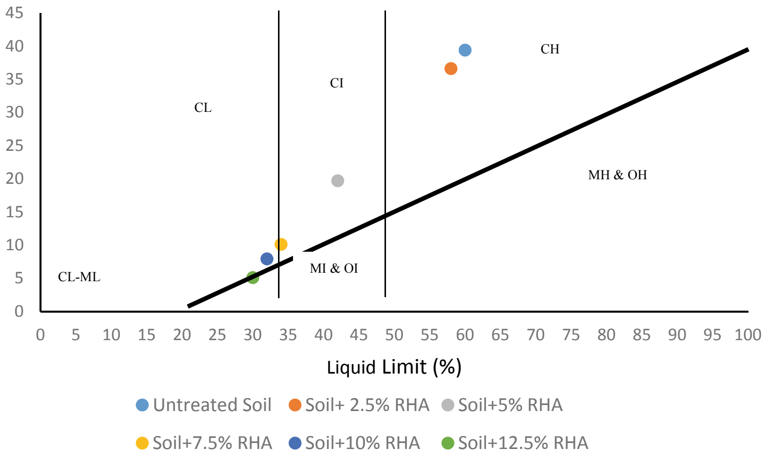

Fig. 2 Effect of rice husk ash on classification of soil

3.2 Compaction

The compaction curves are flatter with increase in RHA content (Fig. 3) and this indicates that the soil can be compacted over a range of water contents [9], thereby reducing the moisture sensitivity. This characteristic can be beneficially applied in the field by compacting the soil on the dry side of optima with higher shear strength and better drainage capacity to an almost equivalent dry density.

Maximum dry density decreases, and optimum moisture content increases on addition of rice husk ash to the soil (Fig. 4). RHA is lighter than soil particles and when replaced with soil the dry density decreases with the increase in RHA content [2]. The reason for the increase in OMC with increase in RHA content can be attributed to the formation of flocs that increase the void ratio of the soil–RHA

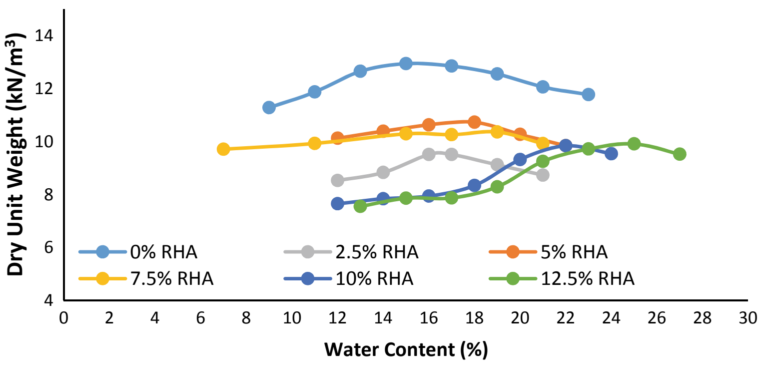

Fig. 3 Compaction curves for different samples

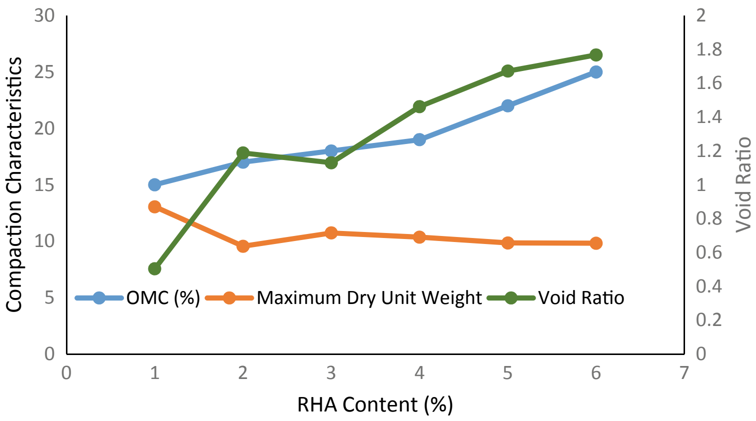

Fig. 4 Variation of compaction characteristics with RHA content

blends. These voids entrap water in them increasing its water holding capacity [5]. This is evident from Fig. 4 which shows an increase in void ratio with increase in RHA content.

3.3 Unconfined Compressive Strength (UCC)

Soil samples blended with rice husk ash exhibit a clear peak than that of the untreated soil sample. The failure stress increases up to 5% RHA addition and then decreases, but then, is still higher than that of untreated soil. At 5% RHA, the failure stress is nearly twice than untreated soil. Figure 5 shows the stress–strain behaviour for various soil–RHA blends, and Fig. 6 depicts the variation in unconfined compressive strength of the soil–RHA blends.

Upon investigating the surface characteristics of rice husk ash, it possesses negative zeta potential, indicating the formation of negatively charged calcium silicate [10]. The increase of UCC is due to the presence of silicon dioxide and calcium oxide in rice husk ash, which reacts in the presence moisture to form calcium silicate, resulting in strong and stable calcium–silicon–oxygen bonding, and thus improving the strength of clay–RHA blends. Also, the reason for the decrease of UCC of successive samples can be attributed to more accumulation of similar charged RHA particles, thus resulting in repulsion.

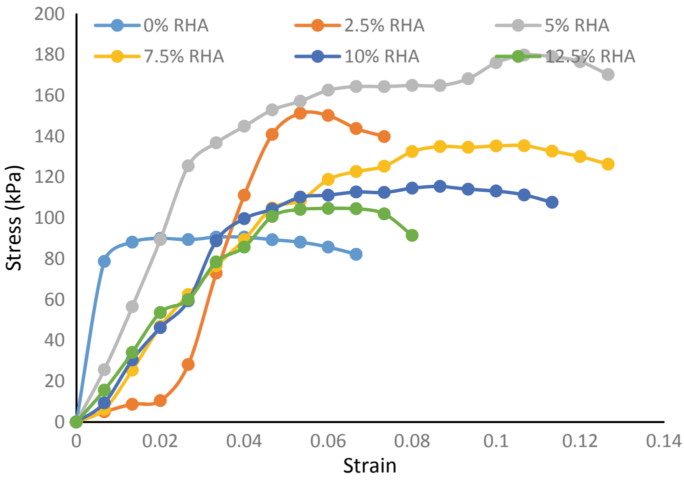

Fig. 5 Stress–strain curves for various samples

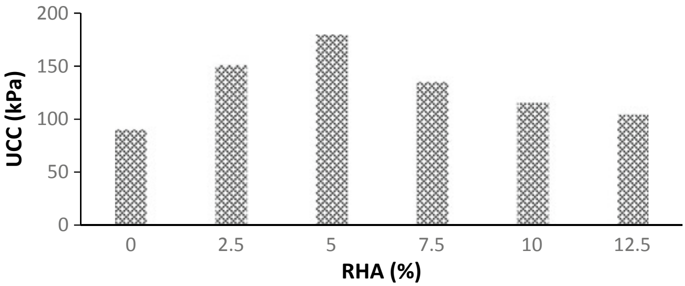

Fig. 6 Variation in UCC for various samples

3.4 *California Bearing Ratio (CBR)*

The variation of CBR is quite similar to that of UCC. CBR value increases from the sample untreated soil to 5% soil–RHA blend, and then decreases. The CBR value of 12.5% soil–RHA blend is less than that of the untreated soil. Table 3 shows the variation of CBR values of various soil–RHA blends.

At 5% RHA, CBR value increases by about 37%, when compared to that of untreated soil. The reason for the increase is due to the reaction of high amount of

Table 3 Variation of CBR values of soil–RHA blends

RHA (%)	Soil	2.5	5	7.5	10	12.5
CBR value	3.83	4.76	5.15	3.96	1.19	1.02

silica in rice husk ash, with calcium to generate pozzolanic materials, which improves the strength of soil–ash blends [4, 11]. Also, the reason for the decrease of CBR value of succeeding samples with RHA content greater than 5% is due to increasing availability of non-plastic fines, which tends to reduce the bonding between particles, and hence, the reduction in CBR value is noted [11].

4 Conclusions

The effects of geotechnical properties on addition of rice husk ash were analysed and are as follows. The liquid limit of the soil showed a steep decrease, while the plastic limit showed a marginal increase, on addition of rice husk ash. The shrinkage limit of the soil marginally increases upto sample 7.5% RHA and marginally decreases for further addition of RHA. The optimum moisture content increases, and the maximum dry density of the soil decreases, on addition of rice husk ash. The unconfined compressive strength of soil steeply increases at 5% addition of RHA, and then decreases on increasing the RHA content further. The California bearing ratio of the soil increases by nearly 37% at 5% RHA content. Results indicate that 5% RHA addition to soil can improve the strength optimally. However, there are some practical hurdles in implementing this method of ground improvement. India is a tropical country, and Tamil Nadu lies on the peninsular region and is subjective to seasonal winds and monsoons. The optimum moisture content might be lost at hotter temperatures, or the heat of hydration, which brings the strength in soil, might not be sufficient in colder temperatures. Also, people residing nearby construction sites might face dust issues, due to wind. Also, the properties of rice husk ash depend on the incineration temperature and grinding method which is a major limitation. In spite of being an economical ground improvement method, further research is required for its practical implementations.

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Geotechnical Properties of Lime Treated Soil Contaminated with Sulphatic Water

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Abstract Chemical stabilization using calcium-based stabilizers such as lime and cement to improve the properties of soils is well-known technique since previous several decades. However, the longevity potential of calcium-based stabilized soils with change in environmental conditions, particularly, in migration of contaminated sulphatic water is a matter of concern for the geotechnical engineer. The gypsum ($Ca_2SO_4.2H_2O$) is main source of sulphate and is abundantly available in soils throughout the world, despite of its low solubility rate. The present work is aimed to study the potential of lime stabilized soil contaminated with migration of sulphatic water. Detail experimental works to determine the plasticity, compaction characteristics and one dimensional oedometer swell percentage have been performed in expansive soil alone/and stabilized with optimum lime content with water having sulphate concentrations of 0, 3000, 5000, 10,000 and 20,000 ppm. The result shows that the plasticity of expansive soil contaminated with sulphatic water reduces drastically with lime treatment. Further, Optimum Water Content (OWC) of soil is observed to be less than lime treated soil contaminated with various concentration of sulphatic water, whereas maximum dry density (MDD) of sulphate contaminated soil reduces with lime treatment. It is interesting to observe that lime treated soil exhibits drastic swell after inundating with sulphatic water having different concentration. The formation of highly expansive ettringite mineral by ionic reaction between calcium-aluminium-sulphate in the presence of water results the swell in lime treated soil.

Keywords Compaction · Gypsum · Plasticity · Sulphate · Swell

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1 Introduction

The expansive soils occur around all over the world. In India, expansive soil is also known as black cotton soil (BCS) which covers an area of 0.8 million km^2 (about 20% of total land area) of country. The major areas of their occurrence in India are states of Maharashtra, Gujarat, southern parts of Uttar Pradesh, eastern parts of Madhya Pradesh, parts of Andhra Pradesh and Karnataka. This type of soil is available up to a depth of 3.7 m on an average in the above parts of India [1]. Expansive soils undergo volumetric changes with temporal variation. Increase in swelling and loss of strength occur in the presence of water/moisture and reduction in moisture content leads to huge crack and shrinkage. Cyclic swell-shrinkage of expansive soil causes differential settlements, resulting in severe damage to the foundations, buildings, roads, retaining structures, canal linings, etc.

Although several ground improvement techniques are adopted to stabilize and modify the expansive soils. However, chemical treatment of such soils by using lime or cement is considered as a most viable and economical method of stabilization. Since cement is costly and scarce resource, lime has been commonly used since past many centuries as a soil stabilizer. Generally, four reactions are attributed for modification of properties of soil lime-mixtures [2]. They are: (a) Cation exchange; (b) Flocculation/agglomeration; (c) Carbonation; and (d) Pozzolanic reaction. It improves the strength and durability of soils by ion exchange and cementitious reactions. Mehta et al. [1] reported that all lime treated fine-grained soils exhibit a reduction in plasticity, improved workability and reduced volume change characteristics. However, all soils do not exhibit improved strength characteristics. It should be emphasized that the properties of soil-lime mixtures are dependent on many variables such as soil type, lime type, lime percentage and curing conditions (time, temperature and moisture) [3].

The application of cement and lime to improve the characteristics of soft fine grained soils is not novel [4–6]. However, recent studies reported that calcium-based stabilization in presence of sulphates create more distress [7, 8]. Gypsum is the major source of sulphate present in soils, despite of its low rate of solubility [9]. Lime treated soil leads to the induced heave due to the formation of ettringite mineral at a highly alkaline environment ($pH > 10.5$) by reaction of calcium, aluminium and sulphates the presence of water [7, 8, 10]. The formation of such minerals is due to presence of sulphates in soil; thus, before the application of lime, it is important to understand the nature of sulphates in soil. The presence of sulphates either in ground or mixing in water may affect the cation exchange and pozzolanic reactions of lime treated soil systems [11]. Serious structural damages including uplifting of tunnel floors, rock under dams, embankments and roads due to heaving and settlement during the hydrations of different calcium sulphate phases have been reported [11–13]. The ettringite formation is controlled by various factors such as clay minerals present, pH, water content, sulphate content and temperature [7, 10, 11]. Hunter [7] reported that the sulphate only affects the pozzolanic reactions, i.e. the long term reaction in lime treated soil, and hence, immediate formation of ettringite is discarded. However, longevity potential of lime stabilized soil needs to be validate subjected to sulphate migration through surrounding surface or sub-surface water bodies.

In the present study, an attempt has been made to examine the physical and swell behaviour of untreated and lime treated soil subjected to migration of sulphatic water. Atterberg's limit, compaction characteristics and one dimensional oedometer swelling tests are performed in BCS and BCS treated with optimum lime content subjected to sulphatic water. Gypsum is taken to synthesize the sulphatic water in order to quantify amount of sulphate concentration (0–20,000 ppm) which affects the durability of lime treated soil.

2 Materials Used and Methodologies Followed

The black cotton soil (BCS) used in the present study is collected from Shivdaspura village, near Jaipur District, Rajasthan-303903, India. The soil was excavated from the depth of 1–1.5 m to the ground. The physical properties of soil are presented in Table 1. The particle size analysis is done as per Indian Standard (IS)-2720 (Part 4) [14]. The combined curve of wet sieving and hydrometer analysis (Fig. 1) shows the presence of sand sized particle (4.75–0.075 mm) of 11.00%, silt sized particle (0.075–0.002 mm) of 13.00% and clay sized particle (<0.002 mm) of 76.00%. It is observed that black cotton soil is predominated with clayey size particles. The specific gravity (IS-2720 (Part 3)) [15] of soil is observed to be 2.38. Atterberg's limits of untreated and lime treated soil are determined by following the standard procedure of IS-2720 (Part 3) [16], IS-2720 (Part 5) [17], respectively.

Table 1 Properties of black cotton soil (BCS)

Property	BCS
Sand (4.75–0.075 mm), %	11.00
Silt (0.075–0.002 mm), %	13.00
Clay (<0.002 mm), %	76.00
Specific gravity	2.37
Liquid limit, %	45.00
Plastic limit, %	24.56
Plasticity index, %	20.44
Shrinkage Limit, %	11.65
Differential free swell index, %	70.00
Optimum water content, %	22.06
Max. dry density, g/cm^3	1.60
CBR, %	1.62
pH value	7.50

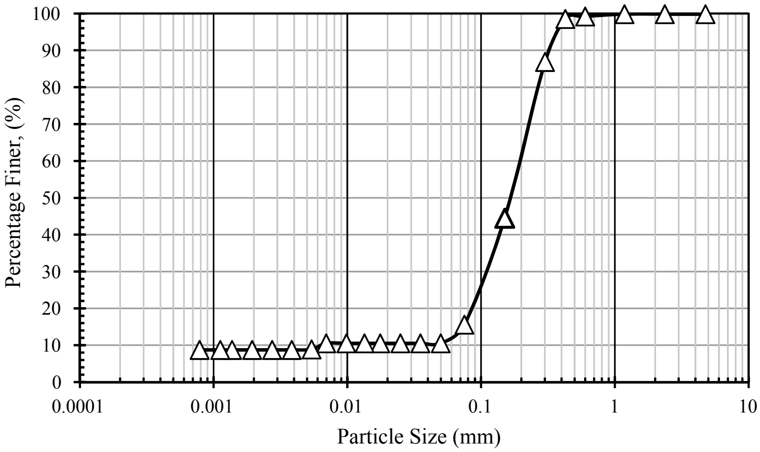


Fig. 1 Particle size analysis of black cotton soil

Mini compaction test procedure developed by Sridharan and Sivapullaiah [18] is used to determine the maximum dry density and Optimum Water Content (OWC) values of untreated and treated soil. The swell percentages of all sample are carried out as per respected Indian standard code IS 2720 (Part 15) [19].

The XRD analysis of soil shows the presence of montmorillonite, aluminium oxide and quartz as predominant minerals (Fig. 2). Also, field emission scanning electron microscope (FESEM) is performed to examine microstructural composition of soil. Microscopic images of black cotton soil (Fig. 2) illustrates the presence of several voids with honeycomb networking patterns. Energy dispersive X-ray spectroscopy (EDAX) is performed to observe the chemical composition of black cotton soil. It is found that BCS is predominated with silica (Si) and aluminium (Al) (Table 2).

Laboratory reagents hydrated lime ($Ca(OH)_2$) and Gypsum ($CaSO_4.2H_2O$) are used as chemical additives.

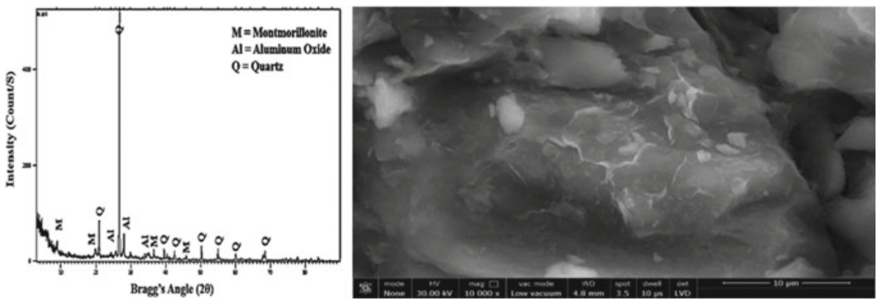


Fig. 2 XRD and SEM examination of black cotton soil

Table 2 Chemical composition of soil

Element	Atomic weight (%)
O	66.41
Si	15.38
Al	12.62
Fe	3.85
K	0.67
Mg	0.54
Na	0.53
C	0.00
Total	100.00

3 Results and Discussions

3.1 Determination of Optimum Lime Content (OLC)

The pH test is conducted on soil with different amount of lime to examine the optimum amount of lime to be used for the stabilization as procedure of Eades and Grim [20]. It was observed that the pH became constant after addition of 6% of lime to the soil by weight (Fig. 3). Hence, 6% lime is considered as optimum lime content and is taken to treat the soil in present study.

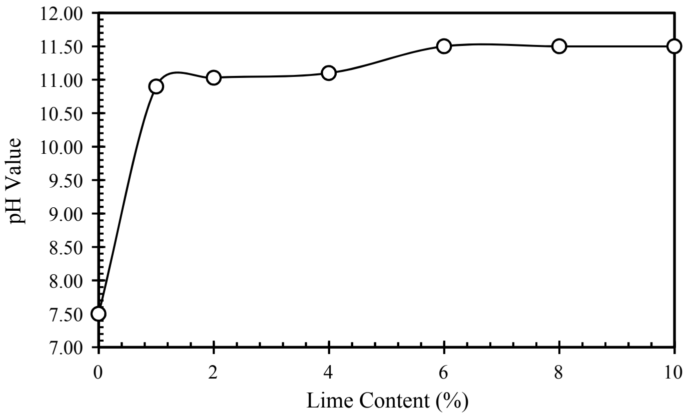


Fig. 3 pH value of soil-lime mixes

Table 3 Atterberg's limits for untreated and lime treated soil with sulphatic water

Sulphate concentration (ppm)	P.L. (%)		L.L. (%)		P.I. (%)	
	Soil	Soil + 6% lime	Soil	Soil + 6% lime	Soil	Soil + 6% lime
0	24.56	36.00	45.00	38.00	20.44	2.00
3000	34.70	26.60	53.00	33.00	18.30	6.40
5000	30.90	22.30	49.00	31.00	18.10	8.70
10,000	30.10	21.10	48.00	30.34	17.90	9.24
20,000	22.05	20.10	45.00	29.67	22.95	9.57

3.2 Atterberg's Limits of Lime Treated Soil

The effect of lime in the Atterberg's limits of BCS is presented in Table 3. It is observed that increase in lime content leads to decrease the liquid limit and plasticity index of soil. The improvement in plasticity of soil with lime treatment is due to the cation exchange and reduction in double diffuse layer by an increase in the electrolyte concentration of pore fluids. The plastic limit subsequently increases with lime treatment of soil.

3.3 Atterberg's Limits of Untreated Soil and Lime Treated Soil with Sulphatic Water

Effect of sulphate in Atterberg's limits of untreated and lime treated soil is presented in Table 3 and is shown in Figs. 4 and 5.

It is observed that liquid limit and plastic limit of soil increase initially with lower sulphate concentration of 3000 ppm and reduce thereafter (Fig. 4). Further, reduction in both liquid limit and plastic limit are observed for lime treated soil with increase in sulphate concentration (Fig. 5). However, reduction in liquid limit and plastic limit are observed to be minimal after 5000 ppm sulphate concentration. On contrary, the plasticity index of lime treated soil increases significantly up to 5000 ppm sulphate concentration and marginal thereafter. It is interesting to note that sulphate contamination of untreated and lime treated soil exhibits a degradation in soil plasticity index. The presence of calcium in gypsum and lime and possible nucleation and formation of ettringite mineral are responsible for variation in Atterberg's limits.

Fig. 4 Effect of sulphate on Atterberg's limits for soil

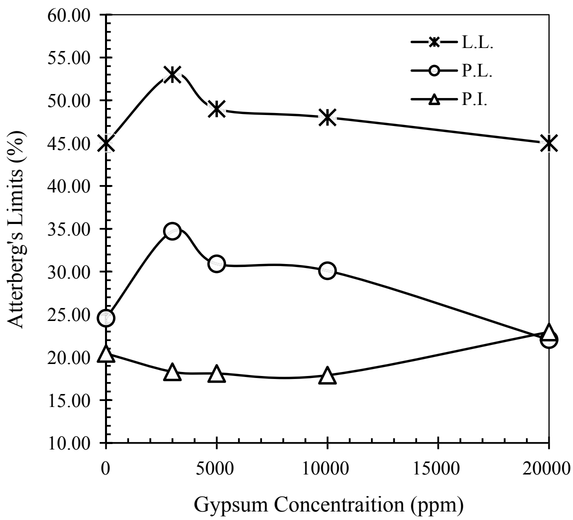
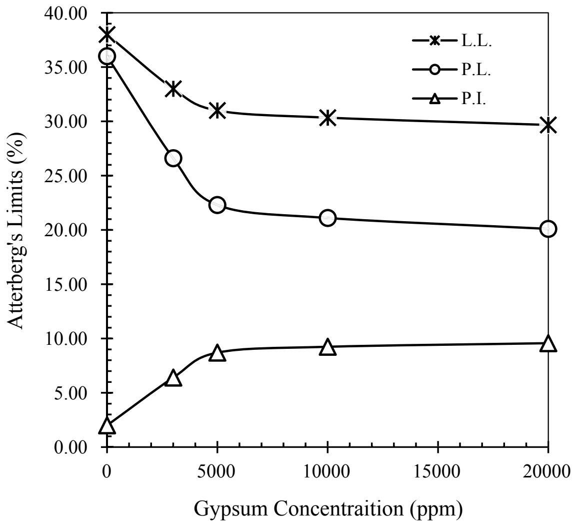


Fig. 5 Effect of sulphate on Atterberg's limits for lime treated soil



3.4 Compaction Characteristics of Untreated and Lime Treated Soil with Sulphatic Concentration

Compaction improves the strength and stability of the soil. The compaction characteristics of untreated soil and lime treated soil in the presence of varying sulphate concentration are presented in Table 4 and are shown in Fig. 6.

It is observed that marginal variation in maximum dry density and Optimum Water Content (OWC) of soil has been observed in the presence of sulphate concentrations.

Table 4 Compaction characteristics of untreated and lime treated soil with sulphatic concentration

Sulphate concentration (ppm)	ρ_{max} (g/cc)		OWC (%)	
	Soil	Soil + 6% lime	Soil	Soil + 6% lime
0	1.60	1.46	22.06	28.23
3000	1.65	1.42	18.66	21.90
5000	1.65	1.45	23.36	23.74
10,000	1.64	1.45	22.46	26.12
20,000	1.64	1.47	19.84	28.10

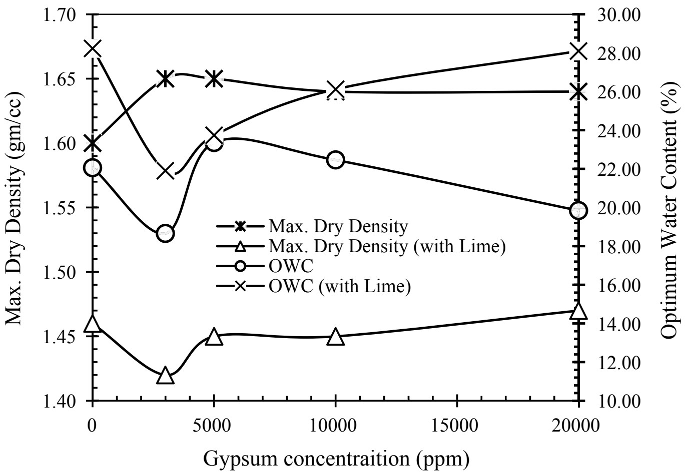


Fig. 6 Compaction characteristics of untreated and lime treated soil with sulphatic concentration

However, the dry density and OWC of lime treated soil reduce in the presence of lower sulphate concentration up to 3000 ppm and increase thereafter. Comparing the compaction characteristics of untreated and lime treated soil, significant reduction in dry density and increase in OWC of lime treated soil are pronounced at any sulphate concentration than that of same measured with untreated soil.

3.5 *Effect of Sulphate Contamination on Swell Behaviour of Lime Treated Soil*

The swelling percentage of untreated and lime treated soil is presented in Table 5 and is shown in Fig. 7.

The swell percentage of soil increases up to 5000 ppm and reduces drastically thereafter. The reduction in swell is due to the increase in percentage of calcium and thereby, formation cementitious compounds. Further, no swell is observed in soil after

Table 5 Maximum swell percentage of untreated and lime treated soil at different sulphate concentration

Sulphate concentration (ppm)	Swell percentage (%)	
	Soil	Soil + 6% lime
0	2.53	0.00
3000	2.58	0.52
5000	3.08	0.60
10,000	0.96	1.40
20,000	0.24	1.24

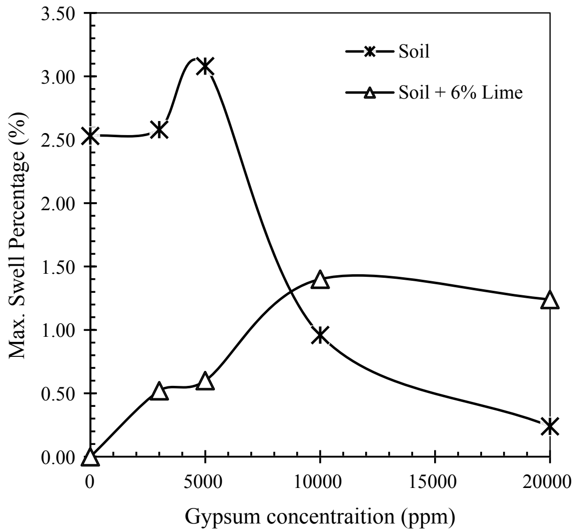


Fig. 7 Maximum swell percentage of untreated and lime treated soil at different sulphate concentration

lime treatment. However, inundation of lime treated soil with varying concentration of sulphatic water exhibits the drastic increase in swell percentage. The formation of ettringite mineral due to the ionic reactions between aluminium, calcium and sulphate is mainly responsible for sulphate induced swell in lime treated soil. Hence, it can be concluded that migration of sulphatic water leads to induced heave in the soil.

4 Conclusion

The major conclusions drawn from the present study are as follows:

1. Sulphate contamination affects adversely the plasticity behaviour of soil and lime treated soil.
2. Lime treated soil at any sulphate concentration undergoes significant reduction in dry density and increase in OWC as compared to same with soil.

3. Swell percentage of untreated soil increase with lower sulphate concentration and reduces drastically with higher sulphate concentration.
4. Drastic increase in swell percentage of lime treated soil has been pronounced in the presence of sulphate concentration. Hence, migration of sulphatic water leads to the cause of induced heave in the lime treated soil.

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Geotechnical Properties of β-Glucan Treated High Swelling Clay

M. Vishweshwaran, S. T. Soundarya, and E. R. Sujatha

Abstract Soil stabilization refers to enhancing the properties of a soil for engineering purposes. A holistic approach in selection of materials for soil stabilization is required since certain materials may cause environmental problems such as pollution during manufacturing, interaction with ground water table, soil nutrients, etc. β-glucan, a biopolymer offers a novel alternative and has greater potential for application in soil stabilization, yet only limited research is available. β-glucans are linear homopolysaccharides composed of D-glucopyranosyl residues linked via a mixture of β-(1–3) and β-(1–4) linkages. β-glucan has good tensile strength along with good bonding and adsorption characteristics. In addition to structural orientation, molecular weight plays a role in stabilization owing to gel forming potential. Polysaccharides have the tendency to form a film coating. β-glucan was added in quantities of 0.5, 1, 1.5, 2, 2.5, and 3% to high swelling clay soil. Index and select engineering properties of the treated soil were determined. Rheological properties such as pH and viscosity were also studied. Durability tests, tests on biopolymers such as gel matrix formation and its degradation were conducted. The test specimens were prepared for 0, 7, 14, 28, and 56 days to study the impact and variation of shear strength on treated soils. Scanning electron microscopy images revealed the bonding of β-glucan and soil. The treated soils have shown improvement in engineering properties.

Keywords Soil stabilization · Biopolymer · Engineering properties

1 Introduction

With rise in global warming, potential environmental friendly solutions could be harnessed efficiently in various industries. Biopolymers are produced naturally by living organisms and their final disposal need not require any treatment since they are biodegradable. Depending on the natural source, various types of polymers in the form of polysaccharides with different matrices are present [1]. β-glucan is a natural

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fiber present in oats, yeast, mushrooms, etc. It is a valuable non-starch polysaccharide present in the cell walls of oats, barley, etc. [2]. Biopolymers have the ability to form an interconnected tumesced polymer network. Biopolymers have applications in improving the stability of food items in spread products [3]. They are also used in cosmetics, pharmaceuticals, materials science industries [4]. In textiles, utility of xyloglucan results in strength increment of yarns [5]. Some of the biopolymers are chained with the opposing groups which improve the interfaces of the biopolymers rendering them to be dissolved in water and oil [6]. Molecular weight of a biopolymer is an important factor in rheological properties of β-glucan [2].

Agar biopolymer induced cohesion to cohesionless soil and improved the strength of the soil [7]. Even at low concentrations, xanthan gum, agar, gellan gum have tremendous improvement in strength of the soils. Biopolymers are prone to degradation and one way to improve the durability of them is to offer thermal treatment [8]. Consolidation test revealed that xanthan gum reduced the compression and swell indices of the expansive soils [9]. Utilization of xanthan gum on sandy soil improved cohesion and friction angle but the increase in cohesion was predominant. Xanthan gum increased the stiffness and decreased the hydraulic conductivity of the sand [10]. Xanthan gum soil samples tested after 750 days exhibited a little increment in strength without any degradation [11]. Biopolymers are generally known to increase the plasticity of soil [12]. Galactomannans are not influenced by strength of the ions because they are anionic. Guar gum is durable enough to resist adverse temperature cycles [4]. Literature studies indicate the potential benefits of biopolymers in soil strengthening, permeability, decontamination of soil, etc.

2 Methodology

2.1 Material

Commercially available sodium bentonite was used. The free swell index of the bentonite was found to be 430%. Sodium bentonite contains montmorillonite and illite mineral groups which cause the soil to swell tremendously. For the bentonite, the liquid limit was found to be 240%. Plasticity index was 174%. Flow and toughness indices were determined as 97.98 and 1.77, respectively.

2.2 Sample Preparation

The specimens for unconfined compression test were prepared by wet mixing method owing to their better performance than dry mixing in clayey soils [13]. Water was added to β-glucan powder at its optimum moisture content when tested with sodium bentonite clay. Thus, β-glucan solution was obtained and kept sealed for 2 h. The β-glucan solution was then thoroughly blended with the bentonite and enclosed to prevent loss of moisture.

2.3 pH and Viscosity

pH test was performed in accordance with IS: 2720 (Part 26)—1977 [14]. 30 g of the untreated, and treated soil were taken in a 100 ml beaker, and 75 ml of water was added. The suspension was stirred for few seconds, and pH meter was calibrated using standard buffer solution. Electrodes were washed with distilled water, dried with the help of an ordinary filter paper, and then immersed in the soil suspension. Three readings of the pH of the soil suspension were taken with brief stirring in between each reading. For determination of viscosity, a series of standard solutions of polymer was prepared and for each standard solution, the flow time was calculated. Viscosity of the biopolymer was tested using U tube viscometer.

2.4 Gel Matrix and Dehydration Tests

β-glucan powder of 1.3 g was poured in a measuring jar followed by addition of water for 0.05 L. This test was performed to interpret the hydration of β-glucan. Dehydration test involved the exposure of β-glucan to sunlight for 180 days by sealing the top of the measuring jar.

2.5 Liquid and Plastic Limits

Consistency of a fine grained soil is the physical state in which it exists. It indicates the degree of firmness of a soil. At liquid state, the soil offers no shearing resistance and can flow like a liquid. Liquid and plastic limits were determined as per IS: 2720 (Part 5)-1980 [15]. Using the slope of the flow curve, flow index was calculated. At plastic limit, the soil loses its plasticity and passes to a semi solid state.

2.6 Standard Proctor Test

Standard proctor test was used to identify optimum moisture content and maximum dry density of the high swelling clay. The maximum dry unit weight of the soil depends on water content, amount of compaction, type of soil, method of compaction, and use of admixtures. 3 kg of air dried samples passing through 4.75 mm sieve were used for compacting the soil. The test was performed in accordance with IS: 2720 (Part 7)-1980 [16]. After four hours of mixing, the test was done.

2.7 Unconfined Compressive Strength

A cylindrical soil specimen, of size 38 mm diameter and 76 mm height, is loaded axially by a compressive force until failure takes place. Unconfined compression test has zero confining pressure. The axial or vertical compressive stress is the major principal stress, and the other two principal stresses are zero. This test is suitable for saturated clays and was performed as per IS: 2720 (Part 10)-1991 [17].

2.8 Durability Test

Durability test was performed for 12 cycles of alternate wetting and drying for the treated and untreated soil specimens. The soil specimens which were prepared as per optimum moisture content were enclosed in a cling film on the sides. The soil specimens were placed in a moist chamber for a week and were then submerged in water for 300 min. The weight of the soil specimens and the dimensions were measured. The soil specimens were subjected to 70 °C for two days. The measurements were recorded once again. This methodology was repeated for 12 cycles, and measurements were noted for changes in volume and weight. IS: 4332 (Part 4)—1968 procedure was followed in performing this test [18].

2.9 Scanning Electron Microscopy

Specimens from the failure plane of unconfined compressive strength test were studied using the images from scanning electron microscope. Micro structural changes due to potential particle aggregation could be illustrated by SEM.

3 Results and Discussions

3.1 pH and Viscosity

The change in pH of the soil on adding β-glucan was negligible. This indicates that the addition of β-glucan does not alter the concentration of hydrogen ions in the soil. Viscosity of the biopolymer did not increase on addition of water. Even when subjected to heating until 90 °C, viscosity did not increase due to lack of gel formation. Viscosity of the β-glucan solution resulted in 0.0009 Ns/m^2 which was equal to the viscosity of water. At lesser temperatures, a thermo-reversible gel could be formed for a period of time [19].

3.2 Gel Matrix and Dehydration Tests

β-glucan did not produce gel instantaneously. After 15 days, the water containing β-glucan powder was marginally viscous. After 45 days, the β-glucan did not deteriorate when it was exposed to atmosphere. The biopolymer solution was transformed into a solid matrix with increase in height of the polymer matrix with increasing number of days. Even after 180 days, degradation did not take place. This test illustrated the stability of β-glucan when subjected to sunlight.

3.3 Atterberg's Limits

Liquid and plastic limits increased as the percentages of biopolymer content increased. It rose from 240% for the clayey soil to 440% for 3% β-glucan replacement. Plastic limit and plasticity index showed similar increasing trends for increasing percentages. At 0.5 and 3% replacement, percentage increase in liquid limit and plasticity index were 8.33%, 4.54% and 83.33%, 37.88, respectively. Variations in the consistency limits were due to specific surface of soil particles. Being a hydrophilic polymer, β-glucan modifies the double layer of the clayey soil. Results of liquid and plastic limits are presented in Table 1.

Flow index denotes the rate at which the soil loses its shear strength with increasing water content. As the water content increases, the flow curves tend to be steeper and the shear strength of the soil decreases. Toughness index is a measure of shear strength of the soil at plastic limit. Toughness index of the treated soil increased

Table 1 Liquid and plastic limits for different percentages

Percentages	0%	0.5%	1%	1.5%	2%	2.5%	3%
Liquid limit (%)	240	260	344	360	396	430	442
Plastic limit (%)	66	69	72	78	84	89	91
Plasticity index (%)	174	191	272	282	316	341	351
% increase in liquid limit	–	8.33	45.83	50	65	79.17	83.33
% increase in plastic limit	–	4.54	9.09	18.18	27.27	34.85	37.88
Flow index	97.98	105.42	113.57	117.52	129.14	134.17	137.11
Toughness index	1.77	1.81	2.39	2.4	2.45	2.54	2.56

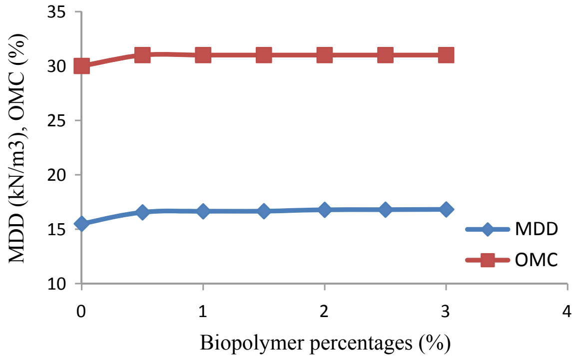

Fig. 1 Optimum moisture content and maximum dry density

as the percentage of β-glucan increased. 40% increase in toughness index was observed from 1% illustrating the fact that soil's resistance against shear stresses were improved.

3.4 Maximum Dry Density and Optimum Moisture Content

Increase in biopolymer content led to increase in optimum moisture content (OMC) and it remained fairly constant on further addition. β-glucan, even though a hydrophilic biopolymer, did not form gel immediately and the increase in water content did not result in higher optimum moisture content. The maximum dry density (MDD) increases by 1 kN/m^3 at 0.5% and it did not rise or decline further for subsequent percentages. Compaction results are presented in Fig. 1.

MDD values of treated soil were higher than the MDD of untreated soil at all biopolymer contents investigated even though the increase was marginal. Fiber formation by β-glucan resists the compactive efforts and deter the particles from moving closer to each other. This limits the increase in dry density though the treated soil fiber becomes stiffer on β-glucan addition. This is evidenced by the increase in toughness index as seen from the Table 1. Increase of optimum water content is a result of the hydrophilic property of the biopolymer used in the study [20–22].

3.5 Unconfined Compressive Strength

Unconfined compressive strength tests were conducted for samples cured on 0, 7, 14, 28, 56 days for 0, 0.5, 1, 1.5, 2, 2.5, and 3% addition of β-glucan to the clayey soil. The results indicate that 2% addition of biopolymer by replacing the soil was the most optimum in increasing the unconfined strength of the soil. 50% increase in

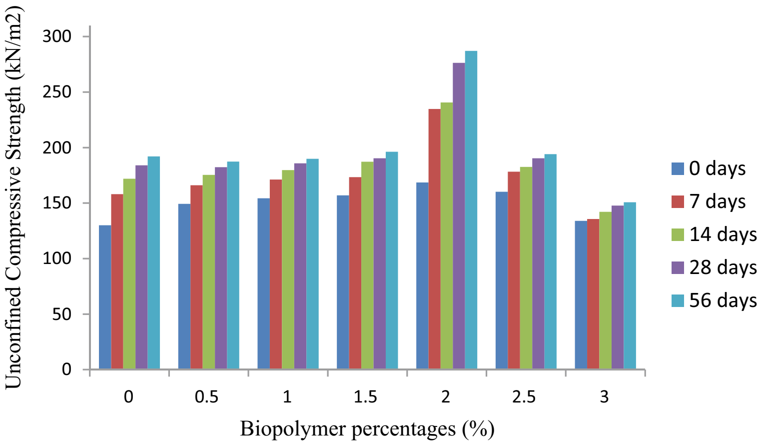

Fig. 2 Unconfined compressive strength for 0–56 days

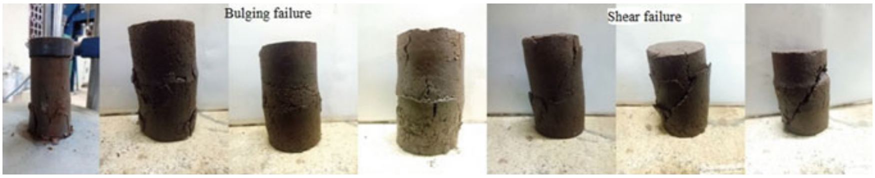

Fig. 3 Unconfined compressive strength failure specimens

strength was observed for the treated soil at 2% on comparison with untreated soil for 28 days. The results are shown in Fig. 2.

After 28 days, the percentage increase in strength was marginal. The increase in unconfined compressive strength was due to the hydrogen bonding of β-glucan with the soil. The development of connection linkages between the soil and biopolymer increased as the number of days increased. Figure 3 shows the failure modes of unconfined compressive strength test.

β-glucans which possess good tensile strength have potential applications in controlling the cracking and tensile failure of many earth structures [22]. For stabilization of slopes, improvement in cohesion increases the shear strength of the soil. 2% of β-glucan shall be best for increasing the strength of the soil which has applications in improving pavement subgrade, stability of slopes, etc.

3.6 Durability Test

There were no signs of appreciable variation in weight and volume of the soil specimens after 12 cycles of alternate wetting and drying. Not more than 3% weight

Fig. 4 Durability test specimens

changes were observed which could be attributed to the stability of the biopolymer and the bonding of soil-biopolymer linkages. The marginal increase in weight may be due to hydration of β-glucan. Durability specimens have been shown in Fig. 4.

3.7 Scanning Electron Microscopy

Figure 5 shows the 28 days microstructure of soil and soil—β-glucan blend.

Scanning electron microscopy images indicate that the soil voids have been filled by the β-glucan and a dense soil-β-glucan network has been observed. The increase in strength with increasing days shall be attributed to the strengthening of hydrogen bond between the soil-β-glucan matrix. In addition to the thread linkages, coating of soil with biopolymer was also observed.

3.8 Cost Benefit Evaluation

California bearing ratio (CBR) of the untreated and treated soil was found to be 6.2% and 12.3%, respectively. The increase in CBR has contributed to reduced surface base course thickness of 40 mm for 20 million standard axle (MSA), as per IRC: 37-2018. For a base course width of 3.75 m and length of 1 km, the decrease in base course thickness of 40 mm contributed to cost reduction of ₹1,557,052. β-glucan's price per kg is ₹275. Subgrade thickness was chosen as 500 mm as per IRC: 37-2018 [23]. Width and length of the subgrade were considered for 5.5 m and 1 km, respectively. Cost benefit ratio was found to be 1.36. Thus, β-glucan treated subgrade is effective and economical in stabilization of expansive soil.

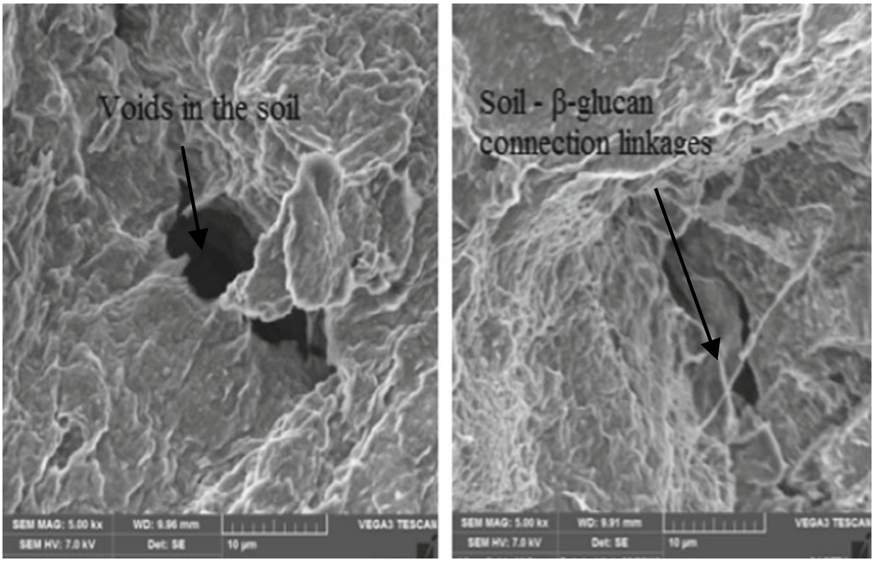

Fig. 5 28 days SEM images of soil and soil—β-glucan (2%)

4 Conclusions

The results of the study points to the following conclusions.

pH and viscosity experienced negligible changes on addition of β-glucan to the soil.

Gel matrix test and dehydration test indicated that the β-glucan gel was promising and stable for 6 months duration.

Increase in optimum moisture content and Atterberg's limits indicate the hydrophilic nature of β-glucan.

β-glucan improves the shear strength of the high swelling clay. 2% addition of β-glucan led to 50% increase in unconfined compressive strength of the soil.

Durability test indicated that, even without cross linking of the biopolymer, the soil-biopolymer mixture did not decompose after 100 days.

SEM results show that the β-glucan—soil bonding has contributed to the strength development.

Cost benefit ratio was found to be 1.36, emphasizing the economical benefits of utilizing β-glucan.

These findings illustrate the benefits of β-glucan in real engineering applications when the focus is on strength improvement of the soil.

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Effect of Various Parameters on Electrokinetic Dewatering of Saturated Clay

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Abstract Electrokinetic dewatering is an effective method to remove water from saturated clay having low hydraulic conductivity. This performs better than other dewatering methods like sand drains, prefabricated vertical drains, vacuum dewatering, etc. In electrokinetic dewatering, low intensity direct current is applied across the soil layer through electrodes, thereby accelerating the flow due to changes in physio-chemical processes. In the present study, a series of laboratory experiments were performed on saturated clay collected from Lunawada region of Gujarat to study the effect of various parameters on electrokinetic dewatering. A water tight wooden box (40 cm × 40 cm × 30 cm) was used to perform the experiments. Hollow circular stainless steel tubes (length = 25 cm and inner diameter = 1.9 cm) were used as electrodes. Perforations with diameter of 5 mm were provided on electrodes at regular intervals to allow and collect water from surroundings during the experiments. Efficiency of electrokinetic dewatering was studied under variations of voltage, electrode spacing and pH. Water collected at cathode was removed manually from top at cathode, and the quantity was measured. The results confirm that dewatering efficiency increases with an increase in voltage as water flows under greater electric gradient at higher potential difference. The decrease in centre to centre spacing between anode and cathode causes overlapping of electric field which enhances dewatering efficiency. Decrease in pH of soil causes soil to be more acidic in nature that results in less dewatering efficiency. Further, cracking patterns near anode for each series of experiments was also studied to enhance the understanding of electrokinetic dewatering. It is confirmed from the experiments that electrokinetic dewatering is greatly influenced by voltage, electrode and pH. Other parameters like electrode pattern, type of soil, type of electrodes, etc., can also be evaluated for better understanding of electrokinetic dewatering.

Keywords Electrokinetic dewatering · Direct current · Stainless steel electrodes · Polarity reversal

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1 Introduction

It is a challenging task for geotechnical engineers to provide safe and economical design of foundations resting on weak/poor soil. Stability and settlement problems for constructions on such weak soils make it mandatory to improve the soil. There are many methods of ground improvement that have been performing good in the field including chemical stabilization, surface and deep soil treatment techniques, preloading with installation of vertical drains, etc. Each method has some limitations due to which it is not applicable for a particular soil. Electrokinetic dewatering is one of the best methods to remove water from soil effectively. It is the application of electric current to the soil through electrodes that are inserted into the saturated soil. Water transfers from anode to cathode via electro-osmosis. The amount of the water drained out from cathode will be equal to the soil consolidation if additional water or solution is not injected into soil through anode. Reuss [13] first reported electro-osmosis. The flow of water through the soil or porous medium is induced by electrical energy that causes due to the thermal or hydraulic gradient [3]. For consolidating different types of soils, electrokinetic stabilization is an innovative method. This also includes soils that possess very high initial water contents in the form of slurry. Although there are various potential applications of electrokinetic stabilization, most of the studies have focused only on soil dewatering, desalination, decontamination and electro-osmotic consolidation. More recently, numerous field applications like electro-osmotic consolidation technique combined with vertical drains and new innovative geosynthetic electrodes found to give best results.

Electrokinetic stabilization is the process of chemical grouting and electro-osmosis. It is most effective method for strengthening of soft clay. Electrokinetic stabilization suits for weak clayey soils which have low hydraulic conductivity and require strengthening [2]. Electrokinetic application is affected by certain factors like soil type, zeta potential, pH, temperature, water content, soil salinity, electrical resistivity and conductivity, type of electrodes. Soil with high organic content exhibits a better response to electro-osmotic consolidation. Higher the negative zeta potential, the water has greater tendency to flow through soil mass. Zeta potential is inversely proportional to the applied voltage. Soil pH greater than or equal to 7 gives better electrokinetic consolidation. When temperature is high, it causes a loss of electric contact between soil and electrodes. If soil salinity increases zeta potential reduces and this can reduce the electro-osmotic flow. Methods to improve efficiency of electrokinetic consolidation are intermittent current, polarity reversal and anode depolarization method [11]. The value of current decreases and pH increases as time passes when dredged sediment was treated by electrokinetics [12]. Laboratory experiments conducted using actual dredged sediments obtained from the Indiana Harbour to determine the extent of electro-osmosis and consolidation using graphite electrode and ionic flocculent. It was observed that with ionic flocculent higher final dewatering is observed.

The traditional methods can be applied on soils which have low to medium plasticity index as given by international standards. But, where the improvement is needed most fine-grained soils are excluded because these pose the greatest problem for the undesirable properties of soil [1]. In dewatering by electrokinetics, shorter pumping interval resulted in more volume of water draining out due to reduction in water head in the drainage pipe. There was 44–46% decrease in the drained volume of water in the test with polarity reversal. The reduction in water content was more in test beds with cement columns compared to those with lime columns. Lime and cement columns had an influence on the undrained strength [9]. Electro-osmosis method combined load with cathode vacuum drainage in the cathode could drain away more water in less time and achieve more settlement, and the ultimate rate of outflow of water is 1.4 times as large as that in traditional electro-osmosis test. It was recorded that after the electro-osmosis method combined load with cathode vacuum drainage in the cathode conducted, the undrained shear strength of soil is higher, besides, the water content is lower and undrained shear strength is about 1.8 times of soil strength consolidated by the traditional electro-osmosis. It was observed that the soil settlement in horizontal direction of electro-osmosis method combined load with cathode vacuum drainage in the cathode is more uniform than that of traditional electro-osmosis and electro-osmosis method combined load [10]. Jeyakanthan et al. [8] presented a design of an electro-osmotic Tri-axial testing apparatus suitable for electro-osmotic treatment of high plasticity black clay and for measuring electro-osmotic permeability and generated pore-water pressure, as well as a testing procedure that accounts for the contribution of electro-chemical changes in the improvement of soil properties. Experimental apparatus was modified from a standard Tri-axial apparatus. The flow rate depends on the coefficient of electro-osmotic permeability which varies with the void ratio and different voltage gradients. The strength improvement due to electro-osmotic treatment is due to electro-osmotic consolidation and electro-chemical changes (ex. cementation, bonding, changes in soil properties due to electro-osmosis). Laboratory study was carried out by Flora et al. [4] on fine-grained dredged sediment (one from the port of Gaeta, and the other one from the Basento River). It was observed that reduction of void ratio and the speeding up of the dewatering process due to the beneficial effect of the electric field is striking for the Basento material, while Gaeta material obtained minor effect. So, the finer and more plastic material is sensitive to the application of the electric field. Further, study is necessary to analyze effect of different potential gradients, orientation of electrodes and variation of pH. To increase the productivity of this method, techniques like intermittent current, polarity reversal and anode depolarization must be checked for its applicability. The aim of this study is to evaluate the use of electrokinetic dewatering in soft clay and examine the effect of various factors like potential difference, spacing, pH and reversal of polarity on electrokinetic dewatering.

Table 1 Physical properties of soil

Property	Value	Unit
Specific gravity (G_S)	2.45	–
Liquid limit (LL)	38	%
Plastic limit (PL)	24	%
Shrinkage limit (SL)	21.94	%
Plasticity index (PI)	14	%
Maximum dry density (MDD)	1.68	g/cm^3
Optimum moisture content (OMC)	19.14	%

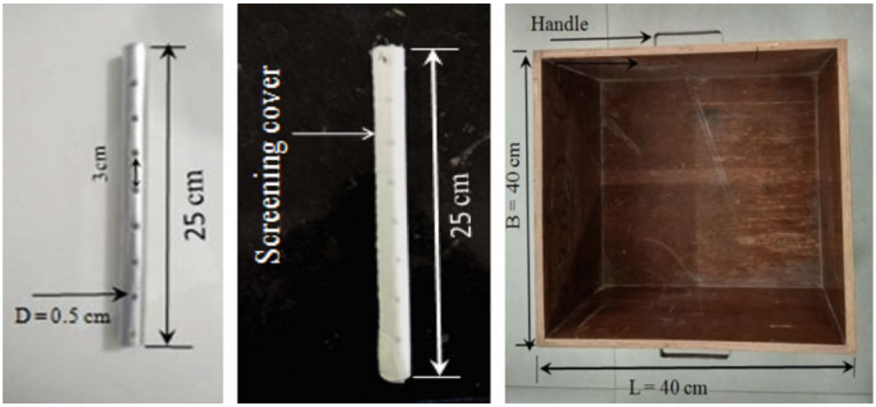
2 Materials Used in the Present Study

2.1 Soil

The material used in the present study is obtained from Lunawada region of Gujarat state. Series of laboratory tests were performed to know the various properties of soil like; specific gravity, Atterberg's limits, particle size distribution, compaction characteristics, etc. (Table 1). Soil used for the experiments was first of all oven dried for 24 h. Lumps of clay were broken into smaller sizes using wooden hammer. Soil passing through 4.75 mm sieve was separated and used to perform experiments. Moisture content of soil for all the experiments was kept equal to its liquid limit by adding distilled water, and it was mixed properly and poured in the box.

2.2 Electrodes

For providing potential difference metal electrodes were used. On the basis of review on the most popular used metal electrode by Malekzadeh et al. [11], stainless steel electrodes were decided (Fig. 1a). Stainless steel electrodes are less susceptible to corrosion compared to copper, aluminium and other steel electrodes. Copper and aluminium electrodes cause the contamination of soil. Compared to titanium electrodes, stainless steel electrodes are less costly. Electrode was prepared by cutting the stainless steel pipe in pieces of length 25 cm. There were 8 electrodes prepared of same length; the diameter of electrode was 1.9 cm (0.75 in.). As collection of water was done from electrode itself, 5 mm holes were drilled on electrodes using drill machine. Total numbers of holes on each electrode were 24; holes were made on each electrode in staggered pattern. To avoid clay particles entering in to the electrode along with water, electrodes were covered with cotton cloth along the length and at bottom (Fig. 1b). Total area of electrode was 70.88 cm^2, and effective area of each electrode was 66.17 cm^2.



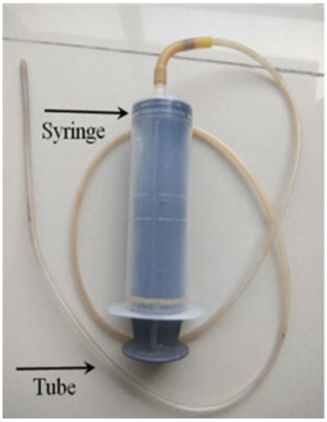
(a) Electrode with perforations (b) Electrode with filter (c) Wooden box

Fig. 1 Details of test box and electrode preparation

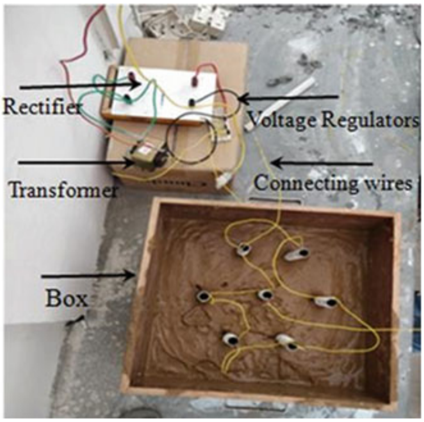
3 Procedure to Perform Electrokinetic Dewatering

The setup for electrokinetic dewatering and dimension of box and electrodes was designed. It requires DC supply for performing electrokinetic dewatering. The setup was prepared for performing the electrokinetic dewatering with dimensions of box 40 cm × 40 cm × 30 cm (Fig. 1c) considering reference of Fourie and Jones [5] who performed electro-osmotic dewatering on clay. The box was prepared using wooden sheet with two metal handles for better handling. For providing water tightness to the box, water proofing gel was applied on inner surface area of box. The water tightness was checked by filling the water in it for 24 h to observe if any leakage is present.

For providing various DC voltages various electric supplies were needed. A transformer (12 V, 3 A) was used to step down the AC voltage. Rectifier was used to convert the AC voltage to DC voltage. Breadboard and voltage regulators 7809, 7812 and 7806 were used to regulate the voltage. A syringe of 20 ml connected with tube was used to collect water manually from electrodes. Length of tube was sufficient to reach the bottom of electrodes (Fig. 2a). After making all the arrangements for electrokinetic dewatering, the prepared soil was poured in the box. Configuration of electrodes was decided to be hexagonal as reported by Glendinning et al. [6]. Total seven electrodes were used in this study, out of which 6 were anodes and 1 was cathode (Fig. 2b). The water collected at cathode is pumped out manually at each hour by syringe and tube for 24 h. As it was not possible to collect water continuously for 24 h manually, it was collected for first six hours and last 6 h. The experiments were performed to study four factors; (i) effect of potential differences, (ii) effect of spacing between electrodes, (iii) effect of polarity reversal and (iv) effect of different pH of soil. Summary of the test series is provided in Table 2.



(a) Syringe with capillary to draw water



(b) Experimental setup

Fig. 2 Experimental setup for electrokinetic dewatering

Table 2 Summary of series of experiments

Series	Parametric study	Group of tests with test legends	Parameters varied for the tests	Parameters maintained constant
A	Effect of voltage	EKD-1, EKD-2, EKD-3 and EKD-4	Voltage = 1.2, 1.5, 1.7 and 10 V	c/c spacing = 12 cm, anodes—6, cathode—1
B	Effect of spacing of electrodes	EKD-1, EKD-5 and EKD-6	c/c spacing = 10, 12 and 14 cm	$V = 1.2$ V and anodes—6, cathode—1
C	Effect of polarity reversal	EKD-4 and EKD-7	Anodes—1 and 6 Cathodes—1 and 6	$V = 10$ V and c/c spacing = 12 cm
D	Effect of pH	EKD-4, EKD-8 and EKD-9	pH = 8.5 and 8.0	$V = 10$ V, c/c spacing = 12 cm, anodes—6, cathode—1

4 Results and Discussions

4.1 Effect of Potential Difference (Series—A)

Series of experiments were performed at different voltages. Based on availability of voltage regulators, it was decided to perform experiments at voltages 1.2, 1.5, 1.7 and 10. Figure 3 shows variation in volume of water collected at various time intervals for different potential differences. It was observed that with the increasing potential

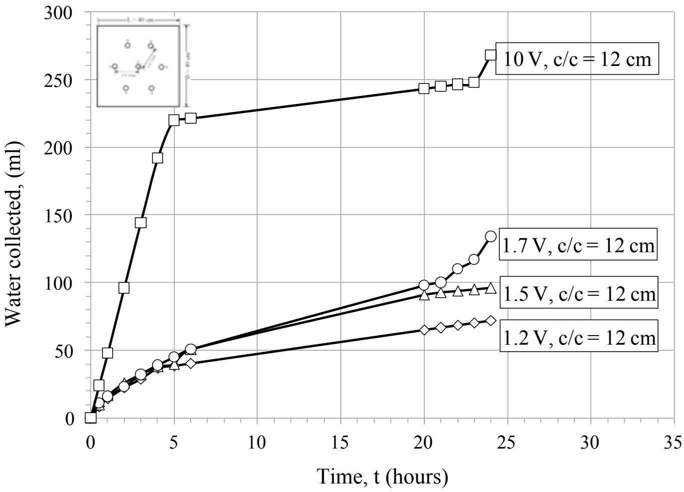
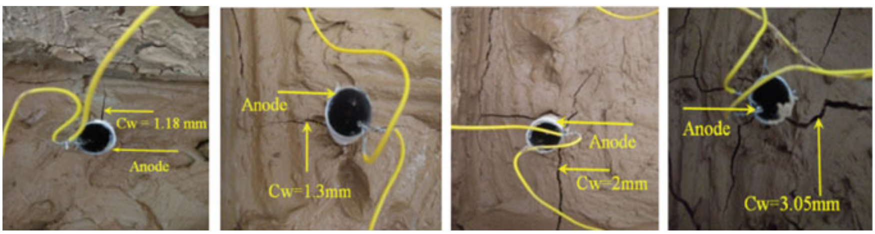


Fig. 3 Water collected at various time intervals

difference amount of water collected increases. Volume of water collected at cathode increases with increase in potential difference because at higher potential difference water flows under high electric gradient.

The width and length of cracks were determined by using Image J software. In this, a known distance of electrode inner diameter (19 mm) is marked by a straight line and the scale is set in 'Image J' software [7]. This procedure is repeated for each and individual figure. Once a scale is set in 'Image J' software, then unknown distances can be determined easily. Figure 4 depicts the cracks observed near anode after 24 h of dewatering. For 1.2 voltage, crack width of 1.18 mm was observed (Fig. 4a), and for 1.5 voltage, it was 1.3 mm (Fig. 4b). However, crack width of 2 mm was observed for 1.7 voltage (Fig. 4c) and 3.05 mm for 10 voltage (Fig. 4d). It can be clearly seen that for higher voltage difference, the length as well as width of the crack were on the higher side as compared to low voltage difference.



(a) V = 1.2V (b) V = 1.5V (c) V = 1.7V (d) V = 10V

Fig. 4 Cracks observed after 24 h at anode for various voltages

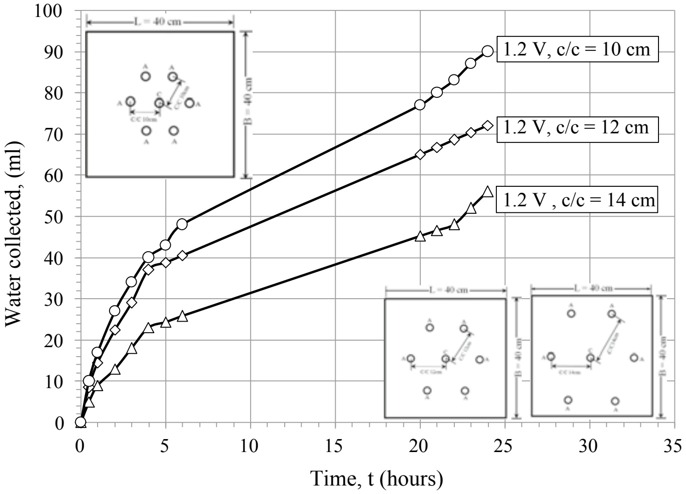


Fig. 5 Water collected at various time intervals

4.2 *Effect of Various Spacing of Electrodes (Series—B)*

Experiments were performed by varying centre to centre distance between anode and cathodes by 100 mm, 120 mm and 140 mm, respectively. A hexagonal configuration, 6 anodes and 1 cathode were used. Potential difference of 1.2 V was kept constant for all the experiments in series B. Volume of water collected at cathode with various time intervals for different centre to centre spacing of anode and cathodes is shown in Fig. 5. It was observed that with the decreasing centre to centre distance between anode and cathode, collection of water increases because electric field of electrodes overlaps more and thus resulting in more draining of water.

Cracks were observed at anodes after 24 h electrokinetic dewatering for different spacing of electrodes (Fig. 6). For c/c spacing of electrodes equal to 140 mm, maximum crack width of 1.05 mm was observed (Fig. 6a) and when c/c spacing of electrodes equal to 120 mm was maintained, maximum crack width of 1.15 mm was noticed (Fig. 6b). Further, maximum crack width of 2.45 mm was found out for c/c spacing of electrodes equal to 100 mm (Fig. 6c). The width of crack was observed to be high for shorter spacing between anode and cathode. It is due do quick and maximum removal of water from anode to cathode.

4.3 *Effect of Polarity Reversal (Series—C)*

Experiments were performed by changing the number of anodes and cathodes. First experiment was performed using 6 anodes and 1 cathode. Cathode was kept at the

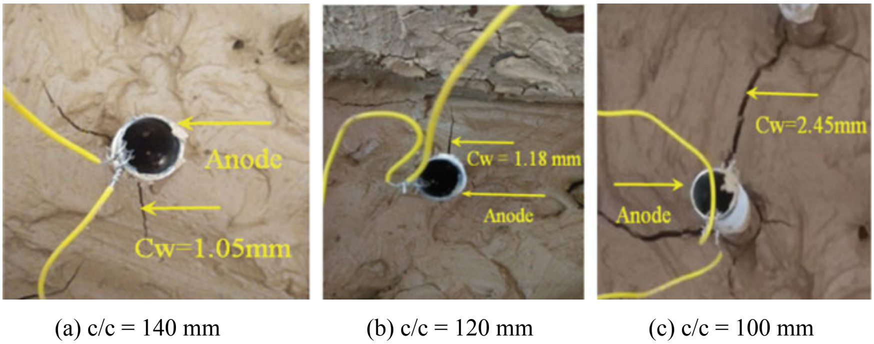


Fig. 6 Cracks observed after 24 h at anode for various spacing pattern

centre of box, and anodes were placed at equidistant from cathode. Anodes were placed in hexagonal pattern and at distance 120 mm from cathode. Second experiment was performed using 6 cathodes and 1 anode. Anode was kept in centre of box, and cathode was placed at equidistant of 120 mm. The voltage for both the experiments was kept constant as 10 V.

Volume of water collected at different time intervals for two configurations; (i) Case 1: 6 cathodes and 1 anode, (ii) Case 2: 1 cathode and 6 anodes are shown in Fig. 7. It can be clearly seen that for case 1 (6 cathodes and 1 anode), water collected is less as compared to case 2 (1 cathode and 6 anodes). It is because water moves from anode to cathode with high gradient when numbers of anodes are more. Water molecules will get highly charged due to overlapping of electric field.

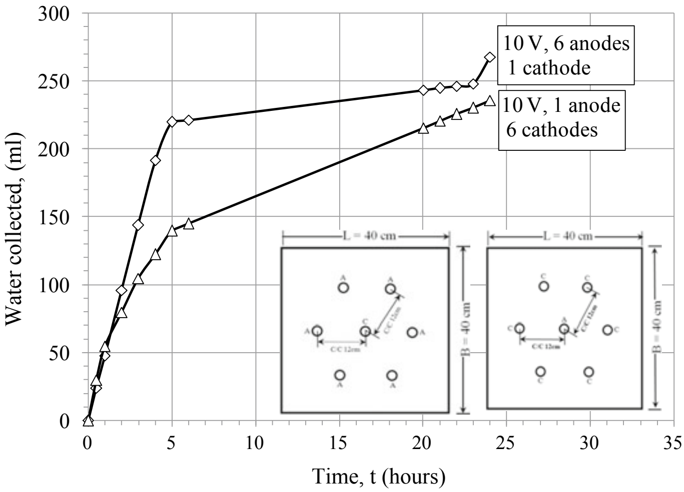


Fig. 7 Water collected at various time intervals

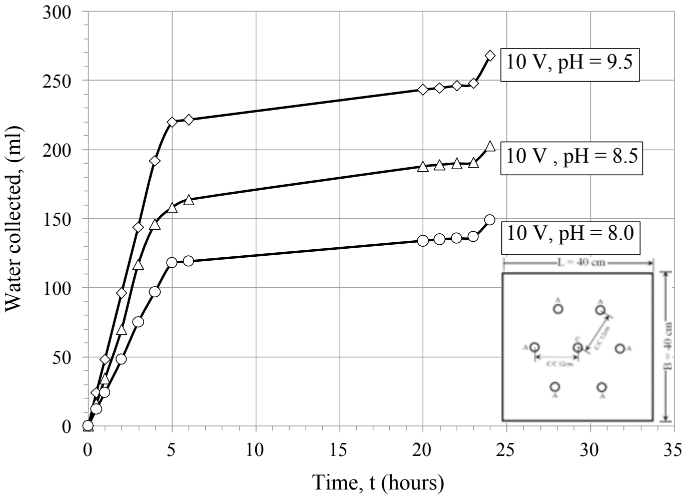
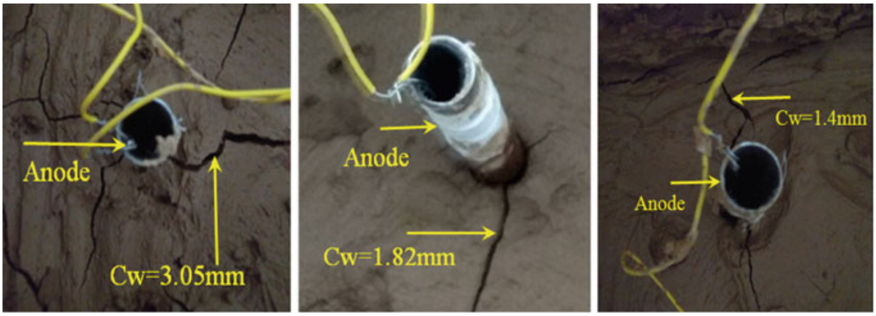


Fig. 8 Water collected at various time intervals

4.4 Effect of Various pH of Soil (Series—D)

The electrokinetic dewatering tests were performed on different value of pH of soil like 9.5, 8.5 and 8.0. Soil pH was changed by mixing sulphuric acid. First a trial was made for changing the pH of soil on 100 g, and then based on calculations, sulphuric acid was added to the soil for changing its pH as per guidelines provided by IS2720 (P-26) 1987. Soil pH was measured using pH metre. For measuring the pH of soil, 30 g of soil was taken and mixed with 75 ml of distilled water using glass rod and then pH was measured. As can be seen from Fig. 8, total volume of water collected at various time intervals decrease with increase in pH of the soil. This confirms that the value of pH of soil should be kept high to remove water from soil at a higher rate.

Figure 9 shows images obtained at anode after 24 h of electrokinetic dewatering for different pH values of soil. Maximum crack width of 3.05 mm was observed at anode when pH of soil was 9.5 (Fig. 9a) and 1.82 mm for soil with pH of 8.5 (Fig. 9b). However, when pH of soil was maintained to be 8, maximum crack width was obtained to be 1.4 mm only (Fig. 9c). The width of crack was observed maximum when pH of soil was maintained on the higher side. Rate of dewatering increases with increase in pH of soil.



(a) pH = 9.5 (b) pH = 8.5 (c) pH = 8.0

Fig. 9 Cracks observed after 24 h at anode for various pH values of soil

5 Conclusions

Different series of experiments were performed to study the influence of various parameters on the performance electrokinetic dewatering. Based on the results obtained from experiments, the following conclusions can be drawn.

1. Amount of water collected at cathode increases with increase in voltage difference for constant spacing and configuration of electrodes. It is because at higher potential difference, water flows under greater electric gradient. Width of cracks developed at anode increases with increase in voltage difference because maximum quantity of water is moving from anode to cathode when voltage difference is high.
2. It was found that amount of water collected at anode decreases with decrease in pH of soil, for constant spacing of electrodes, configuration of electrodes and voltage. Width of cracks developed at anode decreases with decrease in pH of soil.
3. By varying spacing of electrodes, it was found that amount of water collected at anode increases with the decrease in spacing. It is due to overlapping of electric field of two electrodes which results in more draining of water. The width of cracks increases with decrease in spacing that causes more draining of water from anode to cathode.
4. For constant voltage and spacing of electrodes, polarity reversal gives reduction in water collection. It is because water moves from anode to cathode, and with 6 anodes, more water will move towards the cathode. In second case, water from 1 anode moves towards 6 cathodes so water collection is less. Width of cracks developed at anode decreases after reversing the polarity.

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Plasticity and Swelling Characteristics of Geopolymer Treated Expansive Soil

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Abstract Expansive soil causes extensive damage to geotechnical structures owing to its high volume instability. Cement and lime are the most commonly used material to improve these soils by reducing its plasticity, swelling characteristics and increasing strength. However, the production process of these traditional stabilisers is energy intensive and it also serves as a major source of green-house gas emission leading to severe problems like global warming. Geopolymer is a new generation alternative binding material for conventional cement. This is primarily produced from industrial wastes like slag or fly-ash which are rich in alumino-silicates. When activated with alkalis, these products form geopolymers, which provides high strength to soil and have low cost, low energy consumption and is eco-friendly. This study explores the efficiencies of ground granulated blast furnace slag (GGBS)-based geopolymer binder in improving the properties of expansive soil in comparison to cement and lime. In this study, the expansive soil is mixed with 0, 5, 10, 15 and 20% of GGBS and activated with sodium hydroxide solutions of 0.5, 1, 2 and 4 M concentrations. However, cement and lime are mixed with the soil in the proportions of 1, 2, 4, 8, 12 and 15% by weight of the soil. The consistency limits and swelling characteristics of geopolymer, lime and cement treated soils are evaluated at 0, 3, 7 and 30 days of curing. It is found that the plasticity characteristics are improved and swelling and shrinkage of the expansive soil is greatly reduced with increasing concentration of these admixtures. Curing period also influences these properties. It is also observed that the performance of geopolymer is comparable to that of cement and lime. So, geopolymer can be effectively used as an alternative stabilising agent to modify the plasticity and swelling properties of expansive soil.

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Keywords Expansive soil · Geopolymer · Consistency limits · Plasticity characteristics

1 Introduction

Expansive soils are considered as problematic soils because of their highly unpredictable performance in the presence of moisture. A marginal change in moisture content in this soil reduces shear strength which results in high swelling, shrinkage, settlement, and consolidation. These soils are mostly found in arid and semi-arid regions of the world and the presence of montmorillonitic clay mineral imparts swell-shrink potential to these soils. Improvement of expansive soils by treating them with lime and cement are the established methods which are used widely around the world. However, the production process of these traditional stabilisers is energy intensive and it also serves as a major source of carbon dioxide emission leading to serious environmental problems like global warming [1].

Geopolymer is a new generation alternative binding material for conventional cement. The intense amount of work on geopolymeric binders derived from these industrial by-products have proved its effectiveness having similar strength and durability properties as that of conventional concrete. This alkali source provider, in the presence of alkaline medium forms geopolymerization products which is shown in Fig. 1, that have comparable or even better characteristics than calcium-silicate-hydrate products of conventional concrete. The concept of geopolymer was first proposed by Davidovits [2], it was found that kaolinite could be polymerized by alkalis, producing a concrete like material [2]. The formation of geopolymer gel from the geopolymerization products improved the strength capabilities of soil. Marginal lateritic soil could be stabilised by high calcium FA-based geopolymer and used as an eco-friendly pavement material, which would furthermore decrease the need for

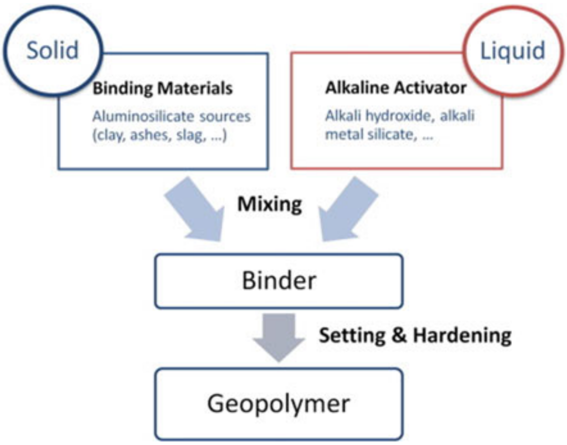
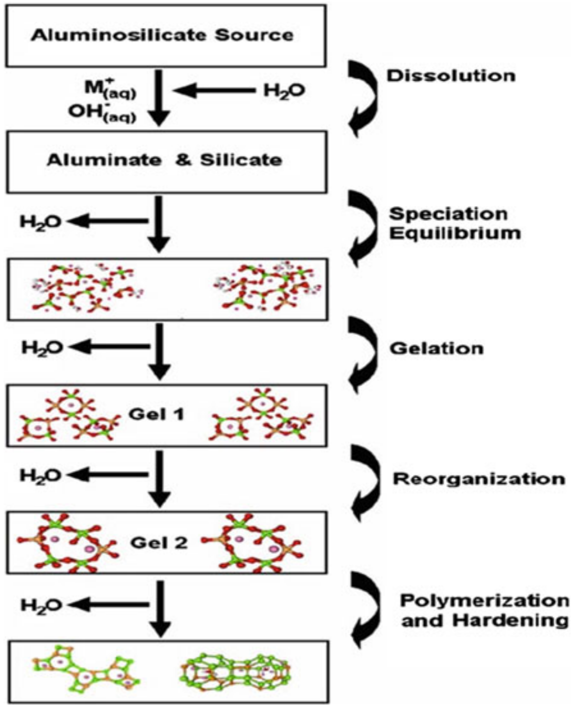


Fig. 1 Geopolymer components [4]

Fig. 2 Conceptual model of geopolymerization



high carbon Portland cement. The compressive strength of the geopolymer treated soil improves by increasing the molarity of alkali activator [3]. The chemical process to produce geopolymers involve three steps: (1) Dissolution of raw materials in alkaline solution to form Si and Al gel on the material's surface, (2) Reorientation, which is condensation of precursor ions into oligomers and (3) Polycondensation to form networked polymeric oxide structures as depicted in Fig. 2. In the present study, an attempt has been made to study various mix parameters which control the stabilising process in the soil geopolymer.

2 Materials and Methodology

Usually geopolymer is derived from alkali activation products of alumino silicate source materials. Here in the present study ground granulated blast furnace slag, an industrial by-product is used. Blast furnace slag is collected from Rourkela steel plant which is rich in alumino silicate and activated by sodium hydroxide solution. The solutions of 0.5, 1, 2 and 4 M are prepared with distilled water 24 h prior to geopolymerization to get homogeneous solutions free from precipitates. The blast furnace slag is mixed in the proportions of 5, 10, 15 and 20% to that with locally available expansive soil. To make the mixture stable, the mixtures are then added with sodium hydroxide solution of each concentration. For a comparative study, the soil is also mixed with dry weights of 1, 2, 4, 8, 12 and 15% lime and cement separately. The mixed samples are left for curing at constant temperature in a sealed container for conducting the experimentations at 0, 3, 7 and 30 days. After each curing period, the test samples are again mixed thoroughly and grounded by a wooden hammer and made to pass through 425μ IS sieve. All the tests are done as per the IS codes.

3 Results and Discussions

3.1 Liquid Limit

The variation of liquid limit with slag content and curing period for different soil-slag mixes are depicted in Figs. 3, 4 and 5 and with lime and cement content are depicted in Figs. 6 and 7. It is observed that for all soil mixes, initially, there is a decrease in liquid limit with increase in additive content. In the alkaline environment, formation of calcium silicate hydrate (C-S-H) gel occurs, which consists of solid products of hydration and water that is held physically or adsorbed on surface of the hydrates. In addition to gel, water exists which is combined chemically or physically with the hydrates. This large amount of water significantly marginalises the influence of the double layer reduction by inducing decrease in water content and thereby the liquid limit.

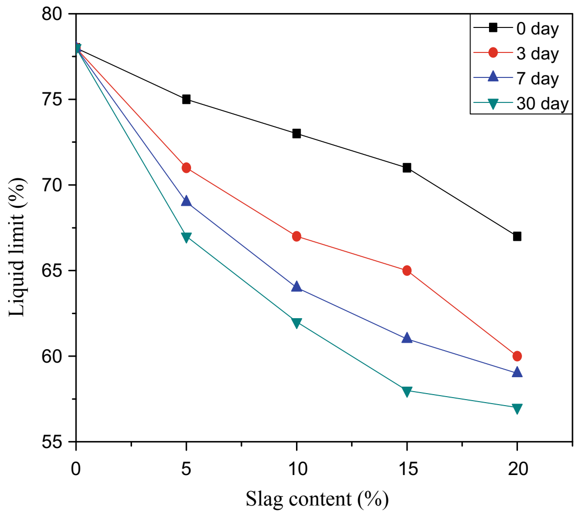


Fig. 3 Variations of liquid limit for soil-slag mix with slag content

Fig. 4 Variations of liquid limit for soil-slag mix with slag content treated with 0.5 M NaOH solution

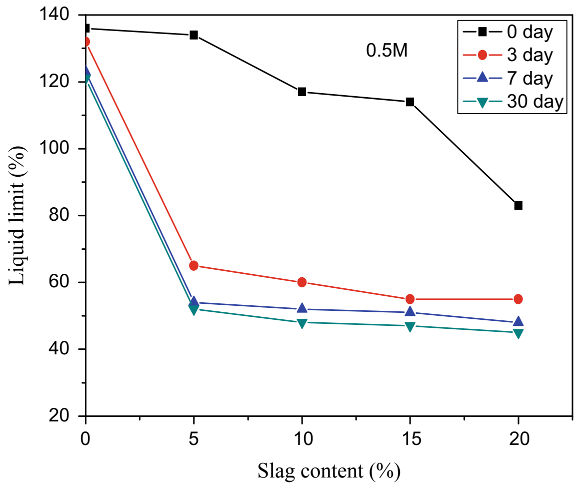
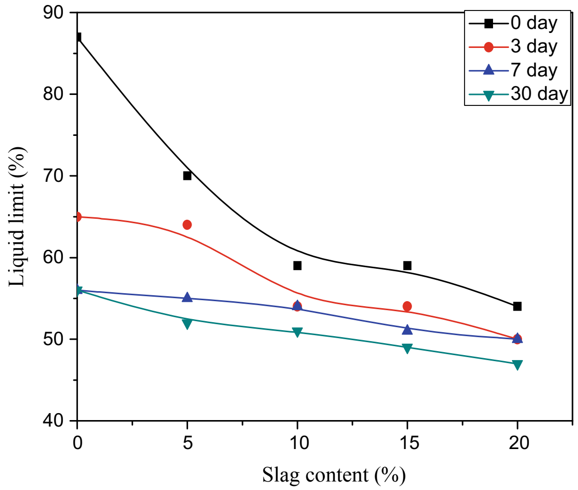


Fig. 5 Variations of liquid limit for soil-slag mix with slag content treated with 2 M NaOH solution



3.2 *Plasticity Index*

The plastic limit is a measure of soil cohesion against cracking while beading the soil. The shear resistance between the particles of the soil should be sufficiently low to be able to slide partially over each other at ease. At the same time, the resistance of the inter-particle shear should be sufficiently high to hold the soil mass in the re-formed place. The plastic limit is, therefore, a measure of the soil water content when approaching a certain resistance to shear or shear strength. Figure 8 shows the variation of plastic index of soil-slag mix with slag content. Figures 9 and 10 shows the variation of plastic index of soil-slag mix treated with 0.5 M NaOH and

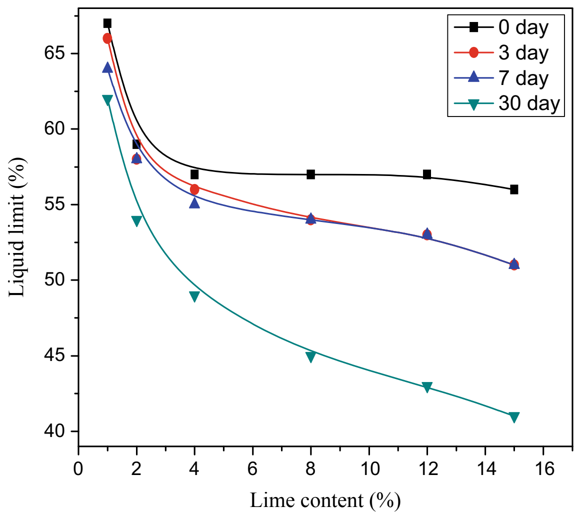


Fig. 6 Variations of liquid limit of soil-lime mix with lime content

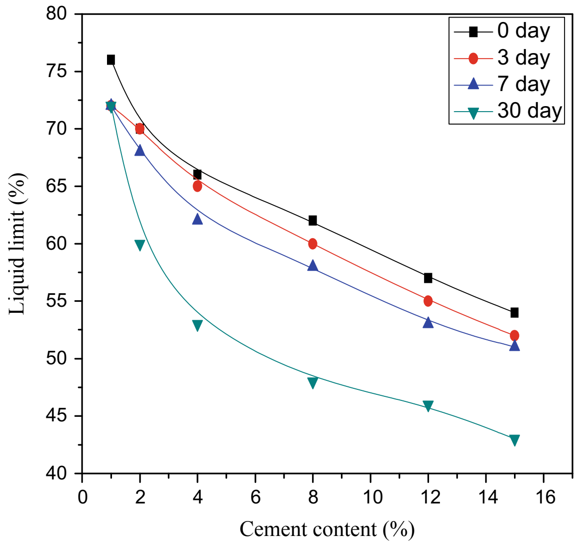


Fig. 7 Variations of liquid limit of soil-cement mixes with cement content

2 M NaOH solution. Figure 11 shows the variation of plastic index of soil-cement mixes with cement content. The thickness of the diffuse double layer decreases with the addition of lime, cement or geopolymer, which increases the concentration of the load and thus the viscosity of the pore fluid. As a result, the inter-particle shear resistance increases, resulting in a sharp increase in the plastic limit. As the liquid limit decreases and plastic limit increases, the plasticity index also reduces with additive content and with curing period. With the addition of lime more than 4%

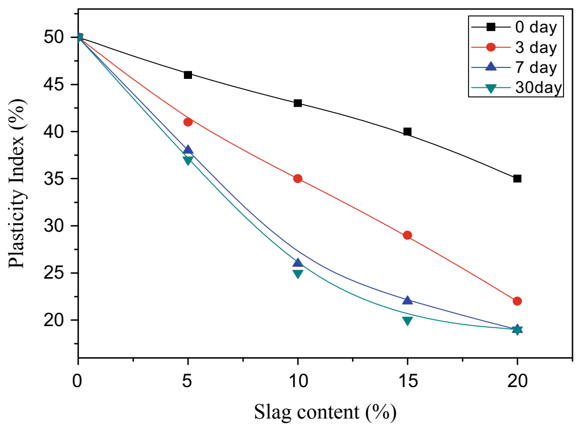


Fig. 8 Variations of plasticity index of soil-slag mix with slag content

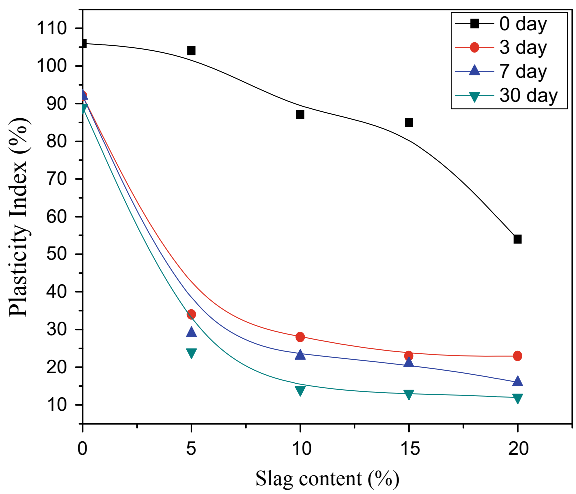


Fig. 9 Variations of plasticity index of soil-slag mix treated with 0.5 M NaOH solution

and for the concentration of NaOH more than 2 M, the soil is modified into crumbly nature as silty soil and becomes non-plastic after 3 days of curing.

3.3 Shrinkage Limit

The increase in shrinkage limit as shown in Figs. 12, 13, 14, 15 and 16 with the additive content (slag, lime, cement) is attributed to the aggregation of particles by the amendment of additive. The soil being highly plastic was initially in a dispersed state. With the addition of lime, the diffused double layer thickness decreases with

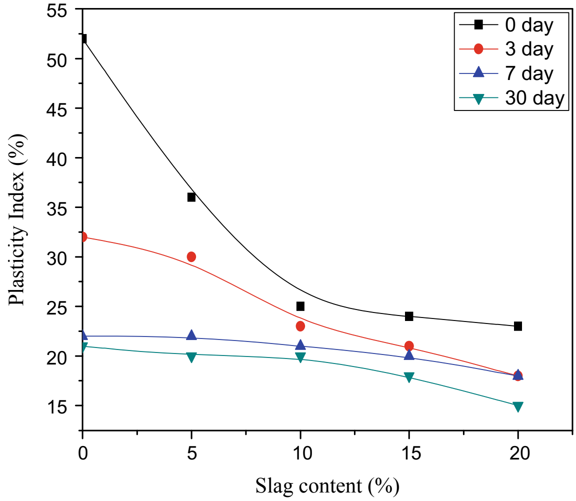


Fig. 10 Variations of plasticity index of soil-slag mix treated with 2 M NaOH solution

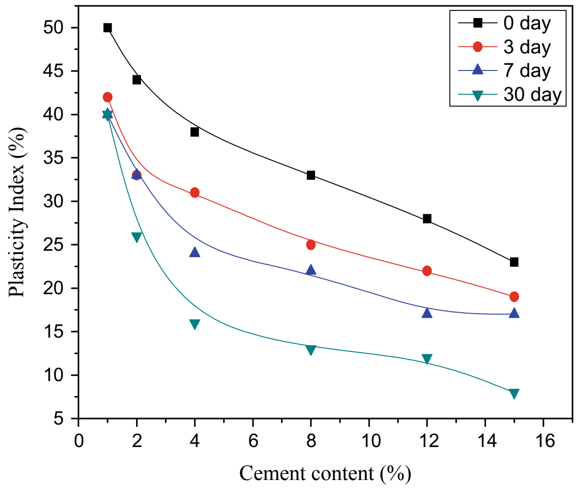


Fig. 11 Variations of plasticity index of soil-cement mixes with cement content

increased electrolyte concentration and thus the repulsion between the clay particles decreases. As a result, the soil particles are coming closer to form aggregated clusters. These aggregated clusters offer increased resistance to capillary suction resulting in volumetric shrinkage resulting in increased shrinkage void ratio and hence water content (i.e. shrinkage limit). With the increase in the curing period, the shrinkage limit has increased further. This is because with prolonged curing aggregation increases which mobilises increased resistance against shrinkage leading to enhanced shrinkage limit.

Fig. 12 Variations of shrinkage limit for soil-slag mix with slag content

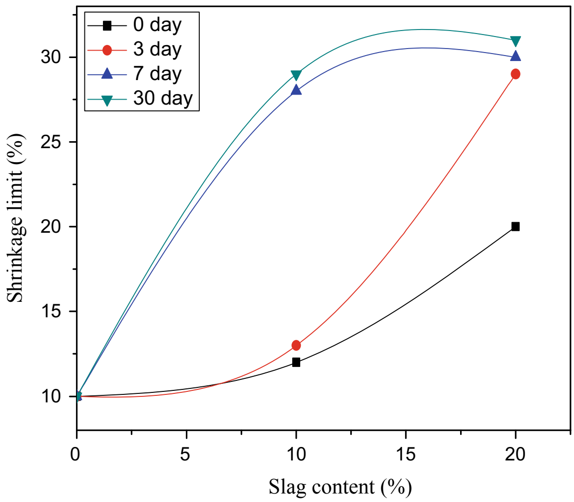
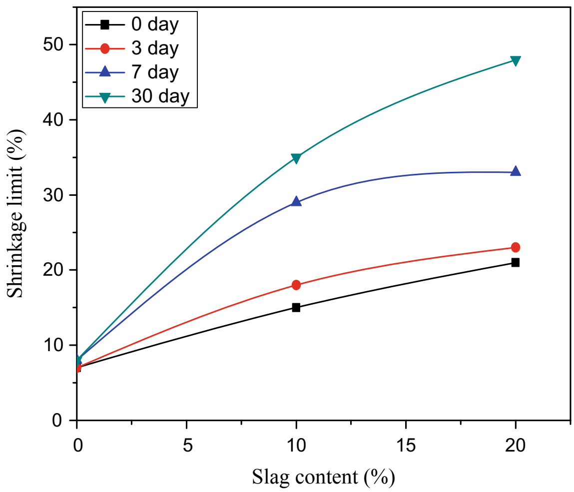


Fig. 13 Variations of shrinkage limit for soil-slag mix with slag content treated with 0.5 M NaOH solution



3.4 Linear Shrinkage and Differential Free Swell

The thickness of the diffused double layer decreases with the addition of lime, cement or geopolymer, which increases the concentration of charge and thus the viscosity of the pore fluid. As a result, the inter-particle shear resistance increases, resulting in a sharp increase in the linear shrinkage index as shown in Figs. 17, 18, 19, 20 and 21 and free swell index as shown in Figs. 22, 23, 24, 25 and 26.

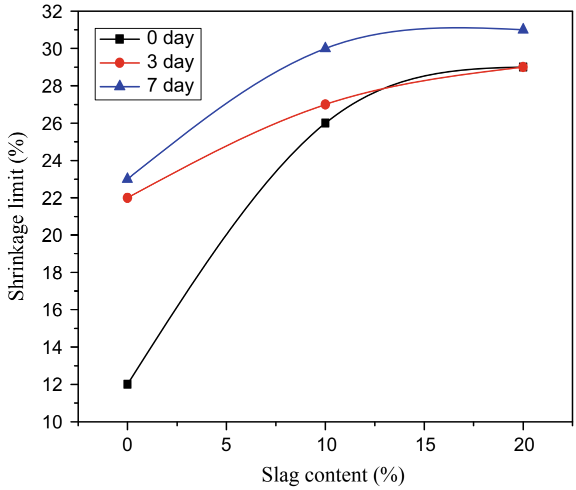


Fig. 14 Variations of shrinkage limit for soil-slag mix with slag content treated with 2 M NaOH solution

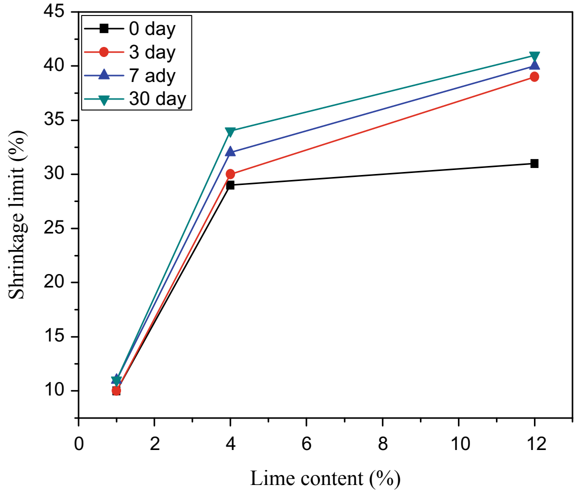


Fig. 15 Variations of shrinkage limit of soil-lime mixes with lime content

3.5 Scanning Electron Microscopy

With an objective of gaining a textural insight of the stabilised soil, microscopy analysis and spectral analysis of the specimen are conducted in scanning electron microscopy. The as-received stabilised sample exemplifies extensive amorphization attributed to loss of hydration product during the initial stages of curing. However, the specimen predominantly shows flocculation of the specimen due to the formation of gel like structure after being cured for 30 days as envisaged by the flaky structure in Fig. 27. Furthermore, the chemical composition of the specimen is analysed via energy dispersive analysis which exemplifies that silica is the major constituent in

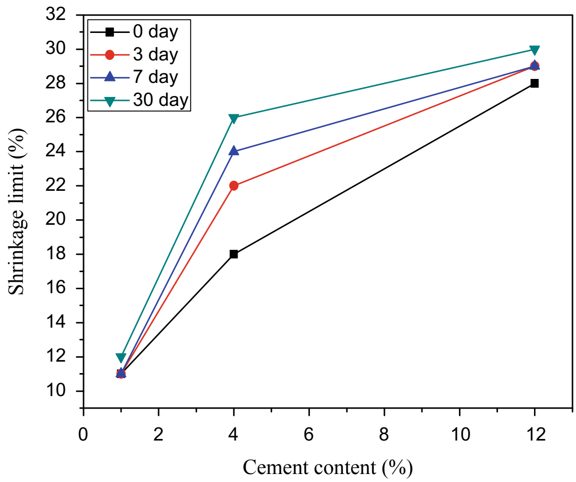


Fig. 16 Variations of shrinkage limit of soil-cement mixes with cement content

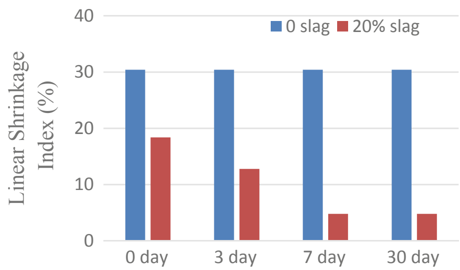


Fig. 17 Variation of linear shrinkage index for soil-slag mix for different curing periods

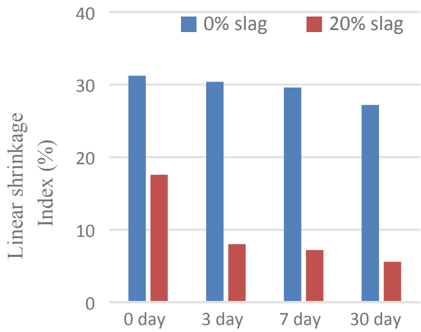


Fig. 18 Linear shrinkage index variation for soil-slag mix treated with 0.5 M NaOH solution for different curing periods

the given stabilised soil specimen. Following silica, alumina predominates the soil composition with traces of calcium oxide, iron oxide and magnesium oxide.

Fig. 19 Linear shrinkage index variation for soil-slag mix treated with 2 M NaOH solution for different curing periods

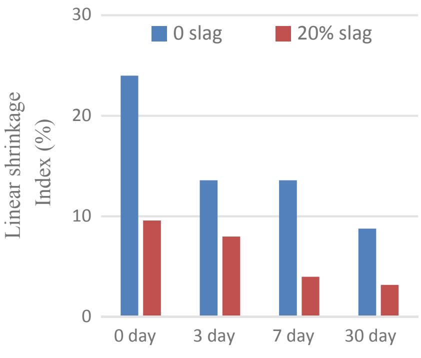


Fig. 20 Linear shrinkage index variation for soil treated with lime for different curing periods

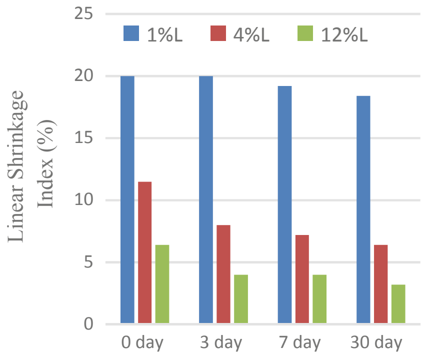
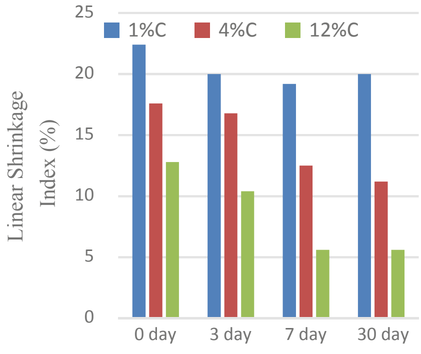


Fig. 21 Linear shrinkage index variation for soil treated with cement for different curing periods



4 Conclusions

This study made an attempt to improve the plasticity and swelling characteristics of expansive soil with the objective of preparing a suitable binder by utilising industrial by-products like blast furnace slag which proved as better additives than conventional stabilisers like lime and cement. Based on the experimental results, the following

Fig. 22 Variation of free swell index for soil-slag mix for different curing periods

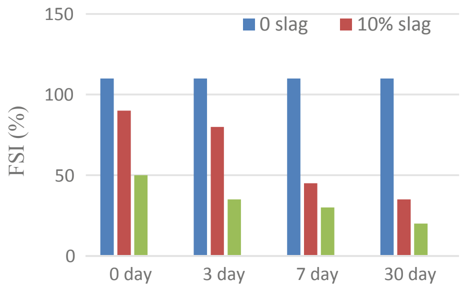


Fig. 23 Free swell index variation for soil-slag mix treated with 0.5 M NaOH solution for different curing periods

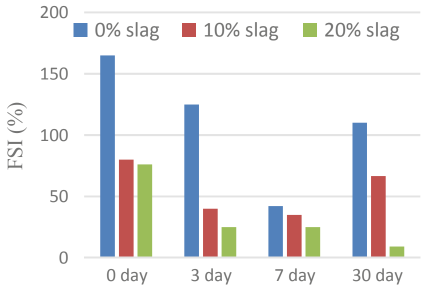


Fig. 24 Free swell index variation for soil-slag mix treated with 2 M NaOH solution for different curing periods

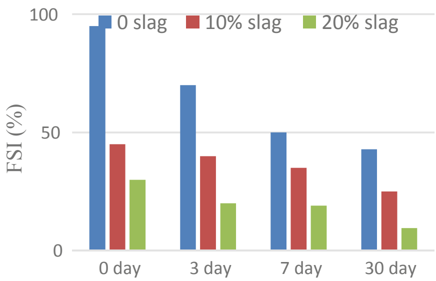
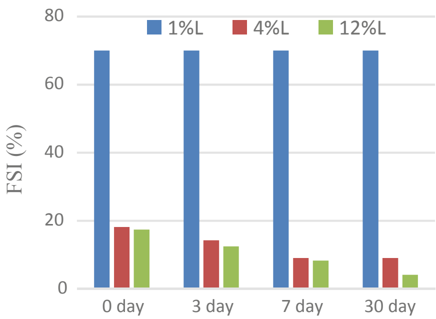


Fig. 25 Free swell index variation for soil treated with lime for different curing periods



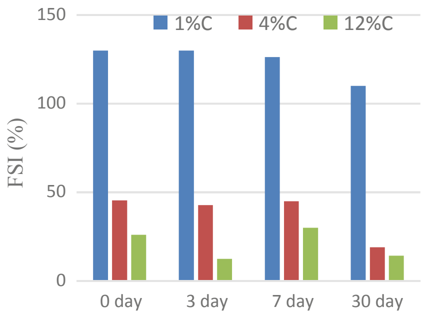


Fig. 26 Free swell index variation for soil treated with cement for different curing periods

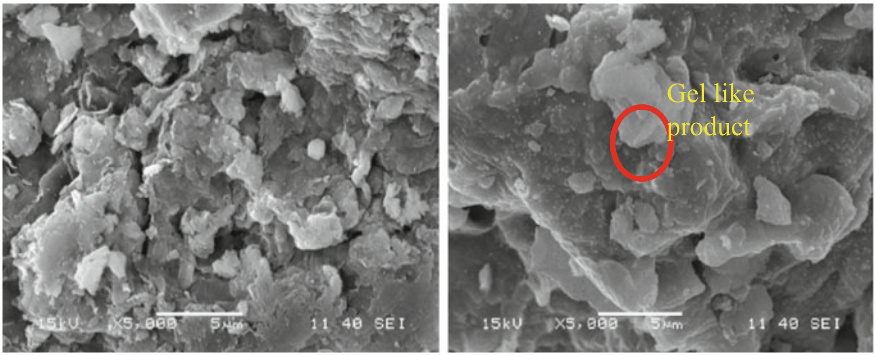


Fig. 27 SEM image of soil-slag mix treated with 2 M NaOH solution after (1) 0 day curing, (2) 30 days curing

conclusions can be drawn. Liquid limit for slag treated soil was reduced from 78 to 58% following 30 days of curing, however, when it is alkali activated with 4 M of NaOH solution, then this reduction is critical from 78 to 40%. Similarly, plasticity index also reduced from 50 to 23% for slag treated soil and with alkali activation, the soil becomes non-plastic for 4 M NaOH solution after 3 days of curing. The soil became crumbly like silty soils. Shrinkage is reduced largely by adding geopolymers to expansive soil. Free swell index values are reduced from 110% for natural soils to almost 0% for geopolymer treated soils. Similarly, linear shrinkage index values were reduced from 30.4% to almost about 0% (0.79%). The scanning electron microscopy also confirmed the formation of a gel like product because of synthesis of geopolymer leading to flocculation of clayey particles after 30 days of curing.

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Improvement in Soil Subgrade Using Natural Fibre (Kenaf and Coir Fibre)

Shalinee Shukla, Ayush Mittal, and Sunit Kumar

Abstract India has one of the largest road networks in the world. Around 20% land area of India is covered with soils having low California bearing ratio (CBR) and shear strength values. Pavements laid over such type of soil lead to continuous deformation as a result of which cracks are developed. It increases the maintenance cost and disrupts traffic services. From past decades, various researchers have tried to overcome such type of problems either by using traditional techniques like lime, cement, mechanical stabilization or by using modern techniques like geosynthetics or natural fibres, etc. The present paper deals with the use of natural fibres (kenaf and coir) as a reinforcing material to assess the improvement in strength characteristics of unreinforced soil. The soaked CBR and maximum dry density (MDD) of virgin soil were 2.11% and 1.823 gm/cc, respectively. Kenaf and coir fibre were cut in length of 15 mm and randomly mixed with the unreinforced soil in different percentages (i.e., 0.25, 0.50, 0.75, 1.0, 1.25 and 1.5%) by weight of dry soil. The results obtained indicate that MDD value decreases with increase in fibre content with maximum value of 2.01 g/cc at 0.25% fibre content. The soaked CBR value increases up to 1% fibre content and then decreases. Increase in CBR value indicates that the bearing capacity of mixture (soil + fibre) increased due to the three-dimensional bonding between them. It can be concluded that natural fibres like coir and kenaf can be effectively used as soil reinforcement material.

Keywords Poor subgrade soil · CBR · Fibre · Reinforcement · Compaction

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1 Introduction

The economic and social development of any country is mainly dependent on transportation infrastructure. Its importance further increases if the economy of a country is based on agriculture. In India, 90% of passenger traffic and 65% of freight traffic passes through roads. The performance of pavement depends mainly on properties of soil subgrade as it serves its foundation [1, 2]. Due to scarcity of good quality road aggregates, limited availability of land and to meet the demand of growing population compel the engineers to make use of weak or soft soils which were earlier left out. This brings the role of soil reinforcement (in form of strips, bars, sheets, rods and fibres) technique into picture since removal and replacement of soil will lead to heavy economic burden. Several investigations have been reported on the use of synthetic fibres for strength improvement of granular soils [3–8]. Various studies have studied the effect of use of natural fibres on strength improvement of coarse grained soils [9–12]. While only limited studies report on the use of natural fibres on fine-grained soils [13, 14].

In the present work, kenaf and coir fibres of length 15 mm are randomly mixed with soil in different percentage by dry weight of soil. A series of heavy compaction and soaked CBR tests were performed. Also the percentage change in pavement thickness was examined on the basis of soaked CBR values.

2 Materials Used

2.1 Soil

The soil samples were obtained from Chakghat, Rewa district, Madhya Pradesh. The soil is fine grained soil, and it was collected by digging trials pits at a depth of 1 m below the ground level as the top soil is likely to contain organic impurity. Grain size distribution curve of virgin soil is shown in Fig. 1. Summary of various index properties are given in Table 1.

2.2 Fibres

Kenaf Fibre

Kenaf fibre comes under the category of bast fibre. The fibre was obtained directly from field after the process of retting. Fibre was air dried and cut in length of 15 mm. Figure 2 shows the typical kenaf fibre. Various mechanical properties of kenaf fibre are presented in Table 2.

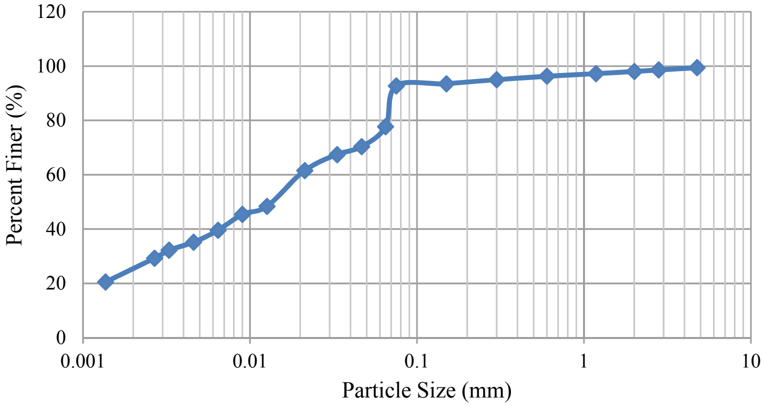

Fig. 1 Grain size distribution curve of virgin soil

Table 1 Engineering property of virgin soil

Properties	Value
Liquid limit (%)	53.05
Plastic limit (%)	33.33
Plasticity index (%)	19.72
Specific gravity	2.66
Natural water content (%)	8.54
Soil classification	MH
Gravel fraction (above 4.75 mm) in %	2.93
Sand fraction (4.75 mm to 75 μ) in %	6.36
Silt fraction (75–2 μ) in %	68.25
Clay fraction (less than 2 μ) in %	22.46

Fig. 2 Kenaf fibre

Table 2 Mechanical properties of kenaf fibre

Properties	Value
Water absorption (%)	120
Relative tensile strength (MPA)	58
Tensile modulus (GPA)	7.7
Specific gravity	1.07
Linear shrinkage (cm/cm)	0.003
Density (gm/cm3)	1.4

Fig. 3 Coir fibre

Table 3 Index properties of coir fibre

Properties	Value
Length (mm)	15–280
Density (gm/cc)	1.15
Breaking elongation (%)	29.04
Diameter (mm)	1–1.5
Specific gravity	1.15
Swelling in water (%)	5
Young's modulus (GN/m^2)	4.5
Tenacity (g/tex)	10.0

Coconut Coir Fibre
Coir fibre comes under the category of fruit fibre. Coir fibres used in this study are of length 15 mm. Figure 3 shows the coir fibre. Various index properties provided by manufacturer are given in Table 3.

3 Methodology

The overall testing was conducted in two phases. In the first phase, the geotechnical properties of virgin soil were studied by conducting various laboratory tests like

Table 4 OMC-MDD value of reinforced soil

Testing	% Fibre	Experimental value	
conditions		OMC (%)	MDD (g/cc)
Virgin soil	0	14.78	1.823
Coir fibre	0.25	13.98	1.942
Kenaf fibre	0.25	11.84	1.950
(Kenaf + coir) fibre	0.25	11.47	2.010
(Kenaf + coir) fibre	0.50	14.10	1.981
(Kenaf + coir) fibre	0.75	14.36	1.972
(Kenaf + coir) fibre	1.0	14.42	1.953
(Kenaf + coir) fibre	1.25	15.55	1.889
(Kenaf + coir) fibre	1.50	16.76	1.830

Atterberg's limit, specific gravity, etc. In the second phase, soil was mixed with different fibre percentages, i.e., 0.25, 0.50, 0.75, 1.0, 1.25 and 1.5%, and was tested for modified compaction and CBR. The pavement thickness was calculated on the basis of CBR values of soil, using pavement design catalogues provided by IRC:SP:72-2015 for low volume rural roads.

4 Test Result and Discussion

4.1 Heavy Compaction Test

The tests were conducted to determine the amount of compaction and the water content required for the sample to achieve maximum dry density. The effects of fibre reinforcement on maximum dry density and optimum moisture content values are given in Table 4. It was observed that for all fibre reinforced specimens, dry density value comes out to be more than that of the virgin soil with a maximum of 2.01 g/cc at 0.25% fibre content, beyond that reduction in MDD was observed. This is due to compactness achieved with the addition of fibres resulting in reduction of voids with void spaces occupied by solid particles having greater specific gravity. The increase in moisture content from 11.47 to 16.76% shows water absorption tendency of natural fibres.

4.2 California Bearing Ratio Test

Variation in CBR value of soil reinforced with various fibre percentages are given in Table 5. The observed CBR values for all fibre reinforced specimens are more than virgin soil with maximum at 1.0% fibre content. CBR value increases up to 1.0%

Table 5 CBR value of reinforced soil

Testing conditions	% Fibre	CBR (%)
Virgin soil	0	2.11
Coir fibre	0.25	2.48
Kenaf fibre	0.25	2.84
(Kenaf + coir) fibre	0.25	3.20
(Kenaf + coir) fibre	0.50	3.91
(Kenaf + coir) fibre	0.75	4.26
(Kenaf + coir) fibre	1.0	5.33
(Kenaf + coir) fibre	1.25	4.62
(Kenaf + coir) fibre	1.50	3.91

fibre reinforcement, and if the fibre content is increased beyond 1%, the CBR value of the reinforced soil samples reduces. This is due to the fact at lower fibre content, three-dimensional bonding between fibre and soil particles is occurring, whereas at higher fibre content, fibres get tangled together preventing bond formation with soil.

4.3 Pavement Thickness and Cost Evaluation

The cost of pavement having 1000 m length and 3.75 m width is determined for both unreinforced and reinforced cases using material and cartage rates from detailed project report (DPR) of Chakghat region, Madhya Pradesh under Pradhan Mantri Gram Sadak Yojana (PMGSY-II). Traffic category T-6 (0.3–0.6 msa) and T-9 (1.5–2 msa) are considered for analysis. Tables 6 and 7 show the cost and thickness of pavement for two different traffic categories, respectively. Reduction in cost is reported for all reinforced cases as compared to virgin soil. Maximum reduction of 31.95% and 22.98%, respectively, for traffic category T-6 and T-9 corresponding to 1% fibre reinforcement, is reported.

5 Conclusions

Based on extensive laboratory tests performed on soil sample reinforced with natural kenaf and coir fibres, it is concluded that as the percentage of fibre content increases beyond 1%, reduction in dry density is observed due to lower specific gravity of fibres. The MDD for reinforced soil varies from 1.83 to 2.01 g/cc. The OMC increases with reduction in MDD and vice versa. The range of OMC for reinforced soil varies from 11.47 to 16.76%. Maximum CBR of 5.33% at 1% fibre content as against 2.11% for virgin soil is also observed. This is due to greater resistance offered by fibres to penetration of plunger. Maximum reduction of 41% in pavement thickness

Table 6 Variation in pavement thickness and cost for traffic category T-6

Testing condition	% Fibre	CBR (%)	Thickness of pavement (mm)	Reduction in thickness (%)	Cost of pavement in Rs. (1 km)	Change in cost (%)
Virgin soil	0	2.11	670	–	4,551,637	–
Coir fibre	0.25	2.48	570	14.92	4,110,234	9.69
Kenaf fibre	0.25	2.84	495	26.12	3,498,574	23.14
(Kenaf + coir) fibre	0.25	3.20	495	26.12	3,498,574	23.14
(Kenaf + coir) fibre	0.50	3.91	495	26.12	3,498,574	23.14
(Kenaf + coir) fibre	0.75	4.26	495	26.12	3,498,574	23.14
(Kenaf + coir) fibre	1.0	5.33	395	41.00	3,096,980	31.95
(Kenaf + coir) fibre	1.25	4.62	395	41.00	3,096,980	31.95
(Kenaf + coir) fibre	1.50	3.91	495	26.12	3,498,574	23.14

Table 7 Variation in pavement thickness and cost for traffic category T-9

Testing condition	% Fibre	CBR (%)	Thickness of pavement (mm)	Reduction in thickness (%)	Cost of pavement in Rs. (1 km)	Change in cost (%)
Virgin soil	0	2.11	845	–	6,088,765	–
Coir fibre	0.25	2.48	745	11.83	5,614,546	7.78
Kenaf fibre	0.25	2.84	595	29.58	5,198,431	14.62
(Kenaf + coir) fibre	0.25	3.20	595	29.58	5,198,431	14.62
(Kenaf + coir) fibre	0.50	3.91	595	29.58	5,198,431	14.62
(Kenaf + coir) fibre	0.75	4.26	595	29.58	5,198,431	14.62
(Kenaf + coir) fibre	1.0	5.33	495	41.42	4,689,470	22.98
(Kenaf + coir) fibre	1.25	4.62	495	41.42	4,689,470	22.98
(Kenaf + coir) fibre	1.50	3.91	595	29.58	5,198,431	14.62

corresponding to 1% fibre content as compared to unreinforced soil is observed. Thus, it can be concluded that use of natural fibres in soil subgrade saves costly base and sub-base aggregate material significantly. Fibre content of 1% is found to be most optimum when compared on the basis of strength improvement, thickness and cost reduction. Deep soil mixing or wet soil mixing technique can be adopted for blending soil with natural fibres using hollow augers, in order to improve the bearing capacity of soil.

These conclusions are of significance for application in engineering projects such as construction of pavement and embankment on weak soil and developing improved reinforcement techniques.

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Stabilization of Sediment Soil by Alccofine and Geogrid

Shimna Manoharan, C. Shashinaga, U. Pradeepa, and P. R. Pavana Kumara

Abstract The main objective of this work is to study the engineering characteristics of sediment soil. The sediment soil is having poor CBR value and is porous and loamy in nature. This property of sediment soil poses problems worldwide that serves as challenge to overcome, for the geotechnical engineers. The aim of this project is to stabilize the sediment soil using different percentages of the stabilizer, i.e., Alccofine 1108SR, and to check for which percentage of the stabilizer added will be the maximum strength gained with the aid of geogrid (biaxial-type geogrid) as reinforcement and also to improve the overall engineering properties of the sediment soil. The results will be then compared with the standard ratio of soil without any stabilizing agent and reinforcement.

Keywords Alccofine · Biaxial geogrid · Sediment soil · Unconfined compression strength

1 Introduction

Soil is a mixture of organic matter, minerals, gases, liquids, and organisms that support life. Soil is the relatively loose mass of mineral and organic materials and sediments found above the bedrock, which can be relatively easily broken down into its constituent mineral or organic particles. Soil consists of a solid phase of minerals and organic matter as well as porous phase that holds gases and the water. Accordingly, soils are often treated as a three-state system of solids, liquid, and gases. For any type of construction, the priority is given to stabilize the soil as the initial step of construction, so that the soil has to bear the effective load coming over that structure [1]. There are many types of stabilization process such as mechanical stabilization, chemical stabilization, cement stabilization, and resin stabilization [1–3].

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2 Materials and Methodology

Alccofine 1108SR—Alccofine is a new generation, micro-fine material of particle size much finer than other hydraulic materials like cement, fly ash, silica, etc., being manufactured in India. It is produced in special equipment to achieve particle distribution in a well-designed range. Alccofine 1108SR has an average particle size of 4–5 microns. The computed Blaine value is around 8000 cm^2/gm. Its specific gravity is about 3.

Biaxial geogrid—A geogrid is geosynthetic material used to reinforce soils and similar materials. Geogrids are commonly used to reinforce retaining walls, as well as subbases or subsoils below roads or structures. Soils pull apart under tension. Compared to soil, geogrids are strong in tension. This fact allows them to transfer forces to a larger area of soil than would otherwise be the case. In this experimental study, Geogrid made of plastic material is used whose aperture size is 15 mm and thickness of about 1.5 mm [4]. The geogrid type used is biaxial. The reason for choosing this type of biaxial geogrid is that, it is stretched along two directions (longitudinal and transverse), thus the stress is equally distributed along both directions, whereas uniaxial is stretched along longitudinal direction [5, 6]. In biaxial geogrids, the longitudinal direction is called as machine direction, and transverse direction is called as cross machine direction.

2.1 Materials

Properties of Alccofine 1108SR

The Alccofine 1108SR are been tested for knowing some of the standard value of some of the basic properties. This helps us to know the standard of material used in the stabilization process of the soil, in this experimental study (Tables 1 and 2).

Properties of Soil The sediment soil for this experimental study is collected from Gantiganahalli lake, near Yelahanka, Bangalore. This soil is been tested to determine the various basic properties of the soil such as natural water content, specific gravity plasticity index, and swell index. All the tests have been conducted as per Indian Standard codes. The results of the basic tests have been tabulated below (Tables 3 and 4).

Table 1 Properties of geogrid

S. No.	Properties	Results
1	Material of geogrid	Plastic
2	Type of geogrid	Biaxial type
3	Aperture size	15 mm
4	Thickness of geogrid	1.5 mm

Table 2 Properties of Alccofine 1108SR

S. No.	Properties	Results
1	Specific gravity	2.94
2	Blaine's permeability value	8068.65 cm^2/g
3	Normal consistency value	36%
4	Compression strength	33.6 N/mm^2

Table 3 Properties of soil

S. No.	Particulars	Results (%)
1	Natural water content	10.79
2	Specific gravity of soil	2.56
3	Liquid limit: Casagrande method	30.89
4	Plastic limit	22.22
5	Plasticity index	10.06
6	Free swell index	20

Table 4 Soil classification based on free swell ratio (FSR)

Free swell ratio	Clay type	Degree of expansion	Dominant clay mineral type
≤1.0	Non-swelling	Negligible	Kaolinitic
1.0–1.5	Mixture of non-swelling and swelling	Low	Mixture of kaolinitic and montmorillonitic
1.5–2.0	Swelling	Moderate	Montmorillonitic
2.0–4.0	Swelling	High	Montmorillonitic
>4.0	Swelling	Very high	Montmorillonitic

2.2 *Standard Proctor Test*

The soil is compacted under standard Proctor test according to the reference code IS 2720: Part 7: 1980. The standard Proctor test is done for the soil without adding any stabilizer, i.e., Alccofine 1108SR and also by adding various percentages of Alccofine as stabilizing agent to determine the OMC and maximum dry density of the sample. Samples have been tested for standard Proctor test with and without addition of Alccofine. The results are computed as follows (Table 5 and Fig. 1).

Table 5 Standard proctor test

S. No.	Particulars	OMC (%)	MDD (g/cc)
1	Soil	13	1.89
2	Soil + 1% Alccofine	16	1.83
3	Soil + 2.5% Alccofine	15	1.89
4	Soil + 5% Alccofine	15	1.92
5	Soil + 7.5% Alccofine	13	1.86
6	Soil + 10% Alccofine	14	1.96

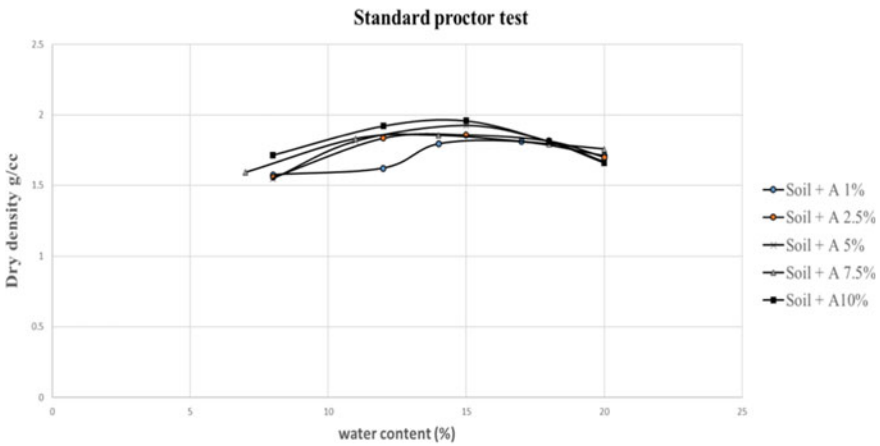

Fig. 1 Compaction curves obtained for various percentages of Alccofine addition

2.3 *Unconfined Compression Test (UCCT)*

Unconfined compression test (UCCT) is conducted on samples consisting of soil with various percentages of Alccofine as stabilizer and geogrid used as reinforcement placed at 1/4th, 1/2th, and 1/4th height from the bottom of the specimen, with curing and also without curing. The curing method adopted in this experimental study is air dry method of curing. It is found that the unconfined compressive strength of the soil is will be increased with the addition of Alccofine. The geogrid is placed at various heights and tested for UCCT. Then, the sample having combination of Alccofine and geogrid is tested for unconfined compression strength.

It is found that the maximum compression strength is found in the sample which is having 7.5% Alccofine, and for the sample which has geogrid at its central position, i.e., geogrid is placed at 1/2th height from the bottom of the sample, under uncured condition (Figs. 2 and 3).

But, after the curing of sample by air dry method for 7 days, when the UCCT is done for the samples, the maximum compression strength is found in the samples which has 10% Alccofine, and for the geogrid reinforced sample, the maximum UCS

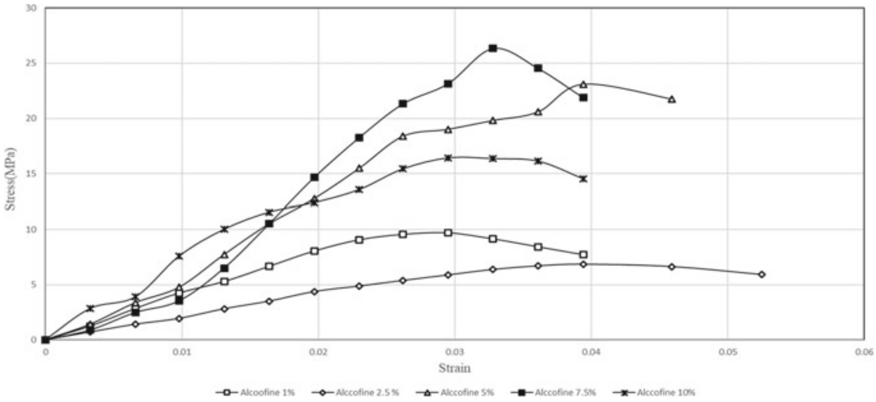

Fig. 2 Unconfined compression test (UCCT) of soil having various percentages of Alccofine, without curing

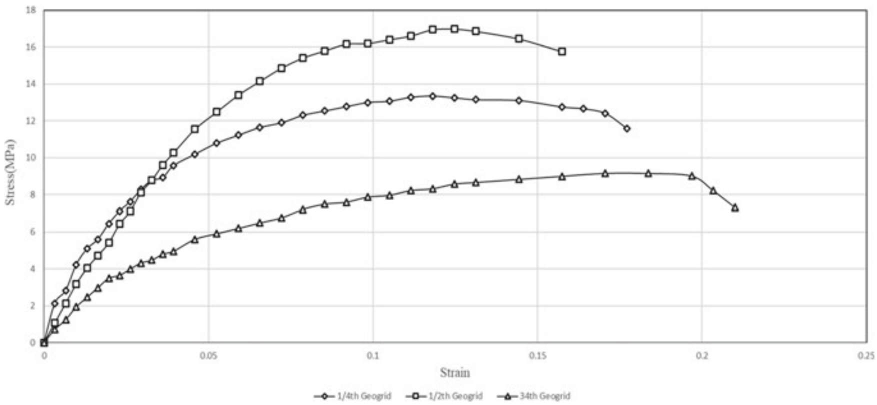

Fig. 3 UCCT test of soil reinforced with geogrid placed at various heights (uncured)

is maximum in the sample where geogrid is placed at 1/2th height from the bottom of the sample (Figs. 4 and 5).

Samples with combination of 7.5% Alccofine and geogrid placed at 1/4th, 1/2th, and 3/4th height from the bottom of the specimen and samples with 10% Alccofine and geogrid placed at 1/4th, 1/2th, and 3/4th height from the bottom of the specimen. This combination is selected because of the reason that the maximum UCS is found for the sample with 7.5% Alccofine under uncured sample and for the sample with 10% Alccofine under cured condition for 7 days. Even though the maximum UCS is found in the sample which is reinforced with geogrid placed at 1/2th height form the bottom of the specimen, it is tested for all the heights again because of the reason that Alccofine being cementitious material, it is not possible to predict how it reacts when it is used in combination with geogrid. We conclude this as the specimen became

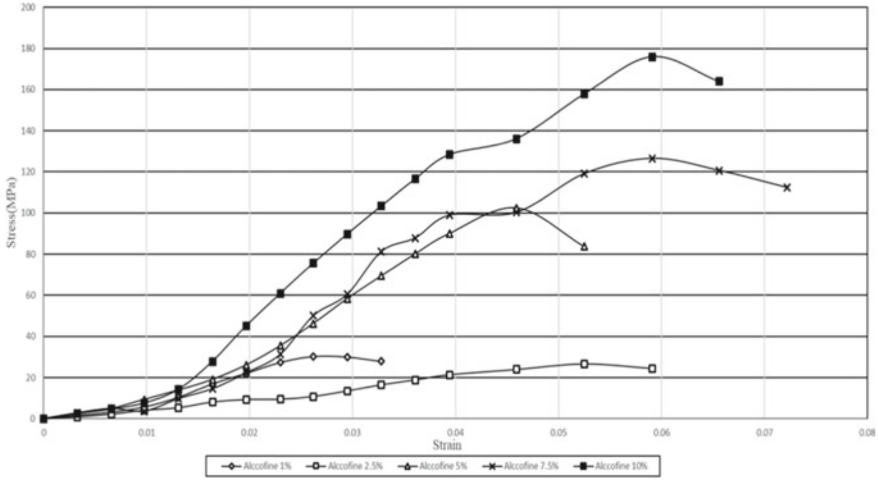

Fig. 4 UCCT test of soil having various percentages of Alccofine, with curing

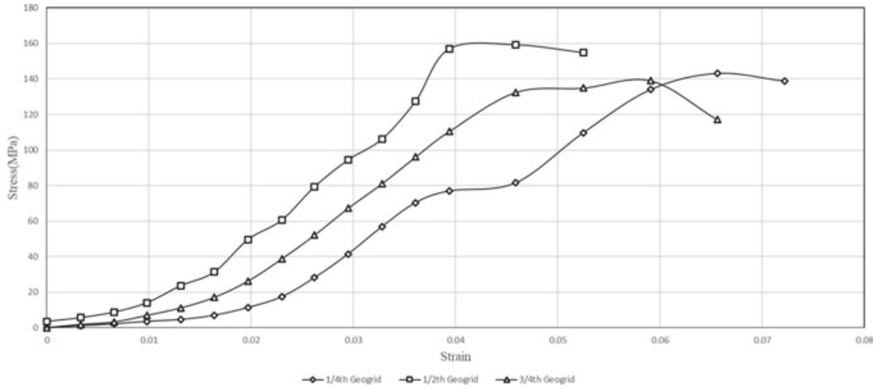

Fig. 5 UCCT test of soil reinforced with Geogrid placed at various heights (air dried)

brittle after 7 days of air dry, it is difficult to predict that how the reinforcement will be reacting on such sample.

Under these combinations, the maximum UCS is found in the samples having 7.5% Alccofine and geogrid placed at 1/2th height from the bottom of the specimen, for uncured samples and in the samples having 10% Alccofine and geogrid placed at 1/2th height from the bottom of the specimen, after air dried for 7 days (Figs. 6 and 7).

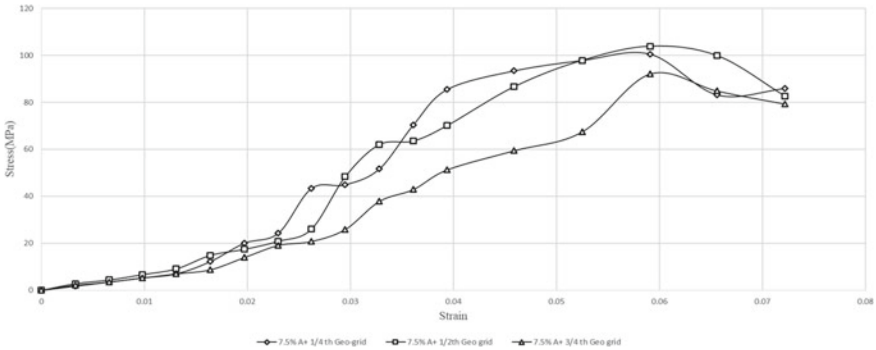

Fig. 6 UCCT test of soil having 7.5% Alccofine and geogrid placed at various heights from the bottom of the specimen, without curing

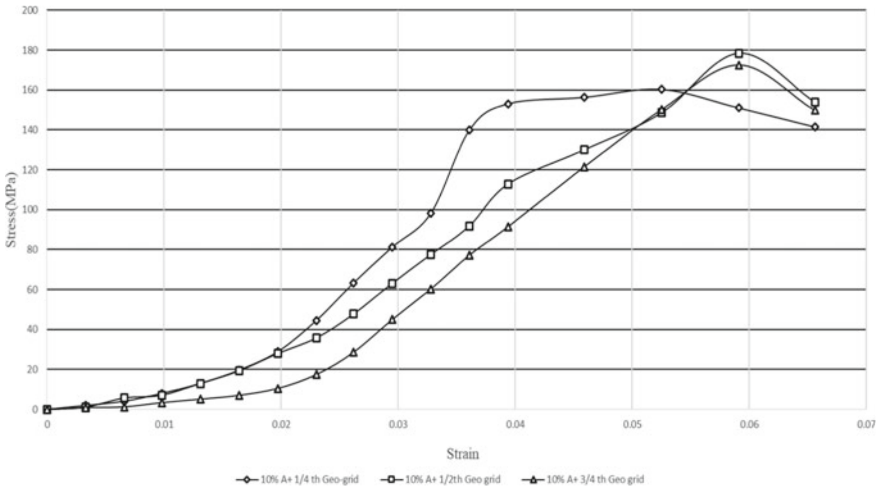

Fig. 7 UCCT test of soil having 10% Alccofine and geogrid placed at various heights from the bottom of the specimen, with curing (air dried)

3 Conclusions

- The plasticity index of the soil is determined as 10.06%. Hence, the soil comes under slightly plastic category.
- The free swell index of the soil is about 20%, and free swell ratio (FSR) is 1.2. As per the soil classification based on FSR, it can be concluded that the dominant clay mineral type present in the soil is a mixture of kaolinite and montmorillonite. Hence, the soil has a mixture of swelling and non-swelling clay type.

- The OMC of the soil samples even after adding various percentages of Alccofine has been within the range of 13–15%, which is nearer to the OMC of raw soil determined.
- The unconfined compression strength is found maximum for the sample having 7.5% Alccofine and for the sample which is reinforced with geogrid which is placed at the center of the specimen, under uncured condition.
- The unconfined compression strength is found maximum for the sample having 10% Alccofine and for the sample which is reinforced with geogrid which is placed at the center of the specimen, under cured condition, i.e., sample cured for 7 days by air dry method.
- When the Alccofine and geogrid used in combination, the maximum UCS is found for the sample which has 7.5% Alccofine and geogrid placed at center of the specimen, for uncured sample, whereas for cured sample, the maximum UCS is found for the sample which has 10% Alccofine and geogrid placed at center of the specimen.

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A Novel Method to Improve the Durability of Lime-Treated Expansive Soil

Nripojyoti Biswas, Sayantan Chakraborty, Anand J. Puppala, and Aritra Banerjee

Abstract Pavements often suffer from different distresses such as rutting and cracking due to the presence of underlying expansive subgrade soils. The ingress and egress of water have a detrimental effect on the performance of the pavements due to the swell-shrinkage behaviour of the subgrade soil. Millions of dollars are invested annually for the maintenance and rehabilitation of such pavements. Traditionally, lime has been used for treating problematic subgrade soils to enhance the strength, stiffness and other engineering properties. However, previous studies have shown that lime-treated soils often incur a significant strength loss when exposed to moisture intrusion, especially in the early curing periods. This research work explores the possibility of using a novel silica-based admixture to enhance the engineering properties of lime-treated soil, reduce the swelling potential and deter the moisture-induced strength loss incurred during early curing periods. Laboratory test results suggest that an expansive soil treated with lime and silica-based admixture has a significant reduction in water absorbing potential and strength loss during the early stages of curing as compared to the soil treated with lime only.

Keywords Soil improvement · Lime treatment · Silica-based admixture · Swelling potential · Durability

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1 Introduction

Lightweight structures and pavements constructed on expansive soils suffer distresses and differential settlements due to swell-shrink behaviour of the underlying soil when exposed to seasonal variations. This phenomenon affects the serviceability and performance of overlying infrastructures, thereby reducing the service life, and subsequently increasing the cost of maintenance and rehabilitation. Calcium-based stabilizers have been used traditionally to stabilize these problematic expansive soils [1–3]. Treatment of these soils with lime results in the reduction of the plasticity and improves the workability [4–8]. The treated soils also show an increase in strength and stiffness properties and a reduction in swell-shrink behaviour [9, 10].

The addition of lime initiates physicochemical changes in expansive soil through the process of reduction in the size of the double diffused layers [1]. Addition of lime initiates cation exchange and facilitates in reducing the plasticity of the soil through the process of flocculation and agglomeration. This immediately improves workability and enhances the strength through 'modification' of the soil [11]. Furthermore, the addition of lime reduces the soil's affinity for water and facilitates in overcoming the problems associated with the potential for swelling and shrinking [1, 12–14].

The optimum lime dosage required to treat a problematic soil is usually determined based on the Eades and Grim pH test as per ASTM D6276. Treating the soil with the optimum lime dosage is required to maintain a high alkaline environment ($pH \geq 12.4$) and is generally considered sufficient to sustain the process of pozzolanic reaction [4]. This facilitates the dissolution of silicates and aluminates present in the clay minerals which reacts with the available Ca^{2+} ions to form calcium-silicate-hydrate (C-S-H) and calcium-aluminate-hydrate (C-A-H) gels, similar to that formed in hydrated cement [2]. The C-S-H and C-A-H gels help in binding the clay particles and improve the engineering properties of the treated soil [2, 12]. The degree of improvement depends on a number of factors such as lime dosage, the curing temperature, the curing time and the type of soil [1, 2, 9].

Although lime-treated soil shows an improvement in engineering properties over the untreated soil, the permanency and long-term durability of these treated soils are affected significantly when exposed to seasonal moisture variations. Studies have indicated that moisture intrusion has a detrimental effect on the lime-treated soil, especially during the early curing periods (<14 days) [15].

Research studies have shown that the durability of treated soil can be improved by increasing the lime dosage. McCallister and Petry [16] showed that the loss of strength due to moisture ingress could be reduced or even neutralized by adding large dosage of lime (4-8%), depending upon the type of soil. However, treating with high percentage of dosage is not suitable for various reasons. Primarily, such high quantity of lime dosage may not be an economical alternative and also the excess lime may infiltrate in the groundwater table and cause palpable health hazards [5]. Therefore, there is a need to find an alternative treatment method that overcomes such shortcomings.

In this context, this research study aims to investigate the use of a laboratory-manufactured novel silica-based admixture as a suitable supplement to the existing lime treatment method. The influence of this admixture on the enhancement of the short-term unconfined compressive strength (UCS) and reduction in swell potential in lime-treated soil has been studied. Furthermore, the effect of curing time on the aforementioned engineering properties has also been investigated. To study the durability characteristics of this improved treatment method, a comparative study of the soaked and unsoaked UCS was performed for estimating strength reduction due to moisture ingress for 0, 3 and 28 days cured lime-treated and lime-admixture-treated samples.

2 Materials and Experimental Procedures

2.1 Materials

Experimental studies were performed using a problematic local soil collected from a road construction site in North Texas. The soil obtained from the site was dried in an oven at 110 ± 5° C for 24 h, crushed, pulverized and finally homogenized. The basic soil characterization tests were performed in accordance with the respective ASTM standards [17], and the results are provided in Table 1. The untreated soil was classified as CH as per ASTM D2487, with a PI of 36.5 (high PI clay), 1D free swell of 16% (high swelling clay), unconfined compressive strength of 330 kPa, and a strength reduction of 95% after 24 h of capillary soaking. Based on the basic soil characterization test results, lime was selected as the most suitable stabilizer [11].

Industrial grade hydrated lime conforming to the ASTM standard C977 was used for treating the problematic soil. The optimum lime dosage of 7% (by weight of dry soil) was selected based on Eades and Grim pH test as per ASTM D6276. The same lime dosage was used for treating the soil with lime, and lime-admixture combination is to facilitate the comparative study.

A novel silica-based admixture was prepared in the laboratory from a locally available geomaterial. The admixture was prepared at 21 ± 1° C and checked for impurity. For preliminary studies, the percentage of silica admixture was assumed to be 30% of the weight of dry untreated soil. This particular dosage was chosen after a performing trial with higher admixture dosages. It was observed that 30% admixture

Table 1 Basic soil characterization test results

Properties	
Liquid limit, w_l (%)	66.0
Plastic limit, w_p (%)	29.5
Plasticity index, PI (%)	36.5
Specific gravity (G_s)	2.72

was required to prevent the immediate strength loss of the lime-treated soil when exposed to capillary soaking. Details of the strength loss after capillary soaking are presented in Sect. 3.1.

2.2 Strength Testing

Sample Preparation Sustainable use of resources has been a long-term goal of the researchers in the present century [18]. The preparation of samples conforming to ASTM D2166 requires a substantial volume of soil [19]. However, sampling restrictions often impede the collection of such a large quantity of soil. Exploration for such soils may further incur extra charges for the project. Therefore, considering the above drawbacks, miniature samples conforming to ASTM-STP479-EB were prepared using Harvard Miniature Compaction setup (Fig. 1a).

The preparation of a miniature sample is less tedious and requires less quantity of geomaterials. Laboratory studies were conducted at the University of Texas at Arlington research facility to compare the UCS of the miniature sample (33 mm diameter) to that of the standard laboratory sample (72 mm diameter). Experimental results indicated that the mechanical performance of miniature samples was similar to that of standard samples prepared at the same aspect ratio (height (H): diameter (D)). Furthermore, the UCS test results were primarily used for a comparative study; hence, the relative changes in UCS values were more important as compared to the absolute UCS values. Therefore, for further experimental studies, miniature samples of diameter 33 mm were used. The samples were prepared at an aspect ratio of 2(H): 1(D) conforming to ASTM D2166 (Fig. 1b).

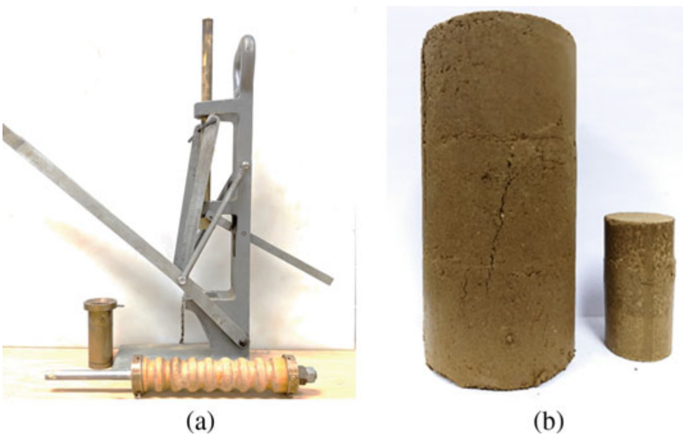


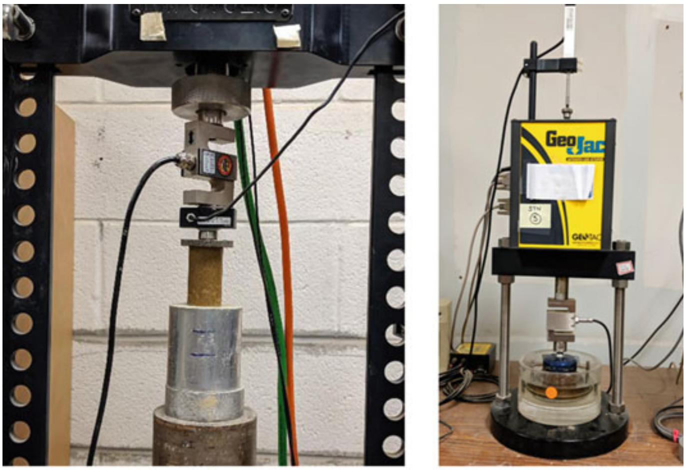
Fig. 1 **a** Harvard Miniature Compactor; **b** standard sample and miniature sample

The lime-treated samples were prepared at a maximum dry density (MDD) of 13.75 kN/m^3 and optimum moisture content (OMC) of 19%. Previous studies have shown that the samples prepared at the same OMC and MDD have similar initial strength [14, 20]. Therefore, for the comparative study, both the lime-treated and lime-admixture-treated samples were prepared at the same target dry density and moisture content corresponding to the MDD and OMC of the lime-treated sample. Lime-treated samples were prepared by uniformly mixing the dry soil with the target lime dosage of 7%. Whereas for the admixture supplemented samples, the admixture was first mixed uniformly with the dry soil, and then 7% lime was added to it. Distilled water was added to both the dry mixtures and hand-mixed thoroughly to prepare a homogeneous mixture. After homogeneous mixing, a mellowing period of 8 h as per ASTM D3551 was allowed before the samples were moulded. The mellowing period allows the initiations of the initial cation exchange, decrease of the size of the double diffused layer and equilibration of the mixture.

The samples were moulded in three equal layers in Harvard Miniature Compactor. The target density was achieved through 25 tamps with 89 N spring force for each layer. Three sets of samples were prepared for each type of mixture corresponding to each curing period. The samples were cured for 0, 3 and 28 days in small airtight moisture proof polythene bags with 10 ml of free water to ensure that relative humidity remained close to 100% for proper pozzolanic reactions [2].

UCS Testing

Unsoaked UCS. The UCS of both untreated and the treated samples were performed as per ASTM D2166. The setup for the test is presented in Fig. 2a. Treated samples were tested after 0, 3 and 28 days curing period. Before the start of the test, a small sitting load of 1 kPa was applied to ensure proper contact of the surface. The samples



(a) (b)

Fig. 2 **a** UCS testing setup and **b** swell test setup were tested at a constant strain rate of 0.5%/min and the maximum strain limit was set at 5%.

Soaked UCS and Durability Studies. The durability of lime-treated soil is generally performed as per ASTM D559. Although the standard is ideally applicable for UCS testing of soil-cement mixtures; professional practitioners use it extensively for lime-treated soils. It is generally observed that this testing method is time-consuming and requires a large quantity of resources [19, 21]. Research studies have suggested that instead of exposing the treated samples to extreme wetting and drying cycles as per ASTM D559, UCS testing after 24 h of capillary soaking can be used as a measure of the durability of lime-treated soil [2, 14]. Therefore, for the present durability studies, the cured samples were subjected to capillary soaking for 24 h and then subjected to UCS testing. A comparative study between the UCS values of lime-treated soil and admixture supplemented lime-treated soil was performed to understand the improvements in immediate and long-term strength retention properties due to addition of the admixture. The test results were compared to the unsoaked UCS test results, and percentage strength loss was used as an alternative measure of the durability of the samples.

2.3 Swell Potential

Sample Preparation The one-dimensional swell tests were performed for both untreated and treated samples in accordance with ASTM D4546. For samples treated only with lime, 7% of lime by weight of the dry soil was added. In case of the lime-treated soil mixed with admixture, 30% of the admixture by weight of dry soil was added to the samples. All the samples were prepared by static compaction at the target moisture content and dry density specified in Sect. 2.2.

Swell Test Swell tests were performed as per ASTM D4546 test method A as shown in Fig. 2b. Both treated and untreated samples were subjected to one-dimensional free swell test under a seating load of 1 kPa.

3 Analysis and Discussion of Results

3.1 Strength Testing

Unsoaked UCS Figure 3 presents the strength gain of the treated samples with an increase in curing period. The average initial strength of the lime-treated and lime-admixture-treated samples was observed to be 385 kPa and 347 kPa, respectively. The initial strength of the both samples was similar to that of the untreated sample

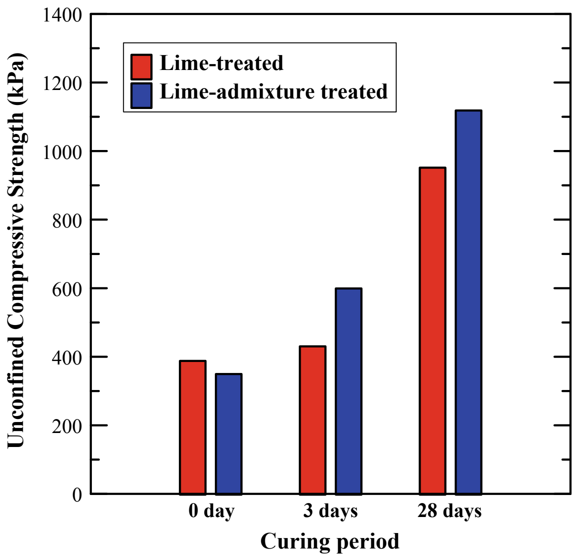


Fig. 3 UCS of treated samples for different curing periods

(UCS value of 330 kPa). This increase in the initial strength as compared to untreated sample may be attributed to the 'modification' induced due to the addition of lime and admixture to the soil. As the curing time increases, the strength increases due to the formation of cementitious compounds that bind the clay particles to form a strong matrix [11, 22].

However, it is observed that the rate of strength gain is higher for the admixture-treated lime-soil mixture in comparison with only lime-treated soil samples. After 3 days, the strength of lime-treated sample is only 427 kPa as compared to admixture-treated sample, which is around 600 kPa. Furthermore, for longer curing time, e.g., 28 days, the UCS value of admixture-treated lime-soil mixture is observed to be 20% higher than the lime-treated soils. The 28 days UCS of lime-treated soil is 950 kPa as compared to 1116 kPa for the soil treated with lime and silica admixture. The presence of secondary silica molecules available from the admixture serves as a source of readily available nucleation site for the formation of cementitious gel, in the presence of available calcium ions and high pH environment, whereas, in case of only lime-treated soil samples, the silicates are solely available from the dissolution of the clay particles. This phenomenon may be responsible for the higher strength of the lime-admixture-treated samples. Overall, the experimental outcomes indicate that the addition of admixture has a beneficial effect on the mechanical strength of the stabilized soil.

Soaked UCS and Durability Studies The improvement in the durability of lime-treated problematic soil was studied by analyzing the unconfined strength behaviour of the capillary soaked samples. The UCS of the samples subjected to 24 h of capillary soaking are presented in Fig. 4. According to NCHRP W144 [11], the minimum

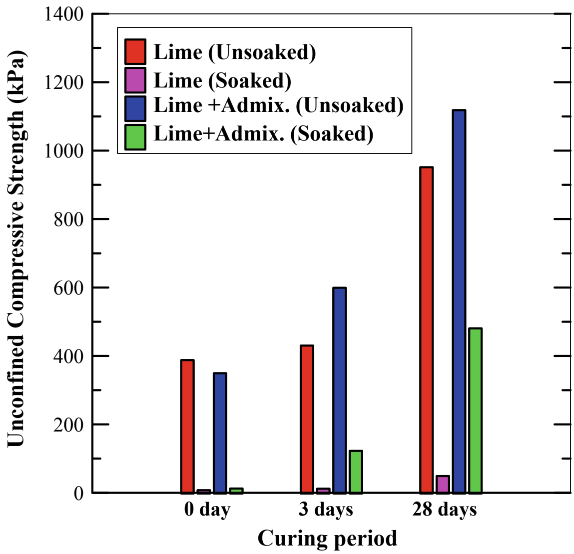


Fig. 4 Unconfined strength of treated samples for different curing time with and without capillary soaking

strength retained after 24 h capillary soaking should be more than 50 psi (345 kPa) for a treated soil subjected to 7 days of accelerated curing (which is equivalent to 28 days normal curing). From the experimental results, it is observed that both the treated samples incur a significant strength loss (>95%) when exposed to capillary soaking immediately after preparing the samples (0-day curing). However, with the increase in the curing period, the presence of admixture has a substantial influence on the strength retained by the soaked samples. After 3 days of curing, the soaked strength of admixture-treated soil was found to be 120 kPa, which is 20% of the unsoaked strength. This is a notable improvement in comparison with only lime-treated soil, which could retain only 2% of the unsoaked UCS of 430 kPa. With further curing for 28 days, it can be observed that the strength retained by lime-admixture-treated sample increased to 40% of its unsoaked strength of 1116 kPa. Whereas, the 28 days cured lime-treated samples failed to retain the minimum target UCS value of 345 kPa when subjected to capillary soaking.

Little [9] stated that the deleterious effect of the soil soaking is significant if the retained strength after capillary soaking for at least 24 h is less than 60%. Although the retained strength by the lime-treated and lime-admixture-treated samples was less than 60% for this particular soil, the addition of silica admixture has greatly improved the performance in comparison with that of only lime-treated soil. Therefore, it may be interpreted that, due to the addition of the silica-based admixture to the lime-treated soil, the amount of cementitious CSH gel formed is substantially more as compared to only lime-treated soil. Hence, after moisture ingress, the strength loss is more significant in the lime-treated soil as compared to lime-treated soil mixed with admixture.

Table 2 Moisture content of capillary soaked soil for different curing periods

Curing period (days)	Moisture content (%)	
	Lime-treated soil	Lime-treated soil with admixture
0	64.56	57.02
3	52.11	36.09
28	50.40	35.20

Formation of the CSH gels probably reduces the pores available for moisture absorption in the treated soils. As the curing period increases, available voids for moisture ingress decreases due to the reduction of porosity. Therefore, this reduction of absorbed moisture further helps in retention of strength in lime-admixture-treated samples. Table 2 shows the final moisture content in the samples subjected to capillary soaking. For the initial curing period, the amount of CSH gel formed is negligible; therefore, the availability of pores for moisture absorption is high. So, the 0-day cured sample shows high water absorption percentage. With the increase in curing period, samples treated with lime and admixture absorb substantially less moisture as compared to samples treated with lime only. This phenomenon may be attributed to the fact that more CSH gels have formed within the sample, which holds the matrix as a strong interconnected unit and reduces the available voids for water intrusion.

From the above observations, it can be inferred that the addition of the silica-based admixture to the lime-treated soil has a beneficial influence in strength retention after exposure to capillary soaking, both for immediate and longer curing periods.

3.2 *Swell Potential*

The swelling potential of the untreated and treated soils are shown in Fig. 5. From the figure, it can be observed that the native clay has a high swell potential of more than 16%. Similar to the UCS testing, the treated samples were tested after three curing periods of 0, 3 and 28 days. The extent of formation of cementitious gel depends upon the length of the curing period after the addition of lime or admixtures. With the progress of time, pozzolanic reactions take place, and the lime-admixture-treated soil showed visible improvement in swell resistance. After 3 days of curing, it was observed that the admixture supplemented lime-treated soil showed improved performance as compared to only lime-treated soil (Fig. 5a). The availability of excess silica from admixture helps in the expediting the formation of CSH gel, which may be the principal reason for imparting this improvement. The swell test on samples cured for 28 days showed that both the treatment methods have a comparable impact on reduction of swelling potential of the soil (Fig. 5b). So, the addition of admixture has the beneficial effect on reducing swell potential of the lime-treated expansive clay and partially counteracting the detrimental effect of moisture intrusion in the early days of curing.

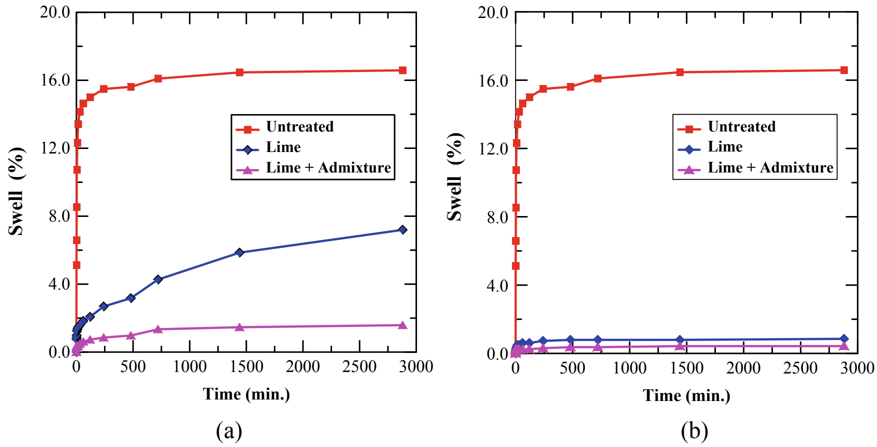


Fig. 5 Swelling potential of treated samples, **a** after 3 days curing period and, **b** after 28 days curing period

The laboratory studies suggest that the addition of silica-based admixture helps in improving the UCS value of the treated samples as compared to only lime-treated soils. The admixture-treated samples when cured for 3 days and 28 days, showed notable strength retention after capillary soaking as compared to only lime-treated soils. This increase was attributed to the formation of additional pozzolanic gels due to the presence of the supplementary source of silica from the admixture. Finally, swell studies suggested that the admixture has a substantial influence in the reduction of swell potential for initial curing periods. The reduction in the swelling potential of the admixture supplemented lime-treated soil after 3 days of curing highlights the effectiveness of this treatment over traditional lime treatment methods, especially during the early ages of curing.

4 Conclusion

The durability of soil treatment is significant for estimating the long-term performance of a pavement section. Researchers have often observed that lime-treated soil fails to perform suitably due to moisture ingress during its early curing period, therefore, incurring more project cost. To overcome this problem, the use of a novel silica-based admixture has been proposed, which can be added during the lime treatment process to improve its mechanical properties and reduce the detrimental impacts of moisture intrusion. Followings are the major conclusions that can be drawn from the findings of this research study:

- The laboratory test results suggest that the novel silica-based admixture helps in considerable improvement of strength over the traditional lime treatment methods.

- The principal advantage of this novel admixture was observed when its performance after moisture ingress for shorter curing period (3 days) was analysed. This novel admixture can be suitably used to improve the durability of the treated soil, especially if there are chances of moisture intrusion in the early curing stages.
- Both short-term and long-term mechanical performance can be significantly improved by the addition of this novel silica-based admixture during lime stabilization.
- Besides enhancing the strength properties, the lime-admixture treatment helped in significantly reducing the moisture absorption and swelling potential as compared to lime treatment alone.

Future prospect of the research includes a detailed study of the morphological and mineralogical characteristics of the silica admixture so that a comprehensive soil treatment methodology using this novel admixture can be suggested for practicing engineers. Further studies are also required to reduce and optimize the proportion of admixture necessary for improving the engineering properties of a problematic soil.

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Strengthening of Cohesionless Soil Using Basalt Fibre Geogrids

J. Jasmin and K. Balan

Abstract Modern soil reinforcement uses more durable materials and advanced methodology in increasing the bearing capacity of feeble soils [2]. Basalt fibre geogrids can be characterized as a green nonpolluting material used in stabilizing the week foundation soils by the interaction of frictional forces that develops at the soil reinforcement interface [3]. This revolutionary fibre obtained from volcanic extrusive basalt rock possess high tensile strength than steel fibres and even effectively replaces glass and carbon fibres in terms of cost-efficiency and performance [7]. Tests were carried out to determine the effectiveness of basalt fibre geogrids in cohesionless soil. Cellular arrangement [8] of geogrid with coir fibre inclusions at different confinement depth ratios [1] (CD/B = 0.25, 0.75, 1.5 and 2) and variation in the number of geogrid cells (N = 1, 5 and 9) were performed. Results show that maximum strength can be attained within the zone of influence at optimum coir fibre content [6].

Keywords Basalt fibre geogrid · Cellular arrangement · Coir fibre inclusions

1 Introduction

The principle of reinforced earth has been effectively utilized more than three thousand years ago by the Babylonians in the construction of ziggurats. A part of the Great Wall of China is also an example of soil reinforcement. Reinforcement can be provided to the soil by using either physical methods like vibration or by using chemical methods utilizing enzymes and resins [10] or by adopting mechanical methods involving geogrids, geonets and geocells. Geogrids can be classified into three categories as punched and drawn type, flexible textile geogrids consisting of polyester fibres as reinforcing elements and laser geogrids including polyester fibres or straps bonded ultrasonically into geogrid meshes. Basalt fibre geogrids can well be designated as nonpolluting green material of twenty-first century and a sustainable product

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Table 1 Properties of cohesionless soil

Properties	Values
Specific gravity, G	2.64
Effective size, D10 (mm)	0.16
D60 (mm)	1.18
Uniformity coefficient, Cu	7.37
Maximum dry density (g/cm^3)	1.82
Minimum dry density (g/cm^3)	1.45
Emin	0.45
Emax	0.84
IS classification	SW

as it does not require any chemical additives, solvents or enzymes during its production process. Basalt products have no noxious reaction with air or water and have proven to be non-carcinogenic in nature with minimum moisture absorption capacity [13]. They possess high tensile strength and resistance to fire and ultraviolet radiations and its excellent damping properties makes it useful in acoustic insulation [14]. Present study focusses on the utilization of basalt fibre geogrid cells in improving the bearing capacity of week soils.

The effect of coir fibre in combination with the geogrid cells has also been studied at various fibre contents of 0.4, 0.6, 0.8 and 1%.

2 Materials Used

2.1 Cohesionless Soil

The soil sample was collected from nearby the campus of RIET, Attingal. The sample was properly cleaned from impurities, oven dried and sieved using IS 4.75 mm and IS 0.075 mm sieve as per IS 2720 (Part-4)—1985 to attain the desirable gradation. The properties of soil are discussed in Table 1.

2.2 Basalt Fibre Geogrid

The properties of basalt fibre geogrid as shown in Fig. 1 of grade 350 gsm are listed in Table 2.

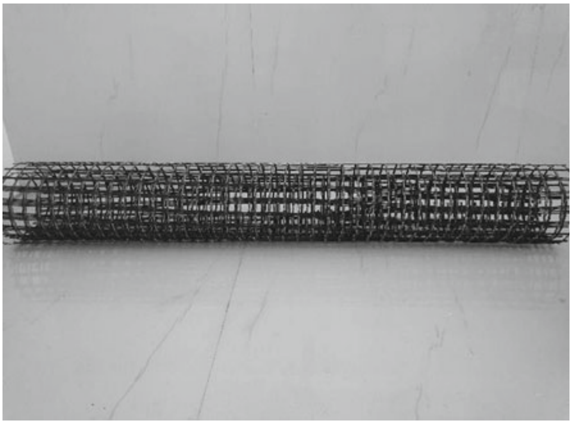


Fig. 1 Basalt fibre geogrid

Table 2 Properties of basalt fibre geogrid

Properties	Values
Opening size (mm)	25
Thickness (mm)	0.08
Weight (g/m^2)	350
Max load—warp (N/m)	80,780
Max load—weft (N/m)	78,900
Elongation at break—warp (%)	6.67
Elongation at break—weft (%)	3.53

3 Methodology

A test tank made of mild steel with dimensions of 600 mm × 600 mm × 600 mm and a mild steel plate of size 100 mm × 100 mm with a thickness of 20 mm was used as the square footing. The footing is placed in such a way that the axis of loading coincides exactly with the centre of the plate. The dimensions of the tank were designed as per IS: 1888-1982 such that the size of the tank was always kept five times the width of the footing. The experimental setup used is shown in Fig. 2.

The test tank was filled with soil using sand raining method at relative densities of 35% and 85%, respectively, with geogrids cells placed at different confinement depth ratios (CD/B). A loading frame was set to work as per the lever arm principle. After applying the balancing load on one side of the lever arm, the load was applied at equal increments and the corresponding settlements were observed using two 50 mm dial gauges. The final settlement was obtained by calculating the average of the two values. Loading was continued until failure occurs due to excessive settlement [4].

Fig. 2 Experimental setup

4 Results and Discussion

Load settlement analysis were carried out to determine the effectiveness of basalt fibre geogrids in cohesionless soil. Following results were obtained.

4.1 Unreinforced State

Load settlement curve was plotted for the soil under natural condition with a relative density of 35%. The curve shows a general shear failure with a failure load at 5.2 kg.

The soil was then compacted in the influence zone of footing that is 1.5 B at a relative density of 85% and the load settlement curve was plotted as shown in Fig. 3.

With the increase in relative density from 35 to 85%, the load carrying capacity has increased by 240% as the soil particles get rearranged to more denser state with less number of voids.

4.2 Reinforced State—Coir-Geogrid Cellular Composites

In cellular composites, the geogrids are placed in the form of cells encasing the soil and coir fibres at varying fibre content and confinement depth of geogrid. The top view of a 3 × 3 cell arrangement with the length of one side as 3 B, where B is the width of footing is shown in Fig. 4.

Fig. 3 Load settlement curve in unreinforced condition

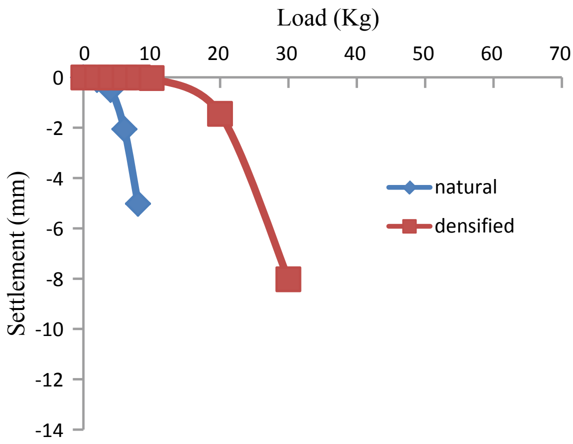
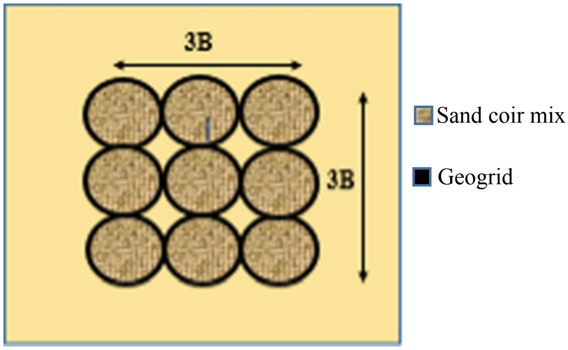
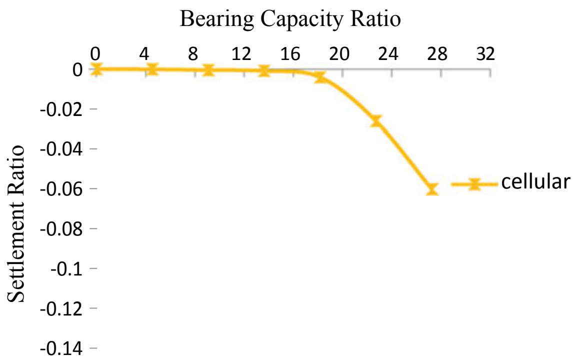


Fig. 4 Top view of cellular model with 9 cells



A graph is plotted between BCR and settlement ratio for the reinforced soil with constant number of cells as N = 9 at optimum fibre content and confinement depth as shown in Fig. 5.

Fig. 5 Cellular form of geogrid at N = 9 cells



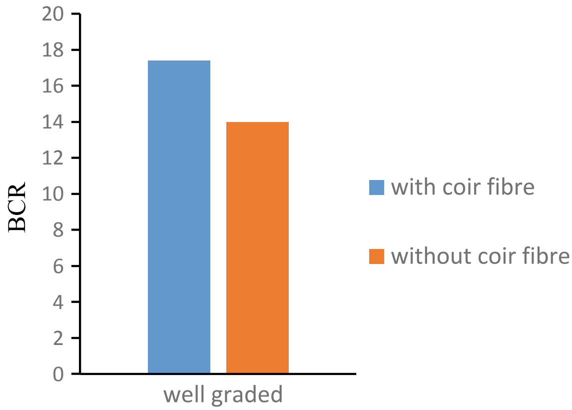


Fig. 6 Cellular form of geogrid with and without coir fibre

An increment in bearing capacity was observed with the addition of coir fibres to the geogrid cells as shown Fig. 6 as fibre strands holds the soil particles without being displaced off at the time of loading.

4.3 Optimum Fibre Content

Basalt fibre geogrids were placed in the form of cells filled confining the sand-coir mix with varying number of geogrid cells and percentage of coir fibre from 0.4 to 1% with confinement depth of cells extending up to the zone of influence [15]. A graph was plotted between BCR and percentage fibre content as shown in Fig. 7.

It was observed that the optimum coir content was observed to be at 0.6% with a maximum number of geogrid cells arranged in a 3 × 3 form. Beyond the optimum point, the fibre content exceeds the percentage of soils particles for which a decrement in bearing capacity was observed [5].

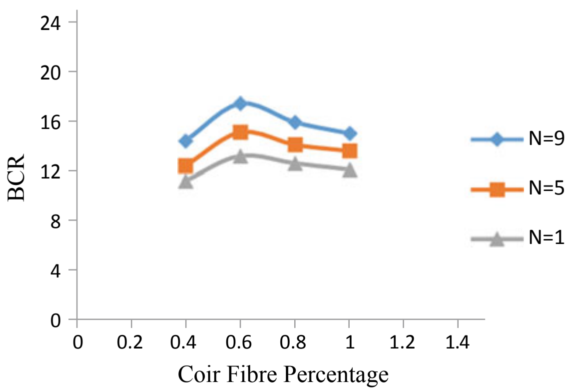


Fig. 7 Variation in BCR at different percentages of coir fibre

4.4 Optimum Confinement Depth

A cellular form of geogrid was placed with varying confinement depth ratios of 0.25, 0.75, 1.5 and 2 B at optimum fibre content of 0.6% as shown in Figs. 8, 9, 10 and 11.

It can be observed that in the first three cases, the cellular geogrid was placed in the compacted fill, whereas, at CD = 2 B, the cellular geogrid was placed penetrating into the loose fill by 0.5 B. All the cells were placed at 0.25 B below the bottom surface of the footing.

Settlement curve was plotted by varying the CD/B ratio of geogrid cells at optimum coir fibre content as shown in Fig. 12. BCR ratio increases with increase in

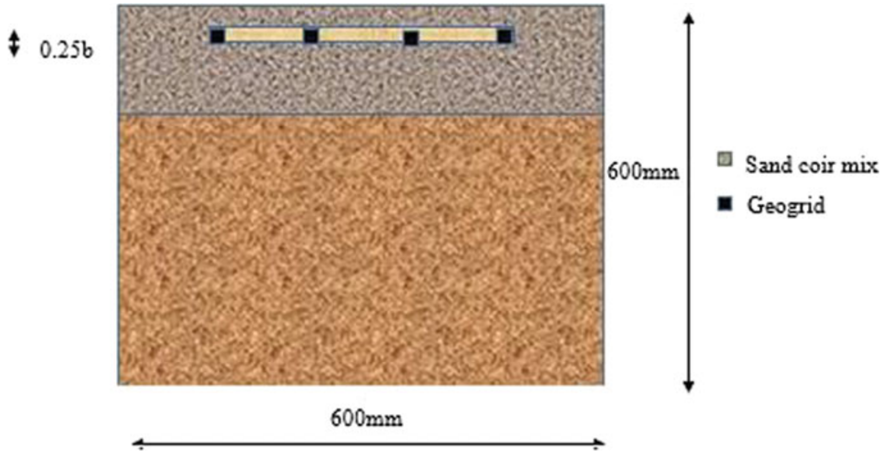


Fig. 8 Cross section of cellular model at CD/B = 0.25

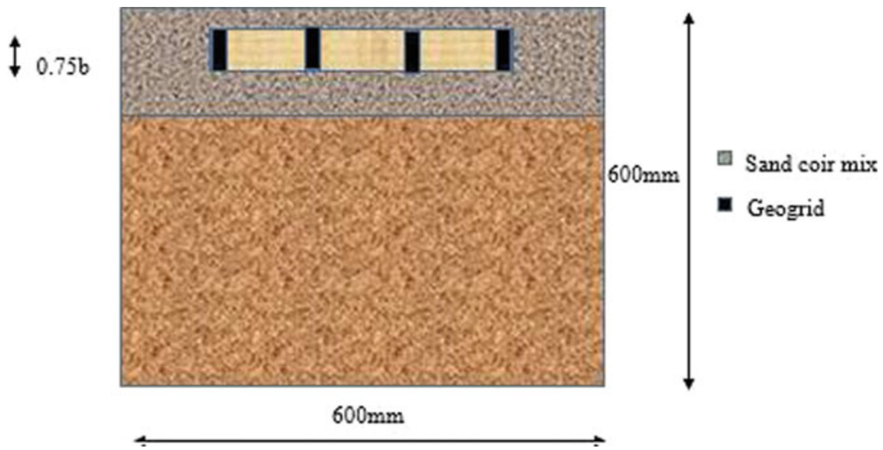


Fig. 9 Cross section of cellular model at CD/B = 0.75

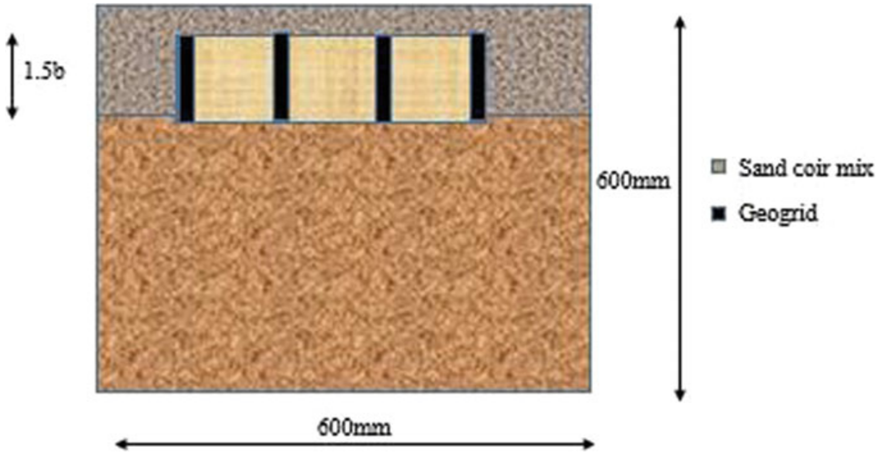


Fig. 10 Cross section of cellular model at CD/B = 1.5

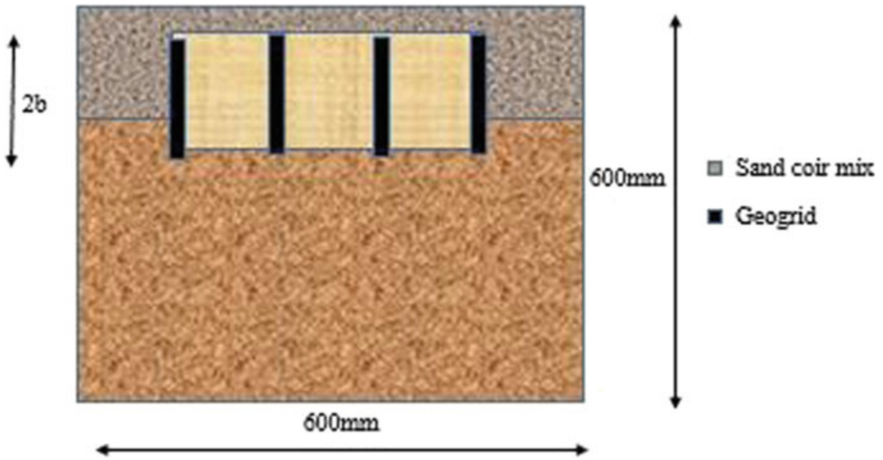


Fig. 11 Cross section of cellular model at CD/B = 2

confinement depth ratio [9] in the reinforced area with a relative density of 85% due to the all-round confining action of the geogrid cells and maximum benefit could be achieved in a 3 × 3 cellular form. It was also observed that there is no increment in BCR beyond

the zone of influence that is at CD/B = 2 which penetrates into loose fill with relative density of 35%.

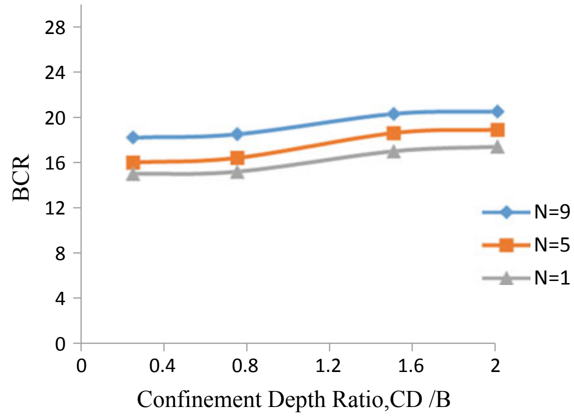


Fig. 12 Cellular model of geogrid with varying confinement depth ratios (CD/B)

5 Practical Significance

Rising cost and decreased availability of areas for urban infill have forced the mankind to take up construction in undeveloped areas which possess weak underlying foundation material. This has set a great challenge for the geotechnical engineers, especially in utilizing cohesionless soil for construction activities. In sandy soils, the settlement occurs when the soil gets too wet causing an increase in pore pressure and thus with low friction between the sand particles it smoothly shifts out of place resulting in the collapse of the structures built on it [11]. Studies have been carried out over the past decades for improving the performance of shallow foundations using geosynthetics [12]. Researches have been carried to the present era to attain an economical and environment-friendly product which can be more effectively utilized in strengthening soils of low bearing capacity. Basalt fibre geogrids have been successful in satisfying these criteria to a certain extend.

6 Conclusion

- The load bearing capacity of the soil increased with the introduction of coir-geogrid cells and the settlement reduced with the increase in cells from N = 1, 5 and 9 due to the increased stiffness of the corresponding cells.
- Maximum benefit was achieved in cellular model at CD/B = 1.5 due to the all-round confining action of the geogrid.
- Lesser settlement was observed at CD/B = 1.5 beyond which there was less influence of the restraining action of the geogrid.
- Optimum percentage of coir fibre added was at 0.6% beyond which the effect is negligible.

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Stabilisation of Kuttanad Soil Using Calcium and Sodium Lignin Compounds

B. S. Sabitha and Y. Sheela Evangeline

Abstract Kuttanad region in Kerala is the lowest lying area in India with an elevation of one metre above mean sea level. Soil in this region is characterised by low bearing capacity, hence, construction works on Kuttanad soil is often problematic and expensive. Traditionally, stabilisation in Kuttanad soil is done by hydrated lime, Portland cement, fly ash, etc. Stabilisation of weak soil with industrial by-products is commonly adopted nowadays because of its economical as well as environmental benefits. One such material is lignin; chemically called as lignosulphonates produced as a by-product in paper pulping industry. Lignin compounds are available in different chemical compositions based on the cellulose separator used during pulping process. This paper presents in detail the effectiveness of stabilising Kuttanad soil using sodium as well as calcium forms of lignin. The effect of lignin on compaction characteristics, consistency limits, unconfined compressive strength and CBR were studied using a series of laboratory tests and the optimum percentage of additive was found out. A comparative study has also been done with the two compounds to identify which one has the best stabilisation capacity.

Keywords Kuttanad soil · Lignosulphonate · Wood pulp · CBR

1 Introduction

In a developing country like India, rural roads play an important role by increasing access to economic and social services, and thereby increasing agricultural income. The major problems faced by roads across the country are they are built in poor subgrade. India being a developing country produces tonnes of industrial and agro-based by-products, most of them are found to have excellent stabilisation capabilities.

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Stabilisation with industrial by-products is commonly adopted nowadays as it is an effective way of disposing the huge piles of such waste products which would otherwise became a pollutant due to improper disposal. Hence, the stabilisation with industrial by-products is found to have economic, environmental as well as social benefits.

Kuttanad is a low lying region comprising an area of about 900 km^2 in the Alappuzha district of South Kerala. Kuttanad soil is a black marine soil deposited by backwater tract characterised by low bearing capacity, hence, construction works on Kuttanad soil is often problematic and expensive. In this study, an attempt was made to investigate the ability of lignosulphonate by-products in stabilising Kuttanad soil. Lignin is the second richest renewable natural resources after cellulose. About 50 million tonnes of lignin is produced worldwide every year [5]. Commercial lignin is produced as a by-product of the paper pulping industry. It is separated from trees by a chemical pulping process. Subgrade stabilisation has been identified as one of the viable answers to consume huge quantities of lignin produced per annum [6].

Laboratory investigations conducted to find the effect of lignin on geotechnical properties suggest that lignin has a considerable influence on the mechanical properties, particle size distribution and consistency limits of soil [6]. Stabilising expansive soil with lignin reduced the swelling potential and improved the resistance to alternate wet and dry cycles. The swell potential of lignin stabilised soil decreased by about 23%. Shrinkage limit of stabilised soil also improved but little change was only observed in compaction and permeability characteristics [1]. Micro-chemical analysis on lignin stabilised soil revealed that the improvement of soil properties is due to the cation exchange between the soil and lignin, hence, forming a cementing substance which holds the soil particles together. It is found that the lignin-based stabilisers have a great potential to enhance the engineering properties of soil [2]. Lignin stabilised soil have better thermal resistivity and mechanical properties. Curing time has great influence in enhancing the properties of soil [7]. From the literatures, it could be concluded that lignin is effective in stabilising weak soil. In this study, the suitability of using lignosulphonates for stabilising Kuttanad soil is explored.

2 Materials Used

2.1 Soil

The soil used in the study is collected from the Edathua region of Kuttanad, Alappuzha district. The soil had been collected from a depth of 2.5 m from the ground surface. The properties of soil sample are presented in Table 1. From the plasticity characteristics as per IS specification, the soil is found to have intermediate plasticity.

Table 1 Properties of Kuttanad soil

Property	Value
Specific gravity	2.20
Natural water content (%)	90.6
Clay content (%)	22.0
Silt content (%)	76.0
Sand content (%)	2.0
Liquid limit (%)	49.0
Plastic limit (%)	18.0
Shrinkage limit (%)	13.7
MDD (g/cm^3)	1.6
OMC (%)	27.0
UCC (kPa)	12.8
Unsoaked CBR (%)	0.9

2.2 Sodium and Calcium Lignosulphonate

Commercially produced lignosulphonates are used for the study. The properties of lignosulphonates used in the study are shown in Table 2.

3 Methodology

Suitability of using sodium and calcium lignin compounds as a subgrade stabiliser is evaluated by studying the strength characteristics and plasticity characteristics. To find the stabilisation capacity of the two lignin compounds, a series of laboratory tests are done by varying amount of lignosulphonates. In the case of sodium lignosulphonate, the content is varied as 2, 5, 8, 12 and 14%, whereas, in the case of calcium lignosulphonate, the content is varied as 0.5, 1, 1.5, 2 and 2.5%. The dosage

Table 2 Properties of lignosulphonates used in the study

Property	Value	
	Sodium lignosulphonate	Calcium lignosulphonate
Sodium (%)	9	–
Calcium (%)	–	6.3
Dry solids (%)	95	93
pH	6	7
Bulk density	500 kg/m^3	500 kg/m^3
Particle size	< 300 nm	< 300 nm

Table 3 Curing given for various tests

Property	Curing (days)
Atterberg limits	28
Compaction tests	0
Unconfined compressive strength test	1,7,14 and 28
Unsoaked California bearing ratio test	7 and 28

range of the additives was selected by trial and error method. The optimum amount of sodium lignosulphonate and calcium lignosulphonates required to stabilise Kuttanad soil is also found out.

3.1 Sample Preparation

The collected soil is air dried and broken down before passing it through sieve with 4.75 mm opening size for standard proctor compaction test and unsoaked CBR test. For UCC, Atterberg limits and other soil properties the soil are sieved through 425 micron sieve. Required amount of lignin and water (OMC of the soil stabiliser mix as given in Tables 4 and 5) is thoroughly mixed with the air-dried soil to conduct the tests.

3.2 Curing

Stabiliser soil samples are sealed by vinyl bag and placed in water for curing before conducting all the tests. The curing period for different tests is presented in Table 3. Compaction tests were conducted just after mixing without any curing as done in the field so as to improve the strength characteristics for long term.

3.3 Testing Procedure

All the tests are conducted as per IS specifications.

- Liquid and plastic limit test: IS 2720 (Part 5)–1985
- Shrinkage limit test: IS 2720 (Part 6)–1972
- Compaction test: IS 2720 (Part 7)–1980
- Unconfined compression test: IS 2720 (Part 10)–1991
- Unsoaked California bearing ratio test: IS 2720 (Part 16)–1987.

4 Results and Discussion

4.1 Compaction Characteristics

Standard proctor compaction tests were conducted to study the effect of lignosulphonates on compaction characteristics of Kuttanad soil. From Fig. 1, it can be seen that with an increase in the percentage of sodium lignosulphonate (SL), maximum dry density (MDD) increased from 1.6 to 1.7 g/cm^3 at 5% SL content, this is due to the cementing ability of lignosulphonates which not only binds the particles but also fills the pores. After 5% with further increase in SL content, MDD decreased this may be because more soil is replaced by finer lignosulphonate. Similar trend was shown by calcium lignosulphonate stabilised soil. Figure 2 shows the compaction

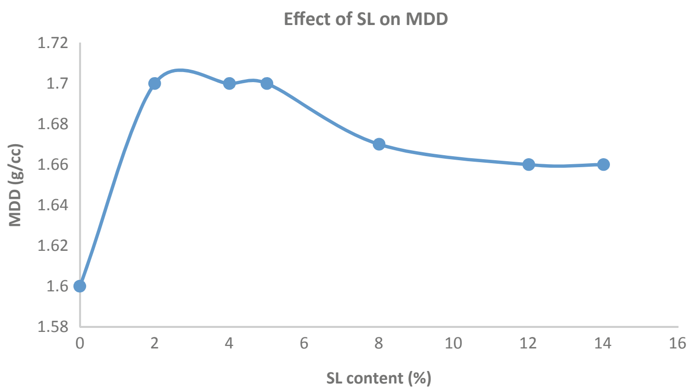

Fig. 1 Compaction characteristics of SL stabilised soil

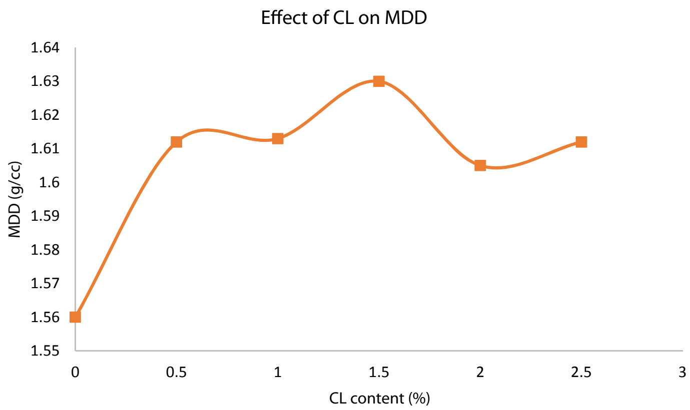

Fig. 2 Compaction characteristics of CL stabilised soil

Table 4 MDD and OMC values of SL stabilised soil

SL content (%)	MDD (g/cc)	OMC (%)
0	1.6	27.0
2	1.7	23.0
5	1.7	19.7
8	1.7	18.5
12	1.7	18.0
14	1.7	17.0

Table 5 MDD and OMC values of CL stabilised soil

CL dosage (%)	MDD (g/cc)	OMC (%)
0	1.60	27.0
0.5	1.61	22.7
1	1.61	20.9
1.5	1.63	22.4
2	1.60	17.4
2.5	1.61	18.7

characteristics of calcium lignosulphonate (CL) stabilised soil. Stabilisation by CL increased the MDD from 1.6 to 1.63 g/cm^3 at 1.5% CL content. Tables 4 and 5 show the MDD and OMC values corresponding to each dosage of sodium lignosulphonate and calcium lignosulphonate, respectively.

4.2 *Effect on Consistency Limits*

Figures 3 and 4 show the effect of sodium and calcium lignosulphonate on the plasticity and shrinkage characteristics of Kuttanad soil. It can be observed that addition of SL decreased the liquid limit by 20% and increased the plastic limit by 8%. It increased the shrinkage limit by 55%. In the case of CL stabilisation, the liquid limit of stabilised soil decreased by 17% and plastic limit increased by 50%. Shrinkage limit of CL stabilised soil increased drastically over 100%. Maximum improvement in consistency limits is obtained at 5% in the case of sodium lignosulphonate and 1.5% in the case of calcium lignosulphonate. The improvement in consistency limits may be due to the change in the particle size distribution. Table 6 shows the particle size distribution of stabilised soil determined by conducting hydrometer test on 28-day cured samples. Shrinkage limit of stabilised soil is found to be more than the plastic limit, such anomalous behaviour of soil is shown by soils were clay fraction is very low, and soils with relatively uniform or poor gradation that does not lead to denser packing and, hence, higher shrinkage limit results. The shrinkage limit obtained for such soils does not represent the boundary between semi-solid and solid states of consistency [4]. In lignosulphonate stabilised soil, the clay fraction got

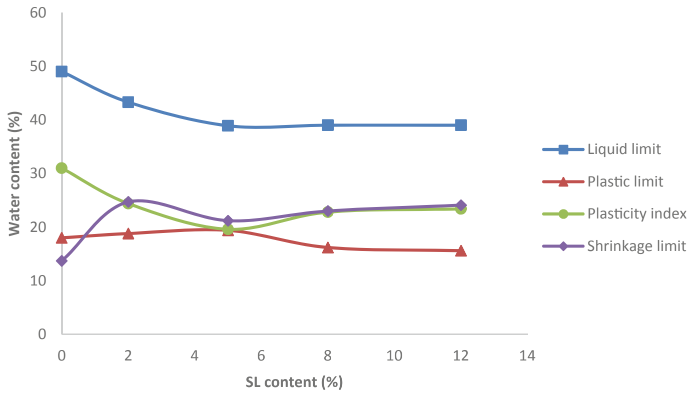

Fig. 3 Plasticity characteristics of SL stabilised soil

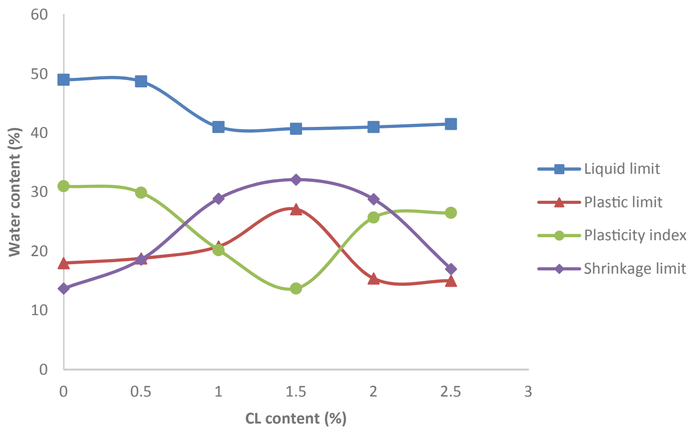

Fig. 4 Plasticity characteristics of CL stabilised soil

Table 6 Results of hydrometer analysis

Fraction	Kuttanad soil	CL stabilised soil	SL stabilised soil
Clay (%)	22	11	15
Silt (%)	76	71	77
Sand (%)	2	18	8

reduced and silt fraction was high compared to sand and clay fraction, thus, leading to a loose packing and shrinkage limit increased compared to plastic limit.

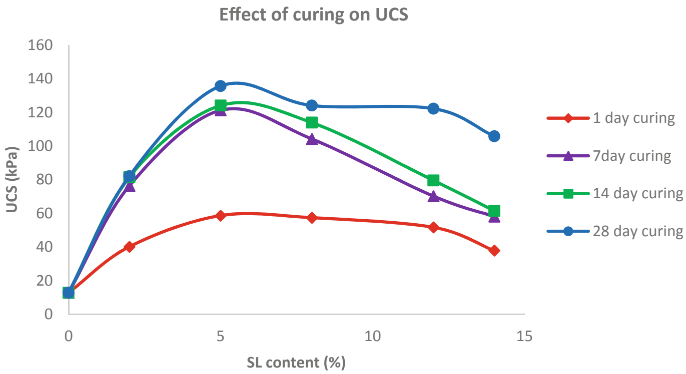

Fig. 5 Effect of curing on UCS of SL stabilised soil

4.3 Unconfined Compression Strength Test

Figures 5 and 6 show the effect of curing on unconfined compressive strength (UCS) of sodium and calcium lignosulphonate stabilised soil. From the figure, it is clear that curing has a significant influence on the strength mobilisation. Lignosulphonate stabilisation increased the strength 10 times as that of untreated soil after 28-day curing. The maximum strength is obtained at 5% SL content conforming that the optimum sodium lignosulphonate for stabilising Kuttanad soil is 5% and the optimum CL content required for stabilising Kuttanad soil is 1.5%. The increase in UCS may be due to the formation of lignin-based cementitious compounds which coats the soil particles and bind them together [3].

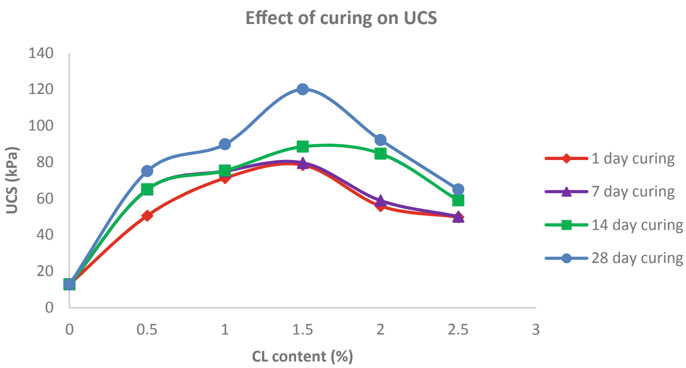

Fig. 6 Effect of curing on UCS of CL stabilised soil

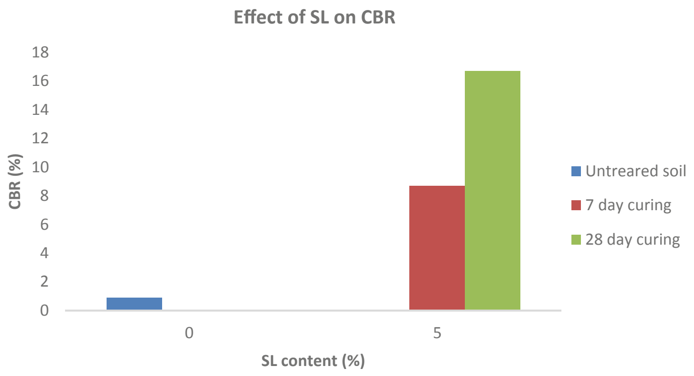

Fig. 7 Variation in CBR of soil treated with SL

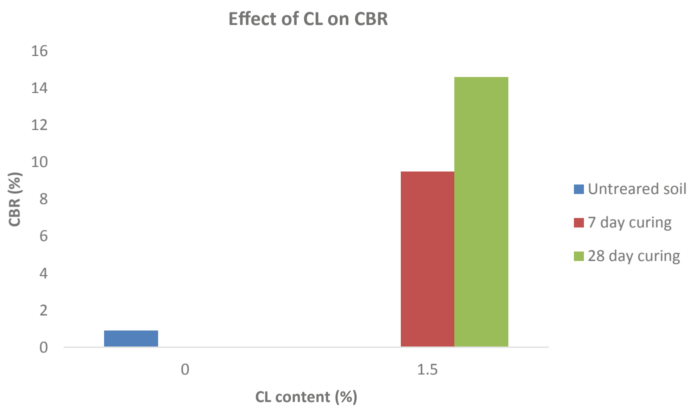

Fig. 8 Variation in CBR of soil treated with CL

4.4 California Bearing Ratio Test

Lignosulphonate stabilisation improved the load penetration behaviour of stabilised soil, thus, increasing the CBR value, and thus making it ideal for subgrade stabilisation. Curing period has a great influence on the CBR value. Figures 7 and 8 show the variation in CBR values after stabilisation with sodium and calcium lignosulphonates, respectively.

5 Comparison

- Calcium lignosulphonate improved the consistency limits better when compared to sodium lignosulphonate.

- Sodium lignosulphonate improved the strength characteristics of Kuttanad soil more when compared to calcium lignosulphonate

6 Conclusion

- The improvement in strength characteristics of the lignin stabilised soil is due to the cementing ability of lignosulphonates which binds the soil particles together and reducing the void space.
- Curing plays an important role in the strength improvement of lignin stabilised soil.
- The change in Atterberg limit is attributed due to the decrease in clay fraction of lignin stabilised soil.
- For different parameters, calcium and sodium lignosulphonates performed better. But since both the additives meet the basic requirements of a stabiliser, both could be effectively used in soil stabilisation.

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Efficacy of Almond Shell Ash Inclusion on the Geotechnical Behavior of Lime Blended Kaolinitic Soil

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Abstract The current study focuses on studying the effect of almond shell ash (ASA) inclusion along with lime in addressing issues related to strength and volume change behavior of a typical problematic soil. Experiments were carried out on both natural as well as amended soil mixtures having varying percentages of lime and ASA (added based on dry weight of soil). The addition of ASA reduced the plasticity characteristics of selected kaolinite rich clay. Of all the mixtures, the mixture containing 1% ASA and 0.5% lime exhibited maximum optimum moisture content (OMC) and lowest value of maximum dry density (MDD). Upon curing to 28 days, the same mixture exhibited the unconfined compression strength (UCS) value 76 kN/m^2. The results pertaining to UCS values indicated that the rate of gain of strength increased proportionally up to 1% ASA addition, followed by which there was a consistent decrease in the rate of gain of strength even in the presence of lime and is attributed to pore saturation with ASA. The addition of lime aids in better binding between non-plastic ASA particles and kaolinite-rich clay which was confirmed by scanning electron microscopic (SEM) images and X-ray diffraction (XRD) studies. SEM images confirmed the presence of agglomerated particles with homogenous mass exhibiting reduced pore opening size for mixture containing both ASA and lime.

Keywords Almond shell ash · Unconfined compression strength test · Scanning electron microscopy · X-ray diffraction

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1 Introduction

Globalization has led to the development of ideas and innovations to meet the demands of ever growing world, and technology has actuated toward utilizing materials of mere negligible importance for achieving comfort in many engineering spheres. Soil poses many intimidating situations when exposed to loads, making soil engineering open its approach to improve engineering properties by different options, both in materials and methods to assure improvement in the strength. The overall intention would be to make the soil capable in withstanding the loads or any kind of situations prone. Since the world is moving toward sustainable methods of development, the use of easily available materials for stabilization of soil is proposed. The present work is an attempt to use almond shell ash as a soil stabilizer.

Since the use of ash in the stabilization is widely proclaimed and since the ash has got cementitious properties, the use of fly ash, rice husk ash, wood ash, saw dust ash, groundnut shell ash, municipal solid waste incinerator ash [1], waste paper sludge ash, etc., are some examples of ashes used for soil stabilization. Lime is a well-known material used for stabilization of soil, and lime as a soil stabilizer assures increase in strength after aging of soil lime mix. Lime is used as additional binder along ASA in the present study, moreover, lime is used as soil stabilizer since time immemorial [2]. Improvement in properties of the soil was observed when it treated with cashew nut shell ash blended with lime. Keeping the lime content, constant soil is treated with cashew nut shell ash, results showed that the increase in density, strength and CBR value is due to the formation of pozzolonic compounds formed during the reaction between lime and ash which stabilizes the soil matrix [3].

2 Materials and Methods

2.1 Soil

The soil sample was obtained from agricultural land in Chikbalapur district (13.3908678° N, 77.8980191° E) of Karnataka. The soil under study is kaolinitic with quartz and silica as major components. The soil was tested for the properties and the following parameters were obtained.

2.2 Almond Shells

The almond shells were procured from the state of Jammu and Kashmir, and the rigid shell covering the edible part of the almond fruit is the material taken for the study. Almond shell is a rigid material having hard fibrous texture and usually utilized to produce heat.

Fig. 1 Almond shells

Fig. 2 Almond shell ash

The almond shell ash was produced by exposing the crushed almond shells to a temperature of 800–850 °C in muffle furnace for four hours and the ash passing through 425 μ sieve was used as an additive in the present study (Figs. 1 and 2).

2.3 Lime

Lime being utilized in this work is procured commercially, since the use of lime with ASA is tested for its behavior and the reference to the work carried out on these materials prior to this attempt was not well established; hence, the quantity of lime used in this work is restricted to a maximum of 1.1%. The effectiveness of ASA needs to be ascertained with a small quantity of lime along this additive.

2.4 Methodology

The soil passing through 4.75 mm sieve was used for testing after all the degradable matter was removed. Index properties of the soil were determined in accordance with the respective ASTM standards [4–9]; the results of the same are presented in Table 1. The tests were conducted on both untreated and treated soil, and the proportion of soil and stabilizer is kept as soil, almond shell ash and 0.5% lime.

The oven-dried soil was spread on a non-perforated surface and the additives (ASA and lime) in the desired quantities were sprinkled over the maximum area of the dried soil, and eventually the dry soil was mixed rigorously to get homogeneous mix for conducting the tests. Targeted geotechnical properties, viz., maximum dry density, optimum moisture content, plasticity characteristics, and unconfined compressive strength were studied with the addition of stabilizer.

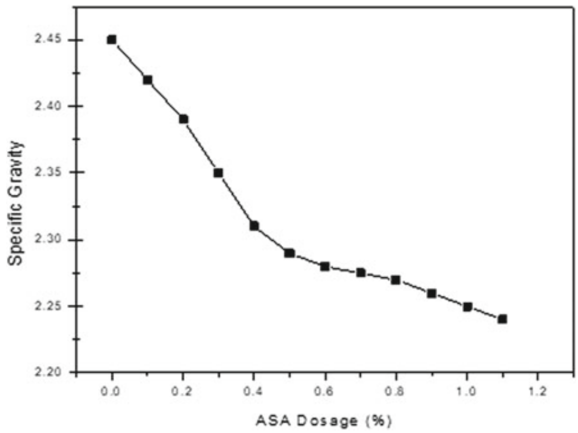
3 Results and Discussion

The addition of almond shell ash has shown a typical variation in geotechnical parameters of the soil. With the increase in % of ASA, the specific gravity of the soil mix was decreased (Fig. 3), this is attributed to the lesser density of ASA getting homogenized in the soil eventually decreasing the density of overall soil mass. It is proclaimed that

Table 1 Properties of soil

Property	Value	Standard specifications
Color	Red	
Natural water content/%	20.86	ASTM D4959(ASTM 2000a)
Specific gravity	2.45	ASTM D854(ASTM 2002)
Liquid limit/%	71.5	ASTM D4318(ASTM 2005)
Plastic limit/%	30.51	ASTM D4318(ASTM 2005)
Plasticity index/%	40.99	ASTM D4318(ASTM 2005)
Classification (USCS)	CH	ASTM D2487(ASTM 2006)
Optimum water content/%	23.49	ASTM D1557(ASTM 2012)
Maximum dry density (kN/m^3)	15.30	
Unconfined compressive strength (kN/m^2)	39.1	ASTM D2166(ASTM 2000b)

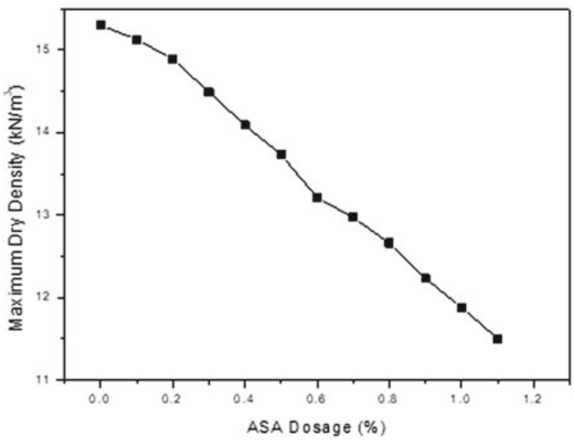
Fig. 3 Variation of specific gravity with increase in the ASA dosage



ashes like rice husk ash have a greater surface area which is another factor causing decrease in the specific gravity of the soil [10, 11]. ASA can also be considered having greater surface area and is found to reduce the specific gravity of soil with increase in percentage of ASA content. Figure 4 shows the variation in maximum dry density (MDD) and optimum moisture content with the variation in the ASA percentage. It is observed that decrease in MDD of soil amended with ash is due to decrease in specific gravity.

Figure 5 shows the amount of moisture required to get maximum dry density, which increases with the increase in the dosage of ASA in the soil. This is attributed to the properties of ash which demand greater amount of water for making the whole ash content to get homogenized with the soil or for the sake getting hydrated [1]; further, the more amount of moisture needed to make the soil mass compacted to the maximum since the voids in the soil mass increase with increase in the dosage of finer.

Fig. 4 Variation of MDD with increase in the ASA dosage



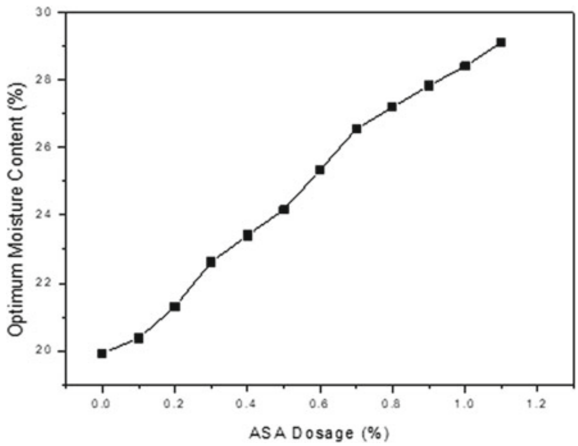


Fig. 5 Variation of OMC with increase in the ASA dosage

3.1 Effect of ASA on the Plasticity Characteristics of the Soil

Table 1 shows the plasticity index of the soil reduces gradually with the increase of the ASA percentage, since increase in the ASA dosage with 0.5% lime demands more water to achieve MDD. The decrease in plasticity index can be caused due to the increase in capacity of the soil amended with lime and ASA to hold greater amount of water in the voids [12], i.e., before the pore saturation level is nearing and when the gel is formed in the voids.

The extent to which the soil continues to be plastic is expressed by the plasticity index (PI), which is the difference between liquid limit and plastic limit [13]. Table 1 shows that the plastic limit of the soil increases from 30.51 to 57.31%, which is an increase of 26.74% when 1.1% by weight of ASA in comparison with PL of soil alone, this increase in plastic limit and reduction in LL resulting in the reduction in PI [10], and this scenario indicates the improvement of strength of soil. The decrease in the value of PI is a strong favor to the strength of the soil ascertaining the improvement in performance of soil against loads [14]. The addition of greater volume of water in the soil develops the risk of reduction in strength because the addition of water increases the value of plasticity index leading to shrinking of the soil mass and reducing the strength [15].

3.2 Effect of ASA on Strength of the Soil

The strength performance of the mixture of the soil and ASA along with lime is represented in Fig. 6. It can be observed that the UCS values are increasing with increase in the ASA % till the dosage of ASA is found to be 1% and with further

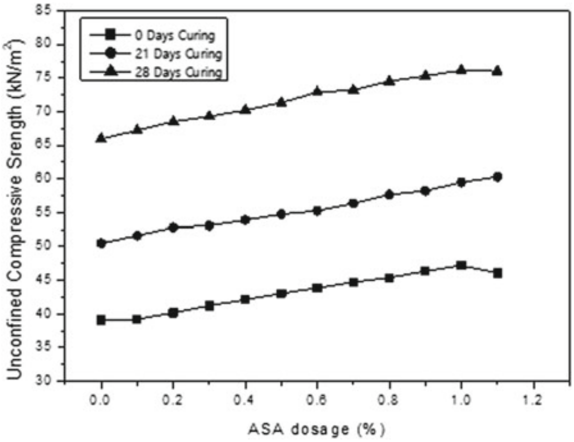


Fig. 6 Variation of UCS results with increase in the ASA dosage along 0.5% lime for different curing periods

increase in the ASA % the strength decreases. The UCS improved when 0.5% lime and 1% ASA are added to the soil (Table 2).

Further, the UCS specimen with age of 28 days portrayed the maximum strength, and the amount of increase of the strength for 28 days is found to be 64.78% when compared with 0 days curing and an increase of 26% strength is observed in the specimen aged 28 days in comparison with the 21 days old specimen for 1% ASA. Lime plays an important role in the bonding of the soil particles together with the ASA leading to a better performance of the amended soil. The increase in UCS value with increase of the ASA along lime is caused due to the formation of a gel like substance between the soil particles when the lime and ASA combine and start reacting and the gel depicts cementitious behavior strengthening the bonds between

Table 2 Summary of results

ASA %	Specific gravity	LL %	PL%	PI	MDD (kN/m^3)	OMC %
0	2.45	71.5	30.51	40.99	15.3	23.49
0.1	2.42	69.3	31.13	38.17	15.13	20.37
0.2	2.39	68.4	31.73	36.67	14.89	21.31
0.3	2.35	67.1	32.27	34.83	14.49	22.63
0.4	2.31	66.3	32.81	33.49	14.09	23.41
0.5	2.29	65.5	33.33	32.17	13.73	24.16
0.6	2.28	64.3	36.52	27.78	13.21	25.33
0.7	2.275	63.2	40.28	22.92	12.97	26.55
0.8	2.27	61.5	45.51	15.99	12.66	27.19
0.9	2.26	60.3	49.32	10.98	12.23	27.83
1	2.25	59.57	54.23	5.34	11.88	28.41
1.1	2.24	58.7	57.31	1.39	11.5	29.11

the soil grains. Further, the increase in the moisture added to the specimen also paves the way to the increase in the UCS value of the soil since the moisture helps the lime and ASA to react completely resulting in through involvement of the entire additive mass in the specimen. Increase in the ASA content, i.e., more than 1% leads to a decrease in the strength, the reason for this decrease in strength is due to the formation of weak bonds between the soil and the gel formed, and the excess formation of the gel leading to the decrease in the compressive strength of the soil is termed as pore saturation. Overall performance of the soil with the addition of the ASA portrays the usual results which are observed when soils are amended with other kinds of ashes.

3.3 Scanning Electron Microscopy Analysis

Scanning electron microscope (SEM) images of the soil, lime, and ASA mixture are shown in Fig. 7. Depicts three resolutions, i.e., 10 μm, 5 μm and 2 μm, respectively, of soil amended with 0.5% ASA and 0.5% lime, and the image with 2 μm resolution helps to distinguish the soil grains and the gel formed in the voids. In Fig. 7c, the soil particles indicated as "soil grain" surrounded by the smaller lumps indicated as "lime + ASA" explains the scenario, wherein the voids between the soil grains are filled by the gel formed by reaction of ASA and lime, and the gel seen in the SEM image appears as a cloudy mass partly filling in the voids paving the way for through bonding of the soil grains and eventually developing the strength confirming the claims made in the research. The maximum improvement in strength is observed when 0.5% lime and 1% ASA are added to the soil. Beyond this dosage, excess amount of additives imparts poor workability. Also, presence of non-reactive lime and ASA reduces the strength of soil.

Figure 8 is the SEM image with the resolution of 2 μm for the combination of soil with 1% ASA and 0.5% lime, and the image shows the soil grains covered completely by the gel formed by the ASA and lime; it can be observed that the image majorly depicts the cloudy mass in even at the sharp edges of the angular soil grains. And it even indicates the formation of perfect homogenized appearance assuring proper bonding of all the soil particles. Figure 9 shows the powder XRD of soil specimen with ASA to understand the mineral composition of mixture and it was found that the soil is predominantly kaolinite with minerals being alumina and quartz. Since the almond ash is an organic element, it was beyond the scope of XRD to detect it.

4 Conclusions

The study taken up gives the experimental results for the soil amended with 0.5% lime and ASA varying from 0.1 to 1.1% with an increment of 0.1% by weight of dry soil. The following conclusions are drawn:

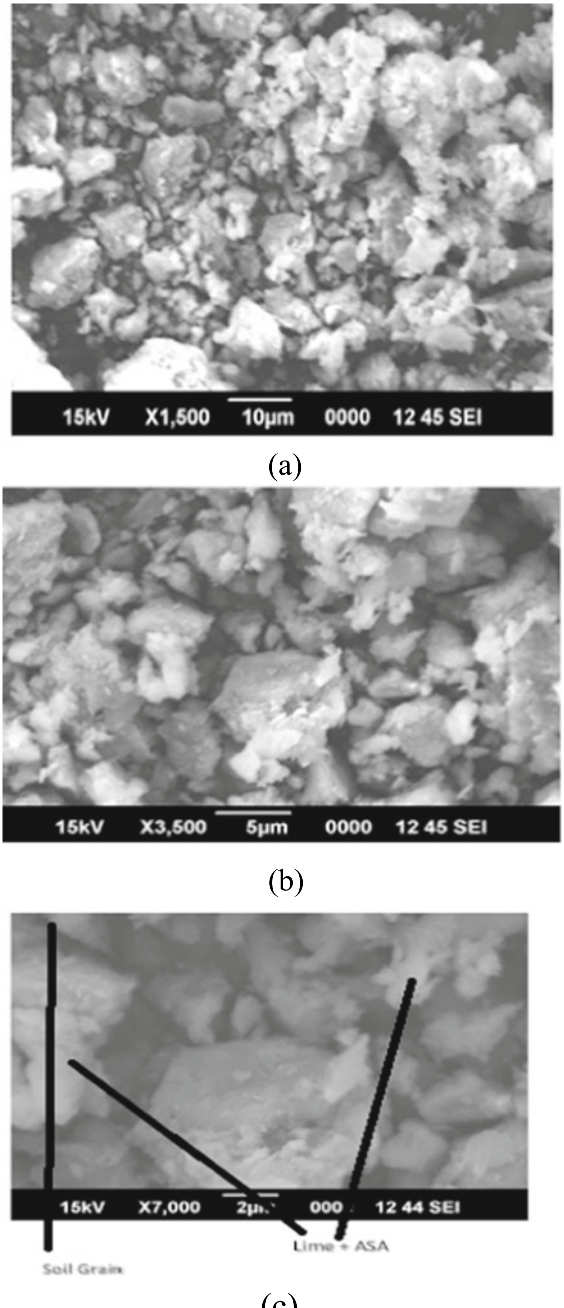


Fig. 7 **a**–**c** SEM images of the soil sample with 0.5% ASA and 0.5% lime

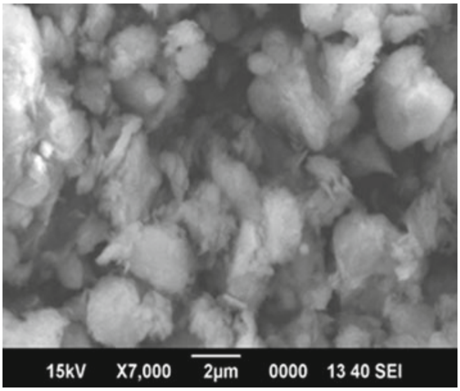


Fig. 8 SEM images of the soil sample with 1% ASA and 0.5% lime

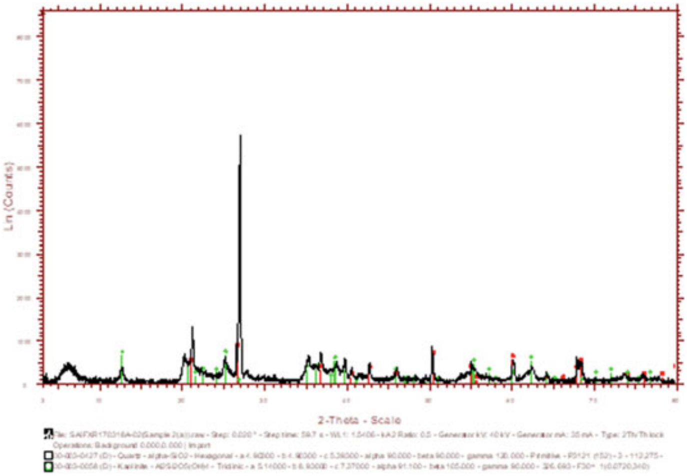


Fig. 9 XRD image of the soil sample with 1% ASA

- ASA is found to be a potential soil stabilizer meeting the demands of sustainable development, since the production of ASA is abundant in some parts of the world.
- 1% of ASA by dry weight of the soil mass along 0.5% of lime is found most suitable for obtaining the maximum strength and further addition of ASA resulted in decrease in UCS value.
- ASA even reduced the PI of the soil indicating its suitability to improve strength by increasing the plasticity of soil.

Use of ASA with lime can be a potential combination for soil stabilization in the countries where the production of ASA is abundant like California, Spain and Kashmir, and it may also achieve the mark of development of sustainable techniques avoiding issues concerned with disposal of ASA.

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Comparative Study of Strength Characteristics of Clayey Soil Mixed with Natural and Synthetic Fibers

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Abstract Soil reinforcement is nowadays an important method of improving ground by enhancing the properties of the in situ soil. Sometimes, soils available near the construction site is not strong and improving such soils will be beneficial for the project. Reinforcing such soils either by synthetic or natural fiber is a technique to improve the strength of such soils (Maity et al. in Proceedings of Indian geotechnical conference, Delhi, pp 285–288 (2012) [2]). However, each day millions of plastic bags of cement or jute bags of food grains are released to the market and disposal of such bags after use generally forms a part of solid waste requiring disposal. However, such bags can be gainfully used in improving soil properties by random mixing for compaction (Santoni et al. in J Transp Eng 127:96–104 (2001) [1]), thereby causing gainful disposal of such materials and simultaneously improving soil properties at a low cost. In view of large scale infrastructure development and wide spread rural road construction in India, such use may be very welcome. Aiming this an experimental program of testing local fine-grained soil, randomly mixed with plastic cement bags and jute bags cut in square shape of varying sizes and mixed with soil in various percentages was made for finding compaction characteristics and California bearing ratio values at optimum moisture content to estimate the improvement of these properties over those of virgin soil. Results of these tests are reported in this paper.

Keywords Re-use · Jute and cement bags · Random mixing · Ground improvement · Disposal

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1 Introduction

Large scale construction for the development of infrastructures and rural roads are being made in India for necessary growth and development for the country. For successful implementation, the optimum and efficient use of construction resources in the country is essential. Major efforts in this country are being directed toward the construction of roads bridges, railways and ports. Large quantities of fill materials are necessary for such cases. Unfortunately, available fill materials near the sites of constructions may not and actually do not fulfill the quality of good fill material in terms of strength and compressibility. In such cases, requirement arises to improve the properties of available local soils to a desired level by different possible methods. One of such methods which has been recommended for use in practice is by using synthetic and natural fibers of different lengths and percentages by weight in random mixing by Santoni et al. [1], Maity et al. [2], Maheswari et al. [3], Singh et al. [4], but in such cases, the cost of used fiber are quite high. On the other hand, millions of plastic cement bags or jute bags are being used for marketing cement or food grains all over the country. After using those materials, these bags form part of solid waste requiring disposal. However, no plan disposal system has been yet being developed. Kalita et al. [5], Saha et al. [6] and Konar et al. [7] have suggested that these worn cement bags or jute bags can be used in improving compaction characteristics of fine-grained soil so that the dual purpose of improvement of soil and disposal of used bags could be possible.

To check the feasibility of this methodology, an experimental program was undertaken to use such bag materials in square shape of different sizes mixed with locally available fine-grained soil in different percentages by weight for checking the compaction characteristics and California bearing ratio (CBR) values at optimum moisture content (OMC) of the mix soil where the mixing is done randomly.

2 Methodology of Work

Large numbers of cement and jute bags holding cement and food grains respectively are being introduced in markets everyday all over India. Such bags after consumption of holding materials become waste products and need disposal to maintain pollution-free environment. From such bags, samples of square shape are of three different sizes of 5 mm × 5 mm, 10 mm × 10 mm and 20 mm × 20 mm, respectively, were taken out. Materials of each size were randomly mixed with chosen soil in three different percentages by weight of 0.5, 1 and 1.5%. The samples of each size are randomly mixed with the dried powdered soil by chosen percentages by weight as stated above. Later, these mix soils were subjected to standard proctor test as per IS 2720 part 7, 1980 [8] to determine OMC and maximum dry density (MDD) of the chosen mixed soil. Then, the said mixed soil was compacted at OMC and subjected to CBR test in unsoaked and soaked conditions, respectively, as per IS 2720 part 16, 1987 [9]. The result of the above tests was utilized to see the efficacy of using the added square-shaped jute and cement bag portions of various sizes and percentages of mixing with the local soil on compaction characteristics and CBR value.

3 Materials Used

Soil: For the present investigation, soil was chosen from a construction area at Newtown near Kolkata in West Bengal. The above soil was collected during soil exploration in a bore hole at the site for residential building projects.

The soil was obtained from auger cutting around a depth of 6 m below the existing ground surface. The above soil was transported to soil mechanics laboratory at Meghnad Saha Institute of Technology, Kolkata for this experimental program. Index properties of the soil were determined as per codal provisions. Grain size distribution curve of the soil is shown in Fig. 1. Summary of geotechnical properties as determined in laboratory are given in Table 1. According to Indian Standard Soil

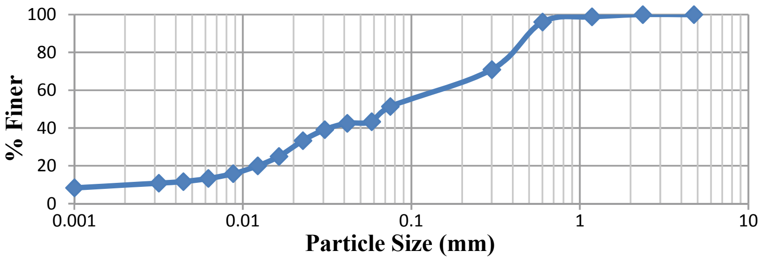

Fig. 1 Grain size analysis

Table 1 Physical properties of soil

Property	Value
Specific gravity	2.61
Liquid limit	46.9
Plastic limit	32.25
Plasticity index	14.65
Sand	48.61%
Silt	43.16%
Clay	8.23%
Maximum dry density	1.74 gm/cc
Optimum moisture content	16%
California bearing ratio (CBR) unsoaked	5.77%
California bearing ratio (CBR) soaked	4.87%

Classification System, "MI" stands for SILT of Intermediate Plasticity.

It is observed that OMC of the soil is 16% while MDD is 1.74 gm/cc. At OMC, the CBR value in unsoaked and soaked conditions is 5.77% and 4.87%, respectively.

Jute and Cement Bag: Jute bags were purchased from local shops while cement bags were collected from a construction site. These bags were utilized for providing samples of square shapes of various sizes as stated above. Jute bags are made from woven jute fabric and cement bags were made from impermeable plastic sheet.

The thickness of the cement and jute samples are 0.048–0.061 mm and 1.4–1.7 mm, respectively, having weight of 0.115 kg/m^2 and 0.5512 kg/m^2, respectively. Typical samples from jute bag and cement bag are shown in Figs. 2 and 3, respectively.

Fig. 2 Used cut pieces from jute bag

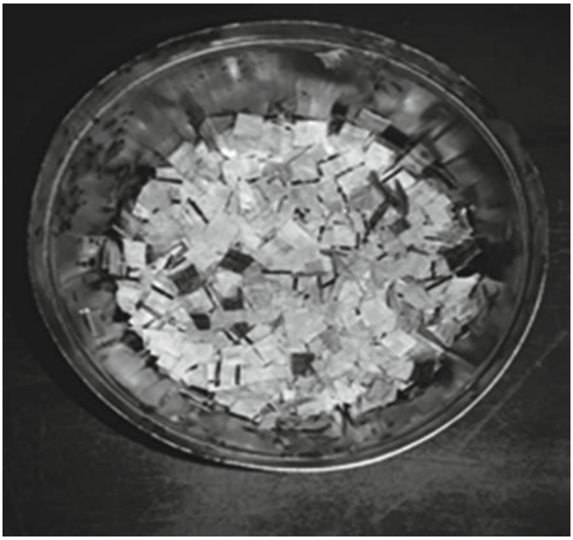

Fig. 3 Used cut pieces from cement bag

4 Result and Discussion

Experimental results of different tests conducted on the selected local soil mixed with different percentages by weight and sizes of square-shaped jute and cement bags cuttings mixed randomly with soil are described in this section. Initially, results of compaction tests are presented, and subsequently, the result of CBR test in unsoaked and soaked conditions on samples compacted at OMC is described.

4.1 Compaction Tests

Compaction tests were conducted on the local soil mixed with different sizes and different percentages jute and cement bag cuttings, and the values of optimum moisture content and corresponding maximum dry density were obtained from those tests.

4.1.1 The Variation of OMC

The values of OMC are plotted against percentage of jute bag cuttings mixed in Fig. 4 and against percentage of cement bag cuttings mixed in Fig. 5, for different sizes of the cutting.

From Fig. 4, it is observed that OMC increases with addition of jute bags cutting of any size and any percentages with respect to that of the virgin soil. However, the relative increase in value of OMC is maximum when the size of jute bag is 20 mm

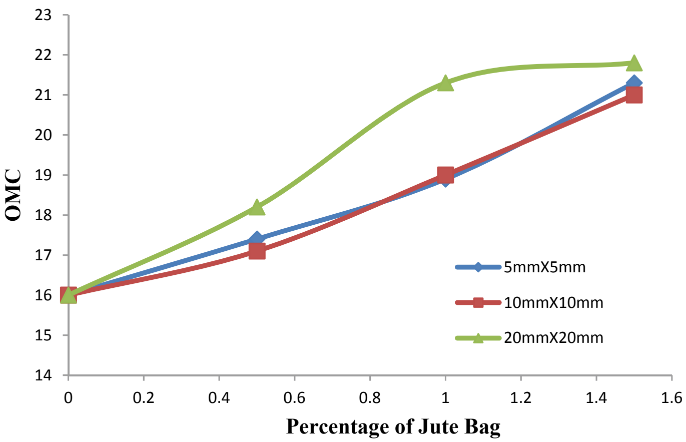

Fig. 4 OMC variation with jute bag % variation

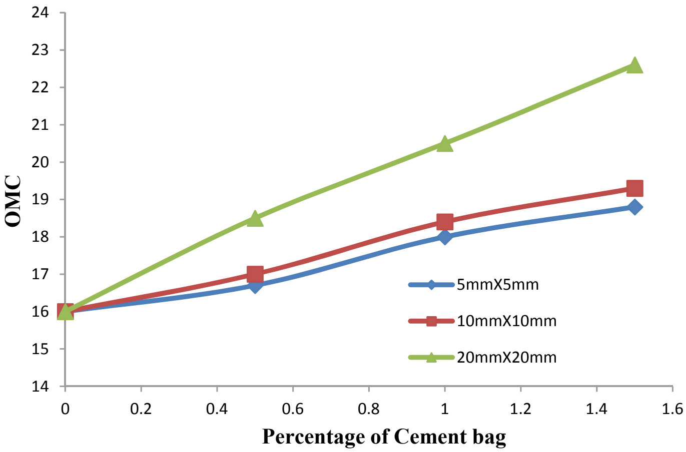

Fig. 5 OMC variation with cement bag % variation

× 20 mm. But the increase in the value of OMC reaches approximately same value for all sizes when percentages of mix are 1.5%.

Similar observations are seen from Fig. 5 for cement bag cuttings. Here also OMC increases with increase in percentages of cement bag cuttings for any sizes for random mixing. But maximum increase for any percentage is observed is largest at 20 mm × 20 mm size. Further, the value of OMC is not converging to same value for different sizes when percentage of mixing is increased.

The difference in behavior seen in Figs. 4 and 5 is probably due to the difference in behavioral pattern of jute and plastic cement bag materials. Plastic bags are totally impermeable where jute is a material which absorbed large amount of water and allow passage of water through its surrounding. When jute is introduced in soil during compaction, it will absorbed water itself and allow the required water movement for compaction of the surrounding soil needed for the compaction. Thus, the value of OMC will be increasing compare to that of soil, but this tendency will continue only up to certain value of percentages of jute bag cutting mix, for example, 1.5% in this case of study. But in case of addition of plastic cement bag cuttings, the additive being completely impermeable and it does not allow water movement through their body, and thereby for compaction of mixed soil requirement of larger amount of water to make compaction to be optimum and the need for excess water will gone increasing as size of impermeable additive goes on increasing. This is directly observed from Fig. 5 that the OMC is increasing with increase of both percentages of mixing and size of mixing.

4.1.2 The Variation of MDD

The values of MDD are plotted against percentage of jute bag cuttings mixed, as shown in Fig. 6, and against percentage of cement bag cuttings mixed, as shown in Fig. 7 for different sizes of the cuttings.

From Fig. 6, it is observed that MDD decreases with addition of jute bag cutting of any size and any percentages with respect to that of the virgin soil. However, the relative decrease in value of MDD is maximum when size of jute bag cutting is 20 mm × 20 mm. But the decrease in the value of MDD reaches maximum in all cases when percentage of mixing is 1.5%.

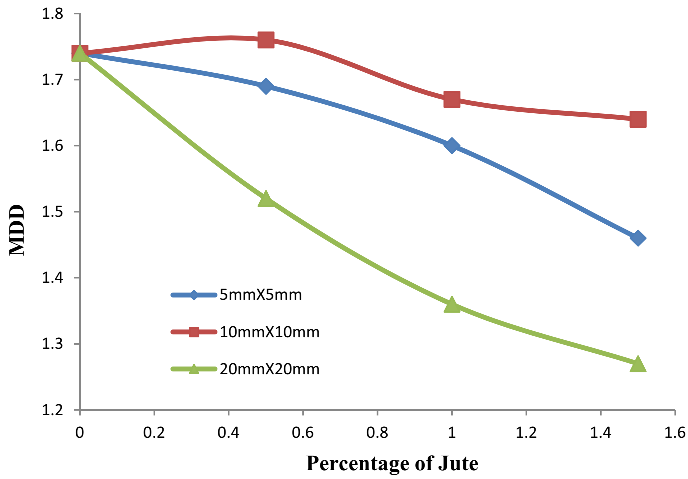

Fig. 6 MDD variation with jute bag % variation

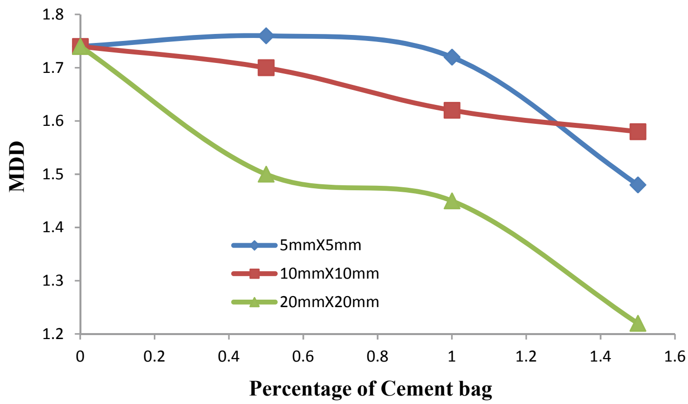

Fig. 7 MDD variation with cement bag % variation

Similar observations are seen from Fig. 7 for cement bag cuttings. Here also MDD decreases with addition of percentages of cement bag cuttings of any sizes for random mixing. But maximum decrease for any percentage is largest again for 20 mm × 20 mm size.

Materials of jute bag and cement bag are very lightweight compared to virgin soil. As a result, when such materials are mixed randomly with soil the density of composite material will be obviously lesser than that of the virgin soil. The decrease of density will further be affected because of variation in characteristics of the material mixed. Jute is a material which soaks water many times more than its volume and allow free passage of water through it. But the material of cement bag neither soaks water and more lightweight than jute. Effect of these factors during compaction of these mixed materials is reflected on the test results presented in Figs. 6 and 7.

4.2 California Bearing Ratio Test

CBR tests were conducted on the local soil mixed with different sizes and different percentages of jute and cement bag cuttings at corresponding OMC, both in unsoaked and soaked condition. The values of CBR obtained from these tests are described below.

4.2.1 The Variation of Unsoaked CBR

The unsoaked CBR values for the soil randomly mixed with different sizes of jute bag and cement bag cuttings are plotted against the percentage of mixing of such materials as shown in Fig. 8 for jute and Fig. 9 for cement bag cuttings.

From Fig. 8, it is seen that for jute bag cuttings, the CBR value for unsoaked condition shows some increase in value for lower size of cutting, namely 5 mm ×

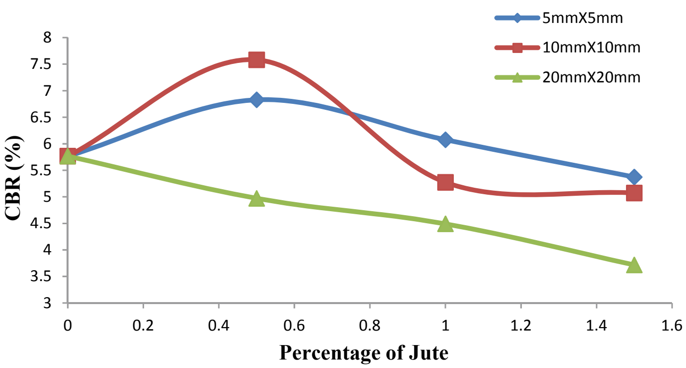

Fig. 8 CBR (unsoaked) variation with jute bag % variation

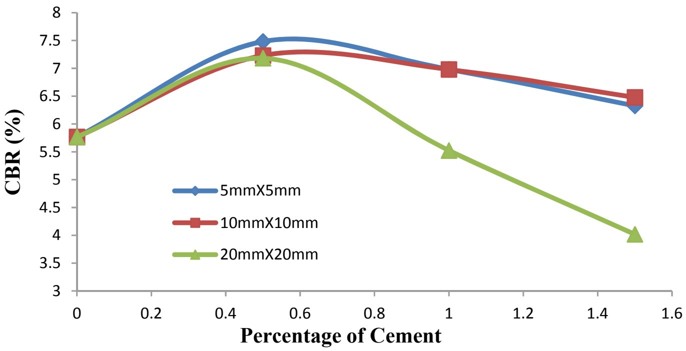

Fig. 9 CBR (unsoaked) variation with cement bag % variation

5 mm and 10 mm × 10 mm when percentages of mixing are 0.5%, but the value goes on decreasing with higher values of percentage mixing but for higher value of size, namely 20 mm × 20 mm unsoaked CBR value goes on decreasing from beginning.

However, from the similar plot for cement bag cuttings in Fig. 9, it is seen that there is nearly 15% increase in unsoaked CBR value for all the tested cutting sizes, but with further increasing percentage of mixing there is a decrease in value of unsoaked CBR, and the decrease is being drastic for higher size, namely 20 mm × 20 mm.

Thus, from the results of unsoaked CBR test on randomly mixed fine-grain soil with jute and cement bag cuttings of square shape having different sizes and mixed in various percentages, improved CBR values are obtained when size of the cutting is smaller and percentage of mixing is around 0.5%.

4.2.2 The Variation of Soaked CBR

The soaked CBR values for the soil randomly mixed with different sizes of jute bag and cement bag cuttings are plotted against percentage of mixing of such materials as shown in Fig. 10 for jute and Fig. 11 for cement bag cuttings.

From Fig. 10, it is seen that for jute bag cuttings, the CBR value for soaked condition shows some increase in value for lower size of cutting, namely 5 mm × 5 mm and 10 mm × 10 mm when percentages of mixing are 0.5%, but the value goes on decreasing with higher values of percentage mixing, but for higher value of size, namely 20 mm × 20 mm soaked CBR value goes on decreasing from beginning.

For the similar plot of cement bag cuttings as shown in Fig. 11, very small increase in soaked CBR value is observed for smallest size of cutting, namely 5 mm × 5 mm, but for other sizes, the most significant increase is observed at that percentage of mixing for other sizes. When percentage of mixing is further increased there is decrease in soaked CBR value.

Thus, from the results of soaked CBR test on randomly mixed fine-grain soil with jute and cement bag cuttings of square shape having different sizes and mixed in

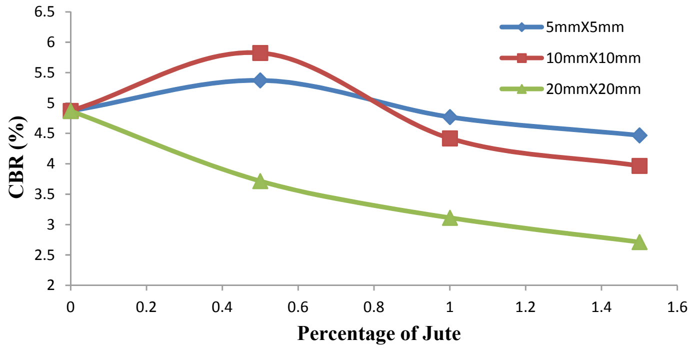

Fig. 10 CBR (soaked) variation with jute bag % variation

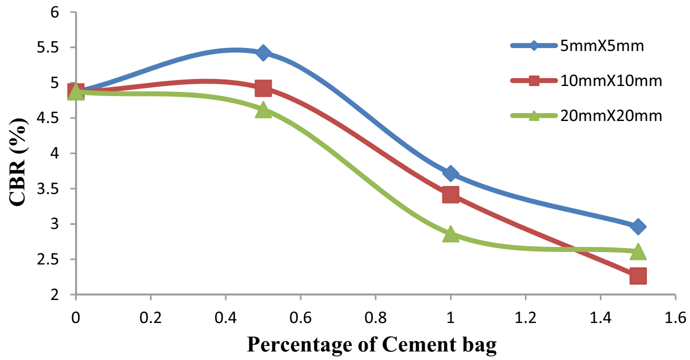

Fig. 11 CBR (soaked) variation with cement bag % variation

various percentages, improved CBR values are obtained when size of the cutting is smaller and percentage of mixing is around 0.5%.

5 Conclusions

Following conclusions may be drawn on the basis of the experimental program on local fine-grained soil randomly mixed with square-shaped pieces of different sizes and percentages from jute and cement bags which require pollution-free disposal.

1 The addition of any type of material used in this program for random mixing cause appreciable increase in OMC value compared to that of virgin soil. For jute materials, maximum increase in OMC was seen for the size of 20 mm $\times$ 20 mm when adding percentage by weight was 1.5%. Similar observation was seen when materials from plastic cement bag were used.

2 As a result of random mixing for both the type of materials, MDD values also show decrease in the value. For jute materials, maximum decrease was observed for 20 mm × 20 mm size when mixing percentage is 1.5%, but for cement bag material, maximum decrease was seen for 10 mm × 10 mm size when mixing percentage was 1.5%.

3 There is a general trend of increase in the CBR in unsoaked or soaked conditions when size of the added material is lower and percentage of mixing is around 0.5%. Maximum increase in the value of CBR for jute material is for the size 10 mm × 10 mm at a mixing percentage of 0.5%. Similar observation was seen for other materials also.

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Optimal Foundation System for a Storage Tank in Liquefiable Soil—A Case Study

Sampat Raj, V. K. Panwar, and Sanjoy Bhowmik

Abstract Storage tanks are constructed in all refinery complexes for the bulk containment of fluids at different stages of the refinery process. This paper presents the optimal foundation solution for a storage tank located at Bongaigaon, Assam. Subsoil at the present location consists of top 11 m loose to medium dense silty sand which is further underlain by dense sand with fines content less than 15%. The refinery site is located in the highly seismic prone zone and topsoil up to substantial depth is susceptible to liquefaction. In order to construct a storage tank in such type of soil, it is necessary to provide suitable foundation system for controlling liquefaction and to satisfy the performance requirement. Stone column is one of the ground improvement techniques to increase the bearing capacity and to reduce the total and differential settlements. Ground improvement using vibro stone columns is used to mitigate the liquefaction susceptibility and to minimize the differential settlement of tank foundation. The various aspects of subsoil conditions, design, construction methodology, quality control and hydrotest results are discussed in this paper.

Keywords Storage tanks · Optimal foundations · Ground improvement · Vibro stone columns · Hydrotest

1 Introduction

Tanks are an integral part of refinery and petrochemical industry which are used to store crude oil, petroleum intermediate and end products. The behavior of these flexible structures is closely associated with the strength and deformation characteristics of the subsoil. Any excessive total or differential settlement may cause distress of the tank shell, bottom plates, piping and nozzle connections, thus, jeopardizing the structural integrity of these structures.

In order to minimize the total and differential settlement, the selection of appropriate foundation system plays a key role. Generally, the foundation system is decided

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Table 1 Foundation quality criteria

Category	Maximum settlement at shell	Foundation quality
1	50 mm	Excellent
2	50–150 mm	Good
3	150–300 mm	Fair
4	Over 300 mm	Poor (soil treatment required)

based on the total estimated settlement under the tank shell. Foundation quality in respect of fixed/cone roof tanks is generally assessed as per the criteria given in Table 1 [1].

Ground improvement becomes imperative in case total settlement at tank periphery exceeds 150 mm (for floating roof tanks) and 300 mm (for fixed roof tanks).

An oil storage floating roof tank having 30 m diameter and 14.5 m height was envisaged at an operational refinery of Bongaigaon, Assam—a state located in north-east region in India which fall under zone of high seismic intensity. Evolving an appropriate foundation system for the proposed tank posed various geotechnical challenges. This includes prevailing subsoil conditions, i.e., the presence of loose saturated silty sands susceptible to liquefaction and higher groundwater table. The selection of optimal foundation system for tank in such type of conditions has involved a detailed techno-feasibility assessment.

2 Site Conditions

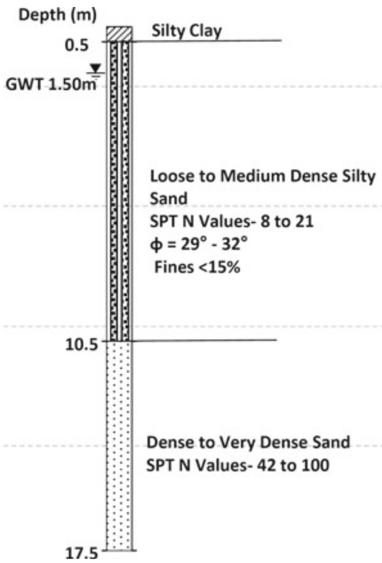
A detailed subsoil investigation program consisting of boreholes and laboratory testing was carried out to characterize the subsurface soil properties at the project site.

The soil profile is presented in Fig. 1, which depict that top 0.5 m soil is silty clay followed by loose to medium dense silty sand up to 10.5 m depth with SPT N value ranging from 8 to 21 and fines content less than 15%. This layer is underlying by dense to very dense sand mixed with gravels till 18 m depth. Groundwater table was encountered at 1.50 m below the ground level.

3 Liquefaction Susceptibility Analysis

Liquefaction is a phenomenon which occurs when a saturated or partially saturated soil substantially loses strength and stiffness in response to an applied stress such as

Fig. 1 Generalized soil profile



shaking during an earthquake or other sudden change in stress condition, in which material that is ordinarily a solid behaves like a liquid.

The proposed site lies in zone V as per the seismic zoning map of India (IS:1893-2016, Part 1) [2]. An earthquake magnitude of 7.5 was considered for analysis. Design groundwater table was considered at 1.0 m depth below finished ground level. The peak horizontal ground acceleration value for the site was taken as 0.36 g.

A liquefaction analysis was carried out based on the empirical procedure developed by Seed and Idriss [3]. For the analysis of liquefaction potential, cyclic stress ratio (CSR) and cyclic resistance ratio (CRR) were evaluated using the corrected SPT blow counts and fines content data for the silty sand. Figure 2 illustrates the induced CSR and CRR with respect to depth. From the expression FOS = CRR/CSR, it was concluded that soil was susceptible to liquefaction up to the depth of 10 m under an event of earthquake.

4 Foundation System for Tank

Based on the critical review of subsoil condition and performance criteria, various foundation options had been explored such as pile foundation, dynamic compaction, vibro stone columns, etc.

Foundation system pile with pile cap generally proves very costly. Foundations using dynamic compaction generate severe vibrations which may hamper the performance and the life of the existing nearby structures. In the operational plants, this method is generally not suitable due to safety-related issues.

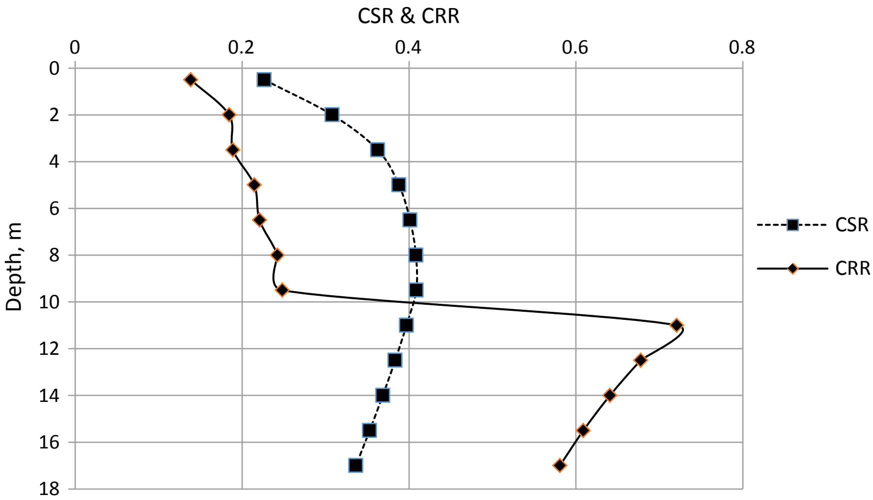


Fig. 2 CSR and CRR with depth (pre-stone column installation)

Ground improvement using vibro stone columns was considered as the most appropriate solution compared to other foundation solutions. It is a technique in which coarse grained materials is poured into ground using vibrating tools to form a vertical column. This technique improves the bearing capacity of the soil and decreases the total and differential settlements.

Vibro stone columns densify the area beneath the tank and also act as drainage which dissipate the excess pore water pressure in the event of earthquake and mitigate the liquefaction susceptibility. The introduction of stiffer material (stone) can potentially carry higher stress levels, and thereby reduce stresses in the liquefiable soil [4]. Consequently, CSR is reduced. The CSR reduction may be estimated using several approaches developed by Priebe [5], Baez and Martin [6] and Goughnour and Pestana [7].

5 Stone Column Design Considerations

The critical parameters for the design of most efficient vibro stone columns are finished diameter of the stone columns, stress concentration factor (n), pattern of installation and area replacement ratio (a_s). The spacing of the stone columns was decided based on the method suggested by Priebe [5] to mitigate the liquefaction potential of the subsoil. Angle of internal friction of the stone, φ_s, was taken as 42°.

The length of the stone column was decided in such a way that it extends through the liquefiable zone and rest in the competent soil strata which is 12 m from the finished ground level. Finished diameter of the stone column was 800 mm with triangular grid pattern since it yields the densest packing. Spacing of the stone columns

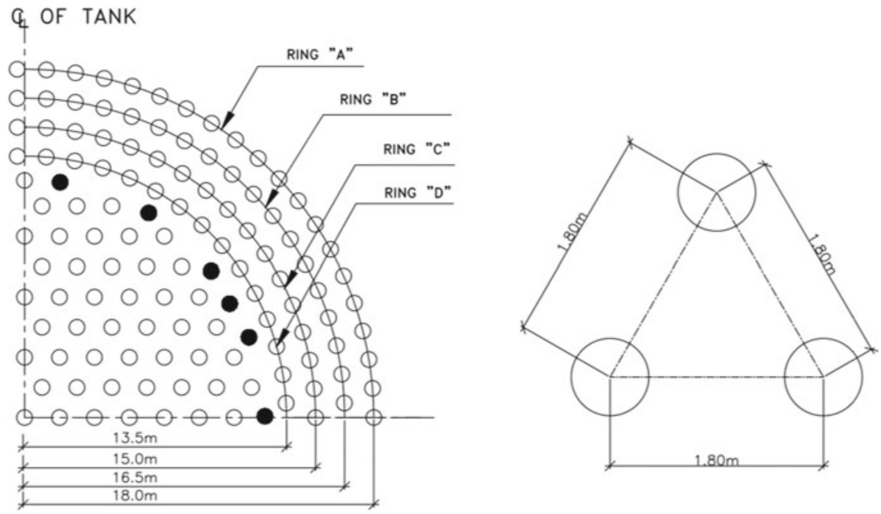


Fig. 3 Stone column layout in a tank quadrant

inside the tank periphery was 1.80 m and along the periphery of the tank was 1.50 m. Total nos. of stone columns were 451 which were decided based on the equivalent area of the stone column. Typical stone column layout is shown in Fig. 3.

Settlements corresponding to hydrotest loading have been estimated about 70 mm at tank center and 40 mm at tank shell. However, during seismic event, the tank would experience inordinate total settlement and large differential settlements that would severely affect the performance of floating roof tank.

Settlement after the installation of the stone columns was estimated by applying the improvement factor (n_o) as suggested by Priebe [8]. Post-treatment settlements at the center and periphery have been worked as about 38 mm and 21 mm, respectively.

6 Load Testing

An initial stone, column group test was carried out at a trial test site in the close vicinity of the proposed tank location to verify the stone column design. The initial stone column test was conducted on a group of three columns.

Additional columns were installed surrounding the test column to simulate the field conditions of compaction of the intervening soil. The load test plan developed for the initial stone column test is presented in Fig. 4.

The load test was conducted as per the Indian Standard codal provisions of IS 15284 part 1 [9] and the maximum load applied was 1.5 times the design load. The load-intensity-settlement behavior of the initial stone column tests is presented in Fig. 5.

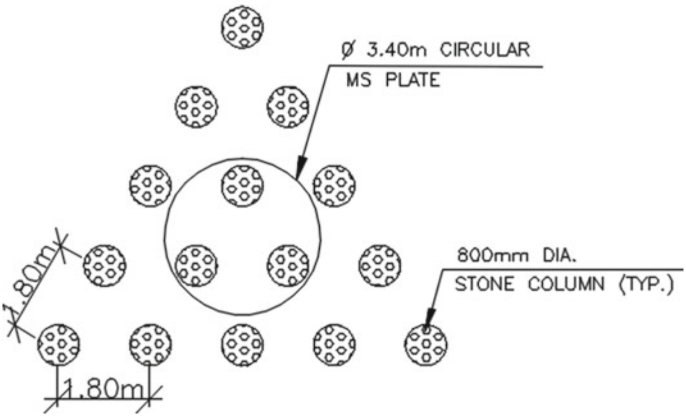


Fig. 4 Plan for group columns load test

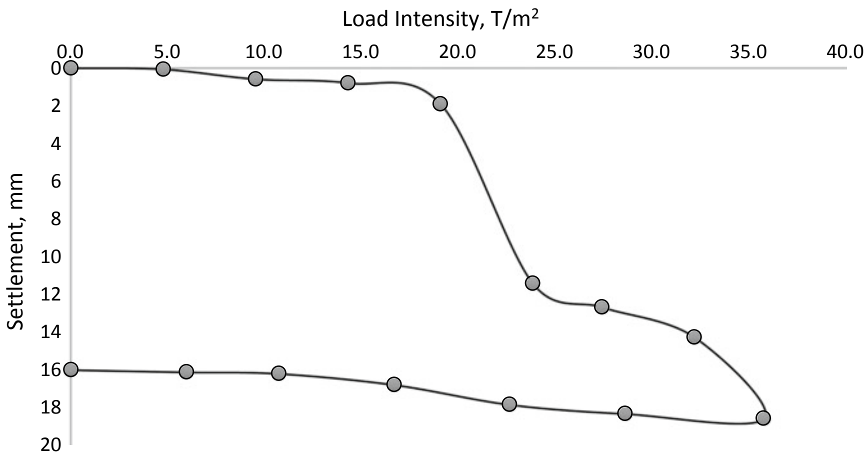


Fig. 5 Load-intensity-settlement curve for group column test

Total settlement at design intensity of 23 t/m^2 was observed to be 11.40 mm, and at 1.5 times, the design load (i.e., 34.5 t/m^2) was 18.60 mm for initial group column test. The observed total settlement at the design intensity was well within the settlement criteria stipulated by IS: 15284-2003, Part 1 (25–30 mm for group columns test) [9].

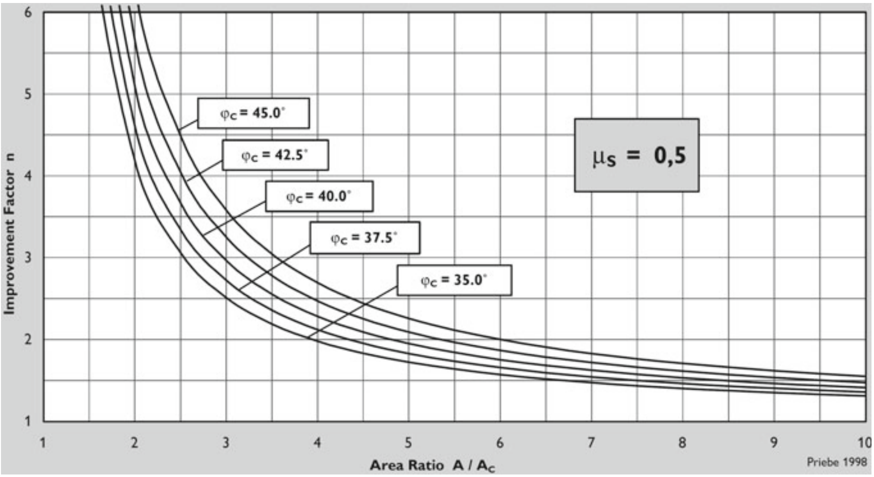


Fig. 6 Design chart for vibro replacement (after, Priebe [5])

7 Liquefaction Analysis Post-treatment

Post-treatment liquefaction analysis was performed according to design procedure by Priebe [5]. The improvement factor (n) which is a function of area ratio and friction angle of column material was calculated as per Fig. 6. The area ratio for the design column spacing was 5.58 and the improvement factor (n) was 1.98. The reciprocal of this improvement factor ($\alpha = 1/n$) was used to reduce the cyclic stress ratio (CSR) and evaluate the remaining liquefaction potential as per the empirical procedure developed by Seed and Idriss [3].

Figure 7 illustrates the induced CSR and CRR with respect to depth. This figure shows that the factor of safety is greater than 1.0 that proves the liquefaction potential of the soil is mitigated after installation of stone columns.

8 Hydrotest Results

Tank was hydrotested up to the design height and the settlement readings were recorded at 8 nos. of equidistant peripheral points. The settlement readings are presented in Fig. 8. The maximum actual settlement observed at the tank periphery was 17 mm which was well within the allowable settlement limits at tank periphery for floating roof tank.

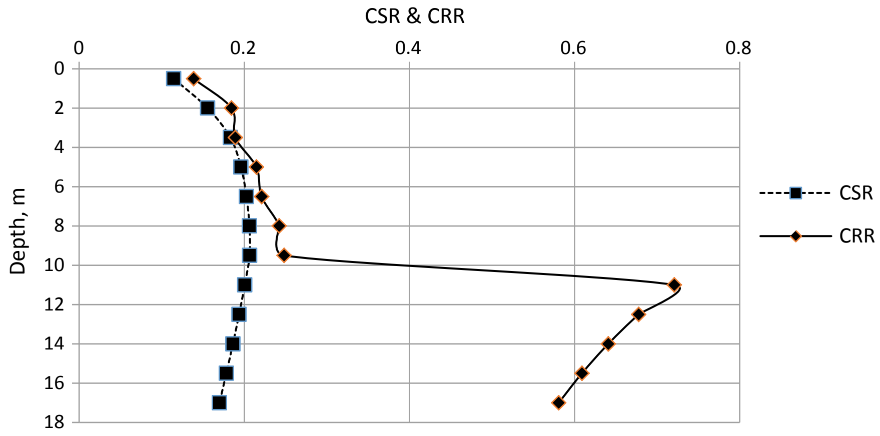
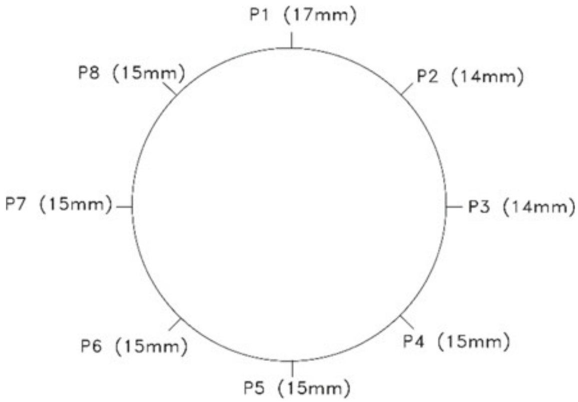


Fig. 7 CSR and CRR with depth (post-stone column installation)

Fig. 8 Observed peripheral settlements



9 Conclusions

The present study demonstrates that the ground improvement using stone column is the most optimal foundation solution for storage tanks in liquefiable soil conditions for an operational plant. Extremely close agreement between the predicted peripheral settlement (21 mm) and the measured settlement at the tank periphery (17 mm) at full hydrotest load confirms the adequacy and reliability of the design parameters and design procedure adopted.

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Response of Multilayered Stepped Geocell Reinforcement in Soil Structures

Haradhan Sarkar and Arghadeep Biswas

Abstract The conventional single layer of geocell reinforcement, as a measure of ground improvement technique, has widely been accepted to strengthen weak soil into a competent foundation layer. Since its inception, several researches are performed with such layer configuration and successfully applied in field. The performance factors of geocell systems have been improved with various laboratory/in-house investigations and their critical appraisals on parametric influences. However, it is noticed that most of the reinforcement volume (about one-third of the total with respect to loading size when full load transmission up to the geocell bottom is allowed) remain unused in case of the single-layer geocell system. Besides, compaction of soil at a greater depth of geocell experiences a great deal of difficulties. In addition, the thicker layers undergo considerable buckling at the top of geocell walls (situated just under the load) affecting localized settlement without any appreciated improvement. Therefore, replacing the single layer, with a multilayered stepped configuration, would mitigate this issue with an additional benefit in terms of material optimization. As of now, the mechanism of multilayered structure has not been developed explicitly which thrusts more emphasis on this issue to be addressed. In this study, the performance of multilayered system is envisaged to investigate through numerical simulation in Plaxis^{2D}. In doing so, initially, the work reported in Biswas (2019) is validated to confirm the parametric considerations. Being successful, it has further been used and extended for the present objective. A comparative performance of single with multilayered configuration is presented to confirm the usefulness of the proposed concept.

Keywords Geocell · Multilayer · Stepped configuration · Bearing capacity · Improvement

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1 Introduction

Geocell is one of the several types of ground improvement techniques favored by most of the geotechnical engineers of modern age. It has several advantages over conventional ground improvement techniques in terms of cost effectiveness, ecofriendly and ease of use, etc. Initiated through pavement application [1–5], several investigations are executed to find out the working mechanism of geocell by varying different parameters controlling the performance of geocell-reinforced structures associated in various civil engineering applications, such as embankments [6–10], railways [11, 12], and footing/foundations [13–15]. However, such studies were focused on single layer of geocell system (Fig. 1), while a multilayered system would be a viable option mitigating different difficulties identified [16]. Unfortunately, very few attempts (mentioned hereunder), so far, are reported on this configurations, and thus, the success and mechanism of multilayer geocell system has yet not explored properly.

Li et al. [17] has conducted a series of model experiments to compare the performance of reinforced embankment with multiple geocell layers and other parametric variations. They have noticed and concluded that the bearing capacity of embankment has increased with the number of reinforcement layers, and accordingly, the vertical and lateral displacements were decreased. Similar observation has also been reported by Tafreshi et al. [18]. They have reported the performance of a series of cyclic plate load tests on unreinforced and reinforced beds by multiple layers of geocell. The result indicated that the performance, in terms of load-settlement behavior, improved with the number of geocell layers [18]. Sarkar and Biswas [19] have reported an analytical study with multilayered geocell system and observed that multilayer of geocell system performs better in terms of reduced stress and settlement [20]. In present study, the authors have extended their previous work [19] with respect to effectiveness of multilayered geocell reinforcement (Fig. 1).

2 Material Characterization and Methodology

Biswas [16] addressed the various issues regarding single-layer geocell-reinforced systems and presented a comparison for different reinforcements on varying foundation configurations. Model tests were performed with a circular footing rested over unreinforced and reinforced sand layers overlying clay subgrades of different strengths ($c_u = 7$, 15, 30, and 60 kPa). The properties of materials used in the test program are reproduced in Table 1 along with calculated geocell properties used for present numerical analysis as per Latha and Rajagopal [10]. In the experimental study (Table 2), geocells were assembled in chevron pattern with the help of bodkin joint. The geocell materials showed the maximum tensile strength as 20 kN/m at 11% of axial strain.

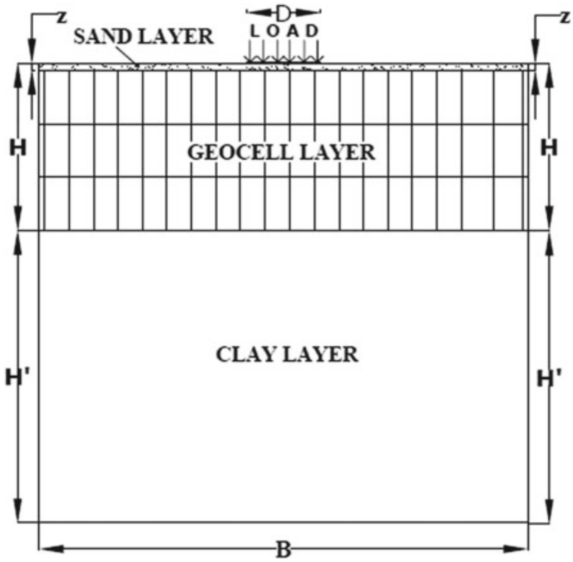

Fig. 1 Schematic diagram of the model test setup adopted by Biswas [16]

Table 1 Material properties

Material	Material properties	Values
Clay (CL)#	Max unit weight (kN/m^3)	17.3
	OMC (%)	19.7
Sand (SP)#	Max unit weight (kN/m^3)	16.43
	Friction angle (from triaxial test)(°)	40
Geocell*	Equivalent diameter (mm)	120
	Modulus of elasticity E_g (kPa)	92,719
	Poisson's ratio	0.3
	C_g (kPa)	92.78

#From Biswas [16]; *Derived as per Latha and Rajagopal [10]

Table 2 Details of test series and parameters considered in Biswas [16]

Foundation types	Test parameters
Homogeneous clay and sand bed	$c_u = 7$, 15, 30, and 60 kPa $D_r = 80\%$
Unreinforced sand layers on clay subgrades	$c_u = 7$, 15, 30, and 60 kPa; $D_r = 80\%$ $H/D = 0.63, 1.15, 1.67, 2.19$
Geocell-reinforced sand layers on clay subgrades	$c_u = 7$, 15, 30, and 60 kPa; $D_r = 80\%$ $H/D = 0.63, 1.15, 1.67, 2.19$; $Z = 0.1D$; $d/D = 0.8$

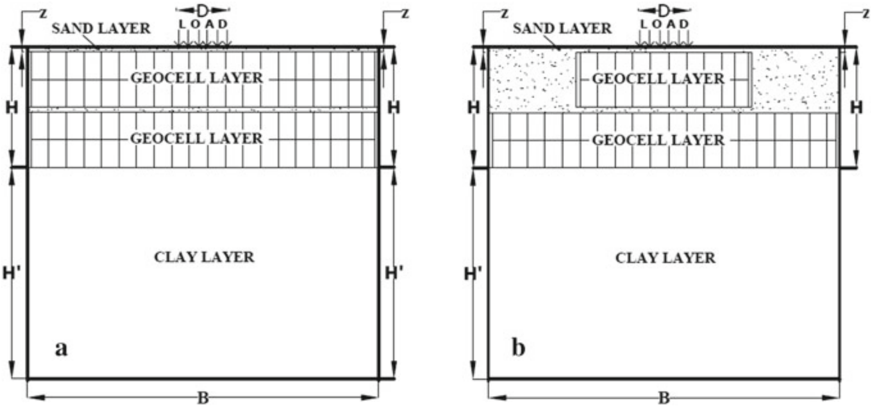

Fig. 2 Proposed configuration, **a** multilayer geocell, **b** multilayer stepped geocell

3 Multilayer Stepped Geocell System

Till now, single layer of geocell is used in most of the soil reinforcing systems. But studies have revealed that difficulties in filling arises where thickness of the geocell layer is high. As per the authors, it can be addressed if multilayer of geocell system can be used (Fig. 2a). Therefore, in this study, geocell-reinforced sand, having *H/D* = 2.19, overlaying clay subgrades is considered (as Biswas [16] has reported all the difficulties were prominently found for this configuration). In this study, two layers of geocell is placed, and a sand cushion of thickness z is considered in between the layers.

The conventional consideration in geocell reinforcing system is that the geocell is placed for full width of the area concern, instead as per the applied loading dimension only. Therefore, it has been found that in such configuration, most of the portion of geocell remain unused. Hence, it could be an effective measure if the geocell length is curtailed as per the load transmission. To address this issue and to find out the effective mechanism to provide multilayer stepped geocell, the configuration of geo-structure having *H/D* = 2.19 is split into two and top-layer of the geocell is cut at its 1/4th width from the edge (from both ends; Fig. 2b).

4 Simulation and Validation

4.1 Composite Geocell-Soil Layer

All the finite element analysis reported in this paper are performed using the PLAXIS^{2D}. In the analysis, axi-symmetric model is used where soils are modeled using a nonlinear elastic–plastic constitutive model following Mohr–Coulomb yield criteria and non-associated flow rule (as per Wulandari and Tjandra [20]). The geocell layers are modeled as an equivalent composite layer as proposed by Latha and Rajagopal [10]. In this method, the geocell-soil layer is simulated as a composite soil having higher cohesion with unaltered internal friction angle. The geocell-induced cohesion is termed as apparent cohesion (c_r) and calculated as Eq. 1. With the modified shear parameters, the equivalent stiffness of geocell (E_g) layer is calculated (Eqs. 1-3); where '$\Delta\sigma_3$' is the additional confining pressure due to membrane stress, 'ε_a' is the axial strain at failure, 'D_o' is the initial diameter, 'M' is secant modulus of membrane, 'K_p' is coefficient of passive earth pressure, and 'K_u' is the dimensionless modulus for the unreinforced sand.

$$c_r = \frac{\Delta\sigma_3}{2}\sqrt{K_p} \tag{1}$$

$$\Delta\sigma_3 = \frac{2M}{D_0}\left(\frac{1-\sqrt{1-\varepsilon_a}}{1-\varepsilon_a}\right) \tag{2}$$

$$E_g = 4(\Delta\sigma_3)^{0.7}\left(K_u + 200M^{0.16}\right) \tag{3}$$

A sample calculation for the input parameters are shown hereunder.

Let, ε_a is the axial strain at failure = 0.125, D_o is the initial diameter = 0.12 m, and M = Secant modulus at failure = 75 kN/m,

Hence,

$$\Delta\sigma_3 = \frac{2M}{D_0}\left(\frac{1-\sqrt{1-\varepsilon_a}}{1-\varepsilon_a}\right) = \frac{2x75}{0.120}\left(\frac{1-\sqrt{1-0.125}}{1-0.125}\right) = 86.9 \text{ kPa}$$

$$c_r = \frac{\Delta\sigma_3}{2}\sqrt{K_p} = \frac{86.90}{2}\sqrt{4.56} = 92.78 \text{ kPa}$$

$c_g = c_r + c = 92.78$ kPa, and,

$$\begin{aligned} E_g &= 4\sigma_3^{0.7}\left(K_u + 200M^{0.16}\right) = 4 \times 86.9^{0.7} \\ &\times \left(550 + 200 \times 380^{0.16}\right) = 92{,}719\text{kPa} \end{aligned}$$

Thus, following are the parameters (Table 3) used in the analysis to define the materials.

Table 3 Material properties and parameters

Material	Poisson's ratio	Undrained cohesion (kPa)	Modulus of elasticity (kPa)
Clay	0.45	7, 15	$600c_u$
Sand	0.3	0	13,000
Geocell	0.3	92.78	92,719

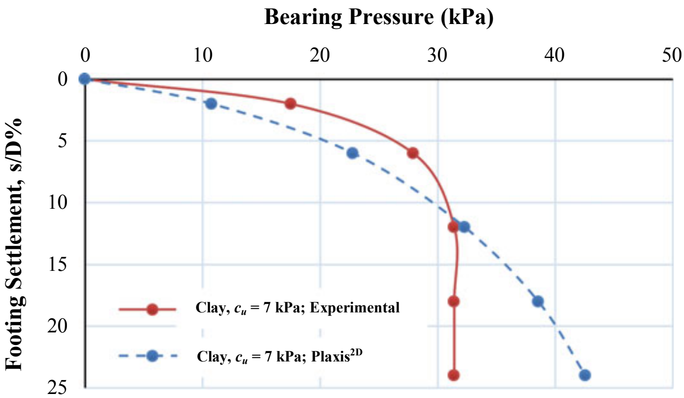

Fig. 3 Comparison between experimental (Biswas [16]) and numerical result (for homogeneous clay bed having $c_u = 7$ kPa)

4.2 *Validation*

Biswas [16] presented a comparative report of laboratory tests performed with circular plate (footing) rested on different surface foundations. The foundations are configured with unreinforced and reinforced sand ($D_r = 80\%$) layers of varied thicknesses overlying a wide range of clay subgrade, from very soft (c_u= 7) to stiff (c_u = 60 kPa). The reinforced layers are comprised of an interface geogrid, geocell, and combinations of geocell–geogrid of different thickness. It is reported that the performance of foundations improves with reinforcement superiority. However, the reinforcement benefits were reduced with an increase in clay stiffness and thickness of overlying sand layers. In this study, initially, the work of Biswas [16] is validated using PLAXIS^{2D} to get the confidence on the parameters to be considered in the numerical analysis. On getting the comparable agreements between the experimental and numerical results (Figs. 3, 4, 5 and 6) for the soil parameters, they are further used for present objectives. In modeling the geocell-soil layers, it is considered as a composite soil layer with modified shear parameters [10]. In Fig. 6, it may be noticed that responses of numerical and experimental observations are deferring in a considerable margin for geocell systems. However, as the present objective is to compare the performance of single- and multilayer (with and without stepped configuration) geocell-reinforced system, thus, this disagreement is neglected (as the performance will be evaluated through the ratios of bearing capacity and settlement levels).

5 Numerical Analysis for Multilayer Geocell System

On successful validation for soil parameters, the study has considered the geocell reinforced foundation for multilayered systems. Numerically, not much variation

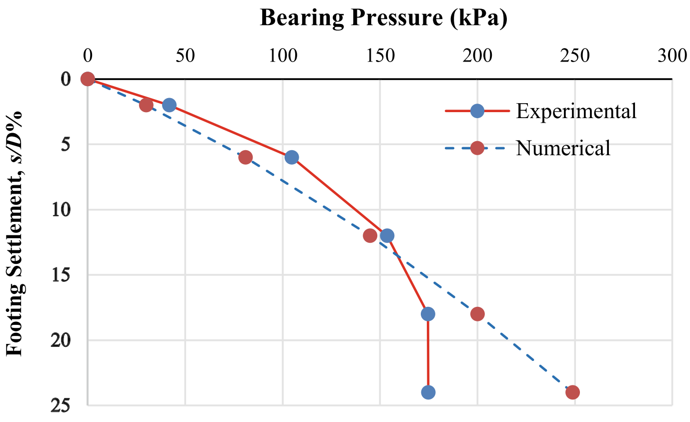

Fig. 4 Comparison between experimental (Biswas [16]) and numerical result (for homogeneous sand bed with 80% relative density)

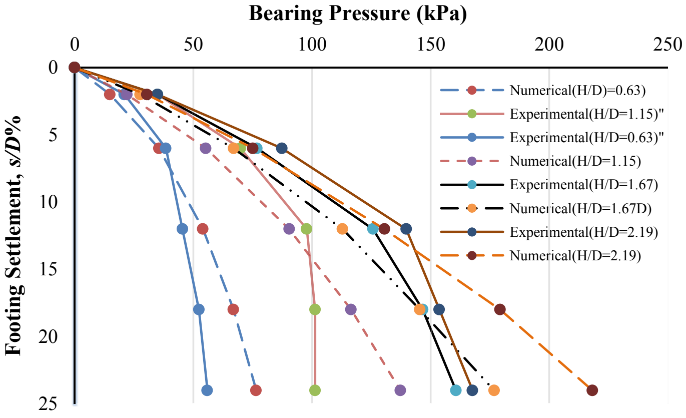

Fig. 5 Comparison between experimental (Biswas [16]) and numerical result (for unreinforced layered configuration for $c_u = 7$ kPa)

with respect to bearing pressure-settlement responses (for $c_u = 7$ and 15 kPa) is found (Figs. 7 and 8) for a single- and multilayer geocell system. This indicates that, practically, the multilayered geocell system should be fruitful for thicker geocells where compaction and buckling is a defining factor. Besides, it may also be noticed that the stepped geocell system have performed as good as the other two, whereas due to the curtailment, a huge savings on the material consumption is made. Thus, as per the objective of this study, it may be concluded that the stepped configuration would be much more effective than a conventional use of single-layer geocell without compromising the benefits (laboratory experiments, as the companion, are planned to be conducted as future scope of this study).

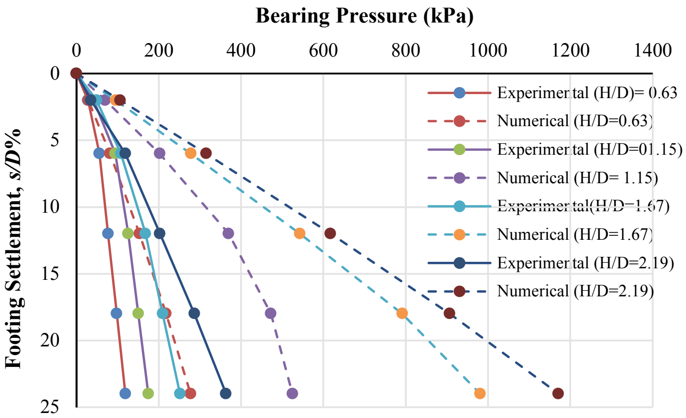

Fig. 6 Comparison between experimental (Biswas [19]) and numerical result for geocell-reinforced layer of different thicknesses for $c_u = 7$ kPa

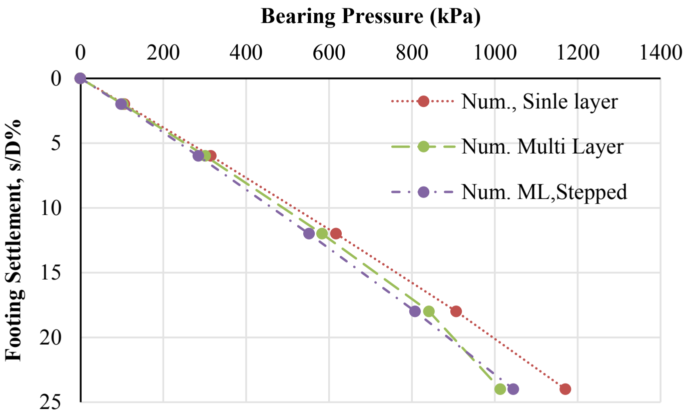

Fig. 7 Comparison between single-layer of geocell system, multilayer of geocell system, and multilayer stepped geocell ($c_u = 7$ kPa)

6 Conclusion

This study has numerically investigated the comparative performance of single- and multilayer geocell systems. Though, the study is in primary stage, however, a clear indication is noticed that a multilayer geocell system, with stepped configuration, would be a better consideration as compared to conventional single-layer geocell systems (Fig. 8). It is observed that for the selected configuration at higher deformation (settlement level), soil system collapses for a single-layer system; while for multilayer stepped geocell system, this phenomena has not been observed. However,

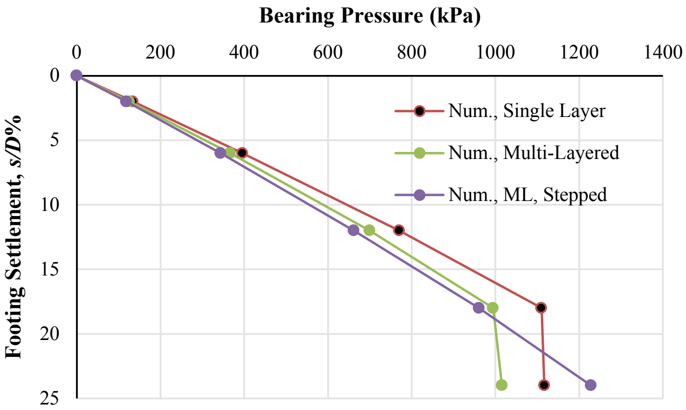

Fig. 8 Comparison between single layer of geocell system, multilayer of geocell system, and multilayer stepped geocell ($c_u = 15$ kPa)

a concrete conclusion can only be drawn after a validation program which can be performed through physical laboratory investigation.

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An Innovative Foundation Technique for Residential Building—Case Studies

B. Vani and P. V. S. R. Prasad

Abstract In recent years, rapid development of urban environment demands large number of residential units which are compounded with scarcity of suitable land. This compelled practicing engineers to find an innovative foundation technique to improve the unsuitable land which are technically feasible and economical. This paper presents case histories of similar conditions where ground improvement techniques using vibro stone columns was chosen to support residential buildings in weak soil strata. The foundation challenges such as bearing capacity, total and differential settlements, and mitigation of liquefaction were addressed. Various aspect of subsoil conditions, design aspects, construction methodology, and quality control measures are discussed.

Keywords Vibro stone columns (dry bottom feed method) · Innovative foundation technique · Post-monitoring results

1 Introduction

Soil improvement techniques are typically implemented as value engineering solutions to classical deep foundations or conventional soil replacement, for wide range of applications such as infrastructure projects, residential buildings, oil & gas facilities, transportation structures [roads, railways and airports], marine structures, power plant structures, and storage tanks.

One of the major functions of geotechnical engineering is to design, implement, and evaluate the feasibility of ground improvement schemes for unsuitable construction lands. In the last two decades, significantly new and advanced technologies and methods have been developed and implemented to assist the geotechnical engineer

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to provide innovative solutions for construction sites where poor soils exist. The selection of the most suitable and economical soil improvement method for each project depends on the soil conditions (soil type, depth of improvement), type and loads of foundations, project modalities in terms of cost and project schedule, and site conditions.

With reference to the previous publication "Optimal foundation solution for residential projects, Vani, B., Annam, M.K in: IGC Conference Proceedings, IISC Bangalore [1], the current paper explains, ground improvement using dry vibro stone columns as an innovative and economical solution for residential buildings are discussed with case studies.

2 Case Histories

2.1 Low Budget Residential Project at Manali

Project Background
Tamil Nadu Slum Clearance Board (TNSCB) proposed to construct a residential project of seven blocks (G + 3) with an area of 5400 m^2 at New Town, Manali in Chennai. The project site consists of soft to firm silty clay (intermediate to high plasticity) for top 11 m with SPT '*N*' <10 which is followed by very stiff clay and medium dense sand. The buildings rest on rafts, and when loading intensity of 90 kPa is imposed on weak ground ($N < 10$), they induce high settlements (>120 mm).

Alternative Foundation System
Vibro stone columns (dry bottom feed method) was selected as an innovative foundation to conventional piling method (since there is restriction to go with pile foundation due to environmental and safety issues) to reduce the settlements, to improve the bearing capacity of the soil (based on IS: 6403: Indian Standard code of practice for determination of bearing capacity of shallow foundations (Reaffirmed 2002) [2]) which satisfies performance criteria of the project. This project was completed within three months using vibro stone columns where the initially proposed pile foundation was six months. There is also a cost optimization when compared to piling and environmentally friendly solution compared to piling.

Design Approach
Based on site limitation and performance criteria, dry vibro stone columns with area replacement ratio 18% (with reference to IS: 15284 (Part 1): Indian Standard for design and construction for ground improvement-guidelines, Part 1: Stone Columns [3] and Priebe, H.J.: The design of vibro replacement. Ground Eng. GT 037-13 E [4] were proposed and executed, and the cross section of vibro stone columns is shown in Fig. 1. The quality of the project was ensured by observing the M4 records, conducting plate load tests, and post-performance settlement monitoring. The paper

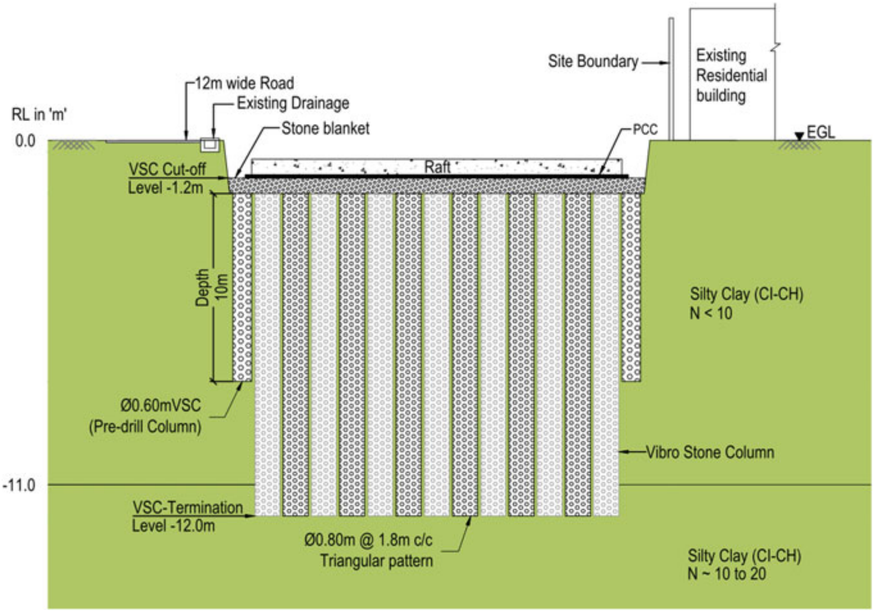

Fig. 1 Cross section of vibro stone columns

explains "Reinforcing of soft cohesive soils with stone columns Hughes, J.M.O., Withers, N.J.: Ground Eng. 7(3), [5] is also used as a reference for this project.

Post-monitoring Results

To understand the post-construction performance of the structures, it is planned to conduct field settlement monitoring up to 2 years of the life of structures. The superstructure load is increased from 0 to 90 kPa in 29 weeks and correspondingly predicted settlements (analytical method) increased from the 0 to 72 mm. Settlement monitoring results for 1.5 years are of 45 mm which is shown in Fig. 2 and almost settlement readings are saturated.

2.2 *Multistory Structure in Haryana*

Project Background

A large residential complex comprising of 13 blocks, of each G + 14 floor towers developed in 12 acres in Haryana state. The site consists of silty sand/sandy silt till 30 m depth from existing ground level. The percentage of sand varies from 35 to 70 throughout the depth. The project location falls under seismic zone IV with zone factor of 0.24 according to Indian seismic code (IS 1893: 2016), and it was required to address liquefaction mitigation and bearing capacity.

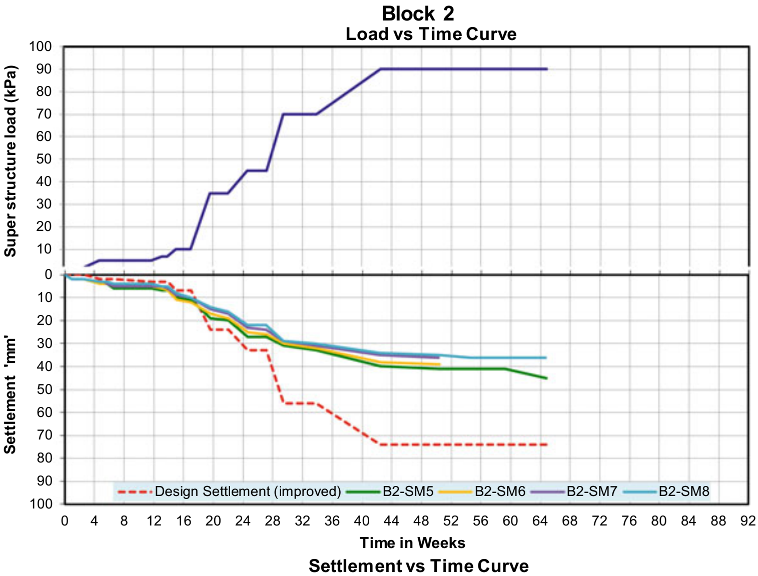

Fig. 2 Post-settlement monitoring data for low-budget residential project

Ground Improvement Using Vibro Stone Columns
Based on the critical review of the subsoil conditions and performance criteria of the project, vibro stone columns was proposed as alternate solution to address liquefaction mitigation and to satisfy the project performance criteria.

Design Approach
Area replacement ratio of 16% was proposed to mitigate the liquefaction potential, to enhance the bearing capacity of soil. Also, the quality of the project was ensured by conducting large size plate load tests.

Post-monitoring Results
Field settlement monitoring was monitored up to 2 years of the life of structures. The superstructure load is increased from 0 to 150 kPa in 47 weeks and correspondingly predicted settlements (analytical method) increased from the 0 to 97 mm. Settlement monitoring results for 2.0 years are of 60 mm which is shown in Fig. 3.

2.3 Residential Project "INFINITY" at Porur

Project Background
Urban Tree Infrastructure Private Limited proposed to develop a residential project comprises of 198 units (Stilt + 4 floors) with an area of 2.5 acres in Chennai. The top 6 m of soil profile comprises of thin layers of silty clay and sandy clay with

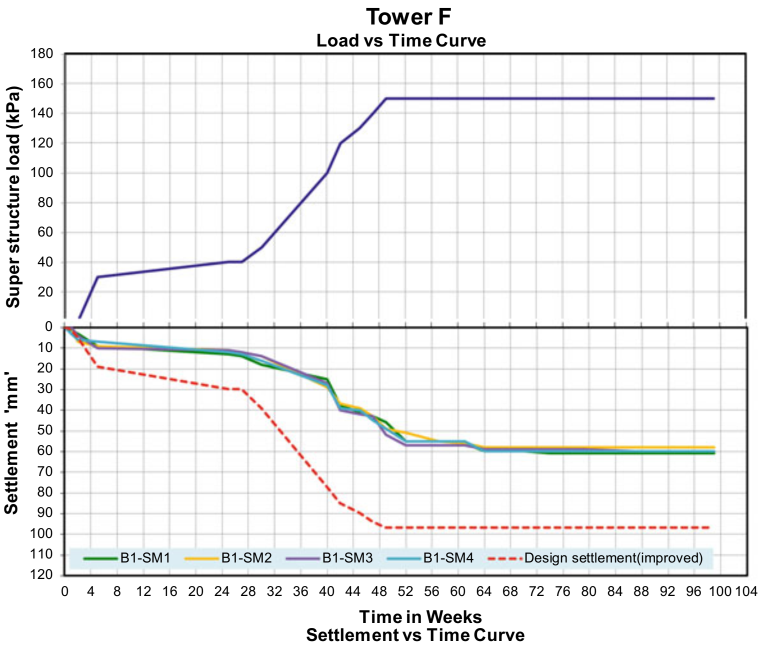

Fig. 3 Post-settlement monitoring data for multistory residential project

varying consistency. Larger settlements (>100 mm) were expected in case of raft foundation due to high structural loads which ranges from 75 to 85 kPa. This project was also read in reference with "Optimal foundations in soft ground: an innovative approach" pp. 459–470. ISBN: 978-981-09-7520-3 [6].

Innovative and Alternative Solution to Pile Foundation

Driven cast-in situ piles resting in hard clay layers below 25 m were adopted by client, and the construction of piles were stopped due to environmental issues. Considering the project boundary conditions, vibro replacement (stone columns with dry bottom feed method) was selected as a viable innovative method to reduce settlement and to satisfy performance criteria of the project. The selected method of ground improvement satisfied in addressing environmental issues, and the stone columns were installed by displacement technique (without removing any soil) which makes the environment would be comparatively clean and tidy. The ground improvement works were completed within 6 weeks (as against 6 months to that of pile foundations) that was made possible through effective project management.

Design Approach

Ground improvement using vibro stone columns (dry bottom feed method) with area replacement ratio of 18% is used as a treatment scheme. Typical cross section of vibro stone columns using dry bottom feed method adopted for the present project is illustrated in Fig. 4.

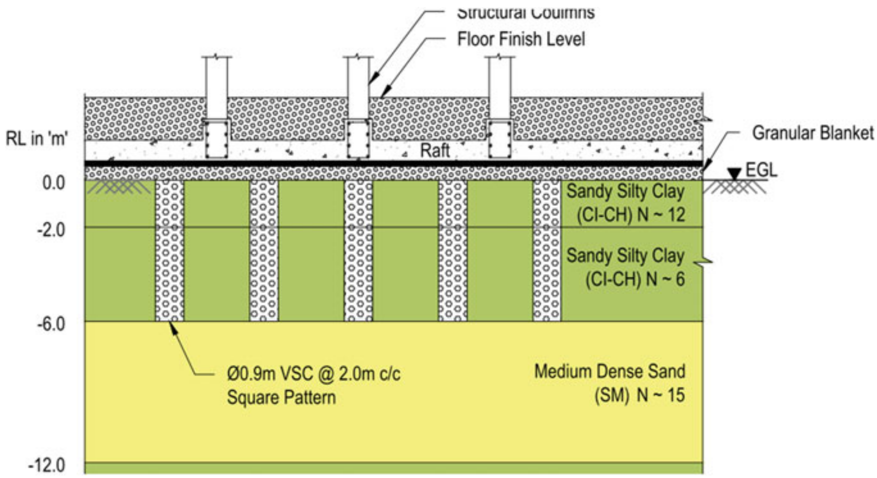

Fig. 4 Typical soil profile showing ground improvement

In order to measure and assure the quality of installed stone columns, real-time monitoring (M4 graph) was carried out in addition to routine single column load tests to ascertain the effectiveness of the design and performance of the ground improvement works.

Post-monitoring Results

The superstructure load is increased from 0 to 80 kPa in 20 weeks, and correspondingly, predicted settlements (analytical method) increased from the 0 to 64 mm (Fig. 5). However, the observed settlements are considerably less than the predicted settlements as well as the allowable settlements of 75–100 mm for raft foundations resting in clayey soils. It is suggesting that the long-term settlements will be of much smaller range than that was expected.

3 Conclusion

Vibro stone columns proved to be an innovative ground improvement solution to support residential buildings on weak soil deposits and best alternative to conventional piling foundation. It is also proven from the results of extensive monitoring that the required performance was achieved. In addition to improving shear strength and compressibility parameters, the ground improvement technique offered acceleration in the overall construction schedule and enabled the project to be completed within stipulated duration.

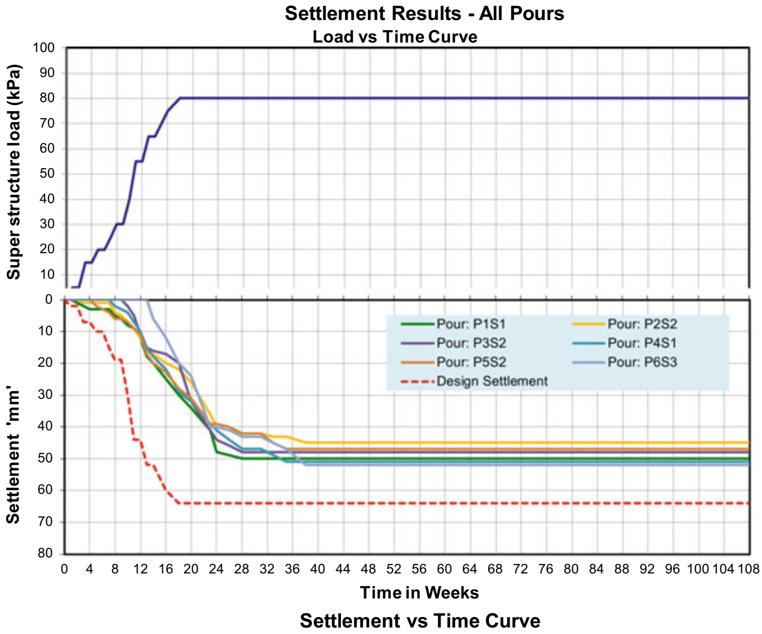

Fig. 5 Post-settlement monitoring data for INFINITY residential project

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Vibro Compaction Technique in Liquefaction Mitigation and Its Value Addition—A Case Study

C. Ramanathan and P. V. S. R. Prasad

Abstract India being a rapidly developing nation requires quality infrastructural developments almost in every field ranging from industries, transportation, education, etc. Hence, the stability of these structures should be least ensured against the geotechnical concerns like bearing capacity, settlement, and liquefaction. The present study is about the development of an institutional campus located in seismically active zone. The proposed development comprises of both academic and residential buildings whose loading intensities ranges from single to maximum of 8 stories proposed to rest on open/pile foundations. The subsoil comprises of loose-to-medium dense sand (fines <12%) revealed that the soil is susceptible to liquefaction. The deep vibro compaction (VC) technique considered to be an effective solution to mitigate the liquefaction and found to be an alternate foundation solution over the conventional piling method. In addition, fully automated real-time quality control measures adopted to ensure the execution of VC works is also discussed.

Keywords Liquefaction · Deep vibro compaction · Quality control measures

1 Introduction

India being a fast-developing country with rapid urbanization, the scarcity of challenge-free construction land increases which in turn forces the utilization of available land with suitable ground treatment. The major geotechnical challenges of these available lands are insufficient bearing capacity and excessive settlements. Apart from the above two, liquefaction possess a major threat in seismically active zones which got significantly distributed across the nation (seismicity map—IS 1893

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part 1: 2016). This leads to the development of various techniques to mitigate the potential threat caused by liquefaction.

A phenomenon where the insitu soil loses its strength and stiffness due to the rapid change in the stress conditions mainly because of earthquake loading causing the soil to behave like a liquid is known as liquefaction. It occurs predominantly in loose, uniformly graded, cohesion-less fine-to-medium grained soils under partial or fully saturated condition.

The liquefaction potential shall be reduced significantly by densification (Vibro compaction) of the loose ground [1] or by providing the required drainage path (Vibro stone columns) for dissipating the rapid buildup of pore water pressure [2]. The densification process would certainly increase the cyclic resisting force, provided the presence of fines content is in the range of 10–15%. Alternatively, the provision of drainage path would reduce the driving force [3, 4].

A simplified procedure to evaluate the liquefaction potential that was developed by Youd et al. [5] helps the practicing engineers to identify whether the insitu soil possess required amount of resistance (CRR—cyclic resistance ratio) to counter the dynamic forces (CSR—cyclic stress ratio). In general, the minimum FOS (=RR/CSR) required against liquefaction potential shall be greater than 1. This paper deals with the case study where the vibro compaction method had been adopted for mitigating liquefaction and also addresses its value addition to the proposed project.

2 Deep Vibratory Techniques

2.1 Mechanism

The deep vibratory techniques include vibro compaction and vibro replacement methods. Based on the relationship between particle size and available vibro techniques, the vibro compaction method was selected for this subject work. In this technique, the vibrator is lowered into the ground with the combination of vibration and high-pressure water jetting. The horizontal vibrations provided by the depth vibrators rearranges the sand particles to a dense configuration from its initial loose state. The desired compaction is achieved only if the induced vibratory force is enough to overcome the residual frictional strength available with in the soil. Based on the soil response to vibration, four different radial zones are defined surrounding the compaction point where the vibratory forces get attenuated with increasing radial distance from probe. The zones are fluidized zone, plastic zone, compaction zone, and elastic zone as shown in Fig. 1. In saturated soil, the fluidized zone is developed when the pore water pressure buildup due to induced acceleration is greater than the rate of dissipation. This in turn reduces the effective stress and breaks the soil friction to allow the soil to rearrange into denser configuration. Hence, the influence of compaction is mainly dependent on the radius of the fluidized zone. In case of dry soil, the water jetting plays a crucial role in the formation of fluidized zone. In

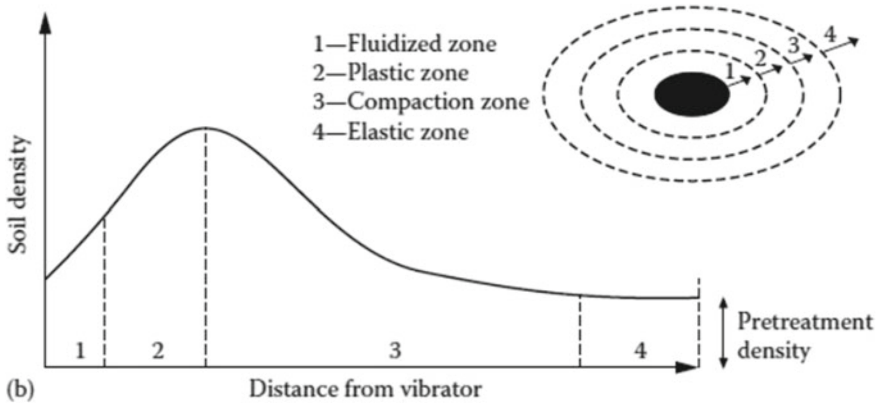

Fig. 1 Response of granular soils to vibration [6]

plastic zone, the soil will not be fluidized, but the compaction energy shall be transmitted to shear the soil particles and forms closer packing. Further, the compaction energy gets dampened and zone of zero improvement is reached. The reduction in void ratio due to the rearrangement soil particles causes subsidence which need to be backfilled at ground level to maintain the required reduced level of the site. The degree of compaction achieved at a point depends mainly on the characteristics of the soil being treated (initial density, grain size, shape, etc.,) and the vibrator (frequency, amplitude and acceleration of vibrations, and holding time at each step).

2.2 Installation Procedure

The vibrator is lowered into the ground under its own weight assisted by water flushing till the required depth and maintained at the same depth till the predetermined amperage or the preset time interval (30–90 s) has elapsed, whichever is earlier. After satisfying the amperage/time criterion, the vibrator is raised to a pre-determined height (say 0.3–1.0 m) and again is held in position to satisfy the amperage/time criterion. These steps shall be repeated till the vibrator reaches the surface. The lift height, holding time, and compaction amperage shall be finalized based on trial works.

3 Case Study

3.1 Project Background

The proposed development was for an institutional campus in North India which comprises of 15 structures including both the academic as well as the residential buildings which were planned to rest upon open/pile foundation systems. The proposed structures consist of single to maximum of nine floors with loading intensity ranges up to 150 kPa. The subsoil consists thick deposit of loose-to-medium dense sand with fines content less than 12%. The site being in seismically active zone (Zone IV; PGA-0.24 G), under earthquake conditions, the soil deposits till 10–12 m depth which was susceptible to liquefy. The upcoming sections deal with the soil characterization, performance criteria, and implementation of the ground improvement works to mitigate liquefaction.

3.2 Subsoil Condition and Liquefaction Assessment

The soil investigation works carried out using the standard penetration results revealed the presence of loose-to-medium dense sand with fines <12% till 10–12 m followed by the dense sand layer till 15 m. This layer was underlain by hard clay till the exploration depth as given in Table 1. The groundwater table during the time of investigation was approximately at 9 m which was expected to be at shallow depth during the monsoon period. Later, the confirmatory investigations were carried out using the electronic cone penetration test (ECPT).

Liquefaction analysis was carried out using the simplified procedure for evaluation of liquefaction potential given in IS 1893 (Part 1): 2016. The maximum liquefiable depth varies from 5 to 12 m across all structures. This could be very well understood from Fig. 2 where the plot between the depth and factor of safety against liquefaction was shown along with ECPT data.

Table 1 Subsoil profile

S. No.	RL (m)		Layer thickness (m)	Soil description	Tip resistance (q_c, MPa)
1	59	57	2.0	Loose sand/Firm clay	1–5
2	57	55	2.0	Medium-dense sand	5–10
3	55	47	8.0	Medium-dense-to-dense sand	9–15
4	47	44	3.0	Dense sand	13–16
5	44	39	5.0	Hard clay	>4

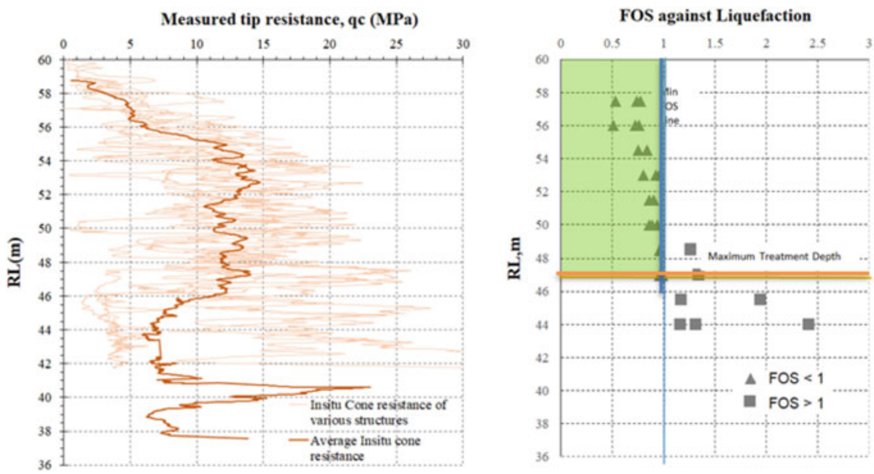

Fig. 2 Evaluation of liquefaction potential

3.3 Implementation of Mitigation Measures

Trial Works

Before the execution of main works, the influential parameters like suitable spacing, compaction amperage, and vibrator holding time of each lift required to achieve the minimum FOS of 1 against liquefaction were identified using trial works. The compaction points were spaced at 2.75 and 3.0 m in triangular grid, and the post-ECPTs were executed at the weakest point of compaction as shown in Fig. 3. The ECPT results of the pre- and post-treatment shown in Fig. 4 were compared, and factor of safety against liquefaction was computed. Both the spacings were found satisfying the target safety factor of 1. Hence, for the main works, the 3.0 m spacing was adopted for structures less than four floors, and 2.75 m spacing was adopted for structures having more than four floors.

Main Works

Though the spacing of 3.0 m satisfies the requirement of liquefaction resistance, 2.75 m spacing was adopted for higher load intensity structures (>G+4) to make utilization of the additional bearing capacity contribution from the ground treatment. Apart from the main compaction points, additional rows were executed all along the periphery of each structure for lateral confinement whose extent was half the liquefying depth. The lateral confinement is mainly provided to prevent the transmission of pore water pressures from the adjacent non-treated zone.

Quality Control MEASURES and Post-treatment Assessments

Effective quality control measures were being followed throughout the main works to ensure the operational parameters that are in line with target criteria. The execution of vibro compaction had been monitored on real-time basis with the support of

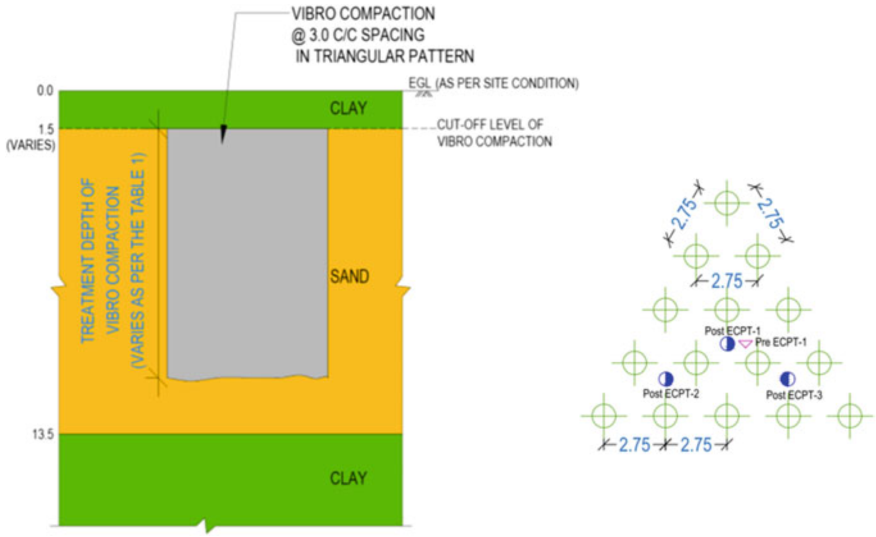

Fig. 3 Typical treatment scheme and arrangement of trial works

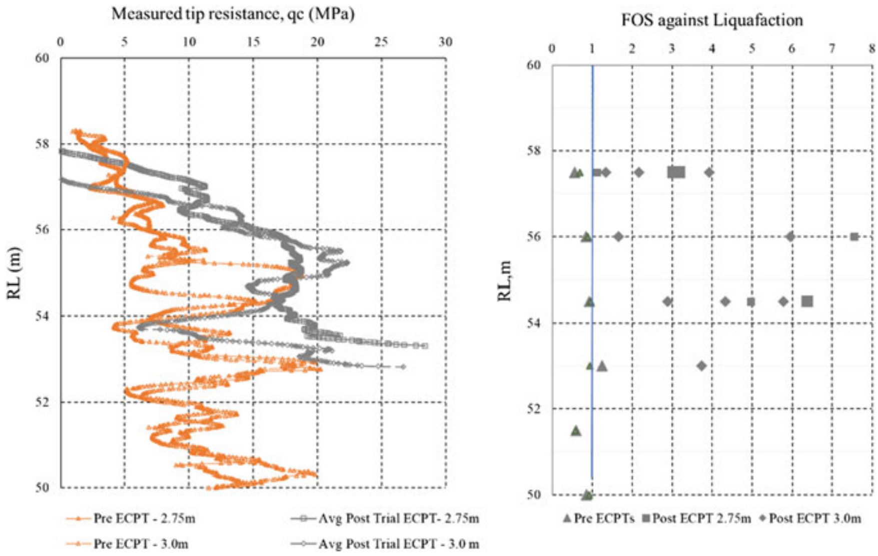

Fig. 4 Pre- and post-treatment ECPT results of trial works

M4 graphs which will provide the data of time and depth of compaction along with compaction effort of each lift as shown in Fig. 5 with which the quality of treatment shall be ensured. Further the efficacy of ground improvement had been confirmed with the support of post-treatment ECPT data. Post-treatment evaluation results shown in Fig. 6 clearly indicates the tip resistance plot shifts right toward the denser state and kept increasing with depth, and thus, the factor of safety against liquefaction is greater than 1 at all depths.

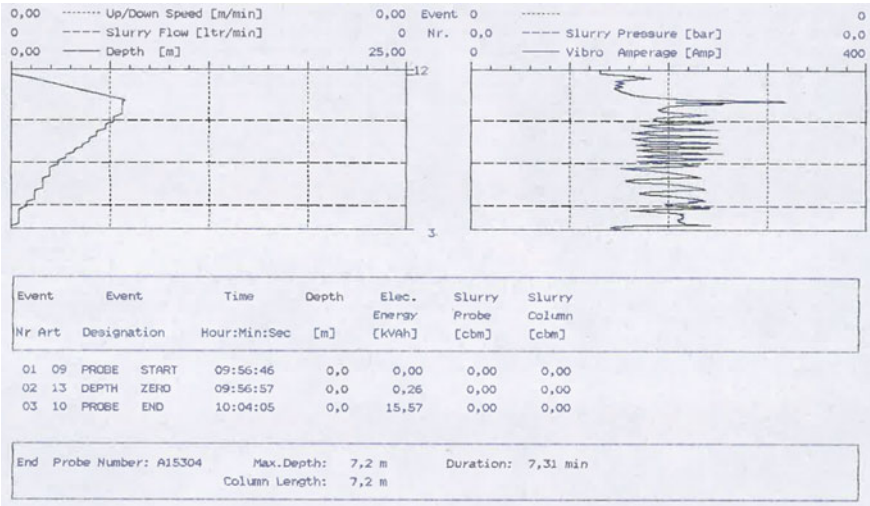

Fig. 5 Typical M4 graph

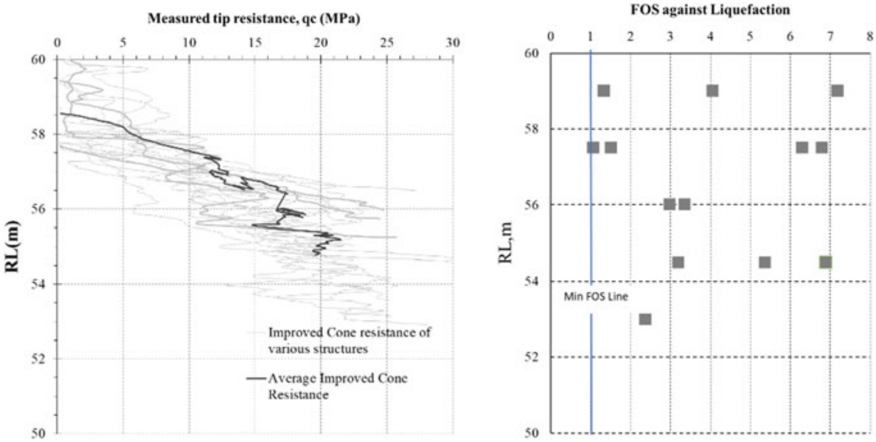

Fig. 6 Post-treatment evaluation of VC works

Fig. 7 Depth vibrator in action and subsidence in the form of conical carter due to compaction

The compaction process causes subsidence of ground in the form of conical carter as shown in Fig. 7 which happens mainly due to the reduction of void ratio caused by the reorientation of the particles toward denser configuration. Suitable backfill material (sand) had been continuously fed from the ground level to compensate the subsidence volume due to compaction. The sand compensation volume came around 12% of the overall treatment volume which once again emphasis on the quality of compaction works executed.

3.4 Advantageous Foundation Solution

The liquefaction phenomenon is independent of the loads acting upon the ground. Hence, irrespective of type and load of structures, the ground needs to be treated for liquefaction till the required depth. However, during the time of budgeting, the cost implication due to liquefaction was not captured, and the pile foundation was the proposed system for structures with more than four floors (loading intensity >120 kPa). The liquefaction mitigation by vibro compaction densifies the soil surrounding the vibrating probe which reflects in the improved shear parameter (friction angle) of the insitu soil. This emanates as an added advantage of obtaining the treated ground with enhanced bearing capacity which would be ample to propose open foundation as an alternate to piling which saves considerable amount cost and time. In terms of production, the vibro compaction works save nearly 60–75% of construction time and accounts for 25–30% savings in cost.

4 Conclusion

The encountered soil conditions pose threat to liquefaction. Vibro compaction was chosen as ground improvement technique, since fines contents are less than 12%. The operational parameters such as spacing and pattern of the compaction points were arrived based on the trial works. Effective quality monitoring procedures were adopted in the main works, and efficacy of the compaction works was ensured by pre- and post-treatment cone penetration tests. The results of the post-treatment found satisfactory and achieved the target factor of safety against liquefaction potential. Shallow foundations were chosen as foundation solution for low rise buildings (G+4). Vibro compaction technique proved to be the effective ground improvement option to address liquefaction mitigation, especially in cohesion-less soils.

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Granular Anchor Pile System for Resisting Uplift: A Review

Jerin Joseph, Shailendra Kumar, J. B. Patel, and Yogendra Tandel

Abstract Granular anchor pile (GAP) system is a modified stone column in which the stone column is reinforced using an anchor rod with an anchor plate placed at the bottom. The anchor rod is embedded in the footing of the structure which rests on the stone column. This system prevents the uplift of the structure which may be caused by uplift forces like the presence of expansive soil below the footing, wind forces, or buoyant forces. This paper presents a review about the application of granular anchor piles, the conditions in which it can be used and the method of installation. A discussion on the parameters like length, diameter, soil type, and its strength which influences the uplift capacity of the granular anchor pile is also given.

Keywords Granular anchor pile · Granular pile · Anchor foundations · Ground improvement

1 Introduction

1.1 Occurrence of Uplift

The foundations of structures are required to transmit compressive forces safely to the subsoil. However, sometimes these may be accompanied with moments in addition to the lateral forces causing uplifting of foundations. The uplift of the foundation is normally caused by expansive soils, frost heave, wind, and hydrostatic force [1]. If the foundation of buildings especially lightly loaded civil engineering infrastructure is built on expansive soils they are subjected to alternate upward and downward movement due to expansion and shrinking when the soil absorbs moisture and dries, respectively. This leads to distress in structural members such as columns, walls, and

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flooring, resulting in unsightly cracking. Uplift forces also act on the foundations of structures such as dry docks, basements, and pumping stations that are constructed below the fluctuating water table due to hydrostatic forces. Structures like transmission towers and tall chimneys are subjected to wind effects which cause a considerable amount of uplift forces under their foundations. The occurrence of uplift under the foundations if left unchecked causes irreparable damage to the building [1]. In expansive soils, some of the techniques used to reduce the heave due to the expansion are sand cushion method, cohesive non-swelling layer method, physical alteration method, chemical alteration, and under-reamed piles. In soft soils and weak deposits the under-reamed pile, driven or cast in-situ piles are used to resist uplift.

1.2 Granular Anchor Pile (GAP)

A new tension-resistant foundation technique was proposed by Phanikumar [2] called granular anchor pile (GAP) in which a conventional granular pile or stone column is converted into a tension-resistant foundation by providing a reinforcement bar of suitable strength in the middle of the pile and the bottom end of the reinforcement is connected to a steel anchor plate or concrete pedestal and the upper end is fixed to the footing as shown in Fig. 1. Kumar [3] extended the research of uplift capacity of GAP to cohesionless and cohesive soils using both laboratory and field tests. GAP has been found to control uplift forces significantly in expansive soils, soft clay, and loose sand deposits through extensive research conducted through laboratory scale and field scale tests. A number of numerical studies also show that granular anchor piles are an effective method to control uplift of foundations. This paper presents a review of the work carried out by researchers on different aspects of the GAP.

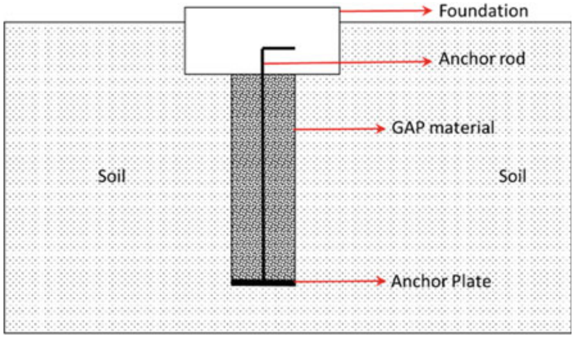

Fig. 1 Cross-sectional details of GAP foundation

1.3 Mechanism of Failure and Uplift Resistance of GAP

Most research available is concentrated on the study of GAP in expansive soil and soft soils. Therefore, the mechanism of uplift resistance discussed in the literature is in terms of the GPA and its interaction with the surrounding expansive or soft soil. When the uplift force acting on the building is transferred to the foundation which is connected to GAP, it is resisted by the weight of GAP acting in a downward direction and the friction along with the pile–soil interface [4–6]. If the GAP is installed in expansive soil, the swelling of the soil further enhances the uplift resistance of the GAP by confining it radially and thus increasing the friction along the surface of the pile [4–7]. The granular material also plays an important part due to its dilatancy [8]. The failure mode of the short GAP and long GAP is either shaft failure or bulging failure, respectively, indicating that the failure mode mainly depends on the length-to-diameter ratio of the GAP [9]. Though there is ample research on the uplift mechanism of the GAP in the cohesive and expansive soils in terms of uplift load response, there is only limited literature that focused on the failure mechanism of GAP in cohesionless soils.

1.4 Installation of GAP in Field

Kumar [3] conducted a field study on the GAP in cohesive and cohesionless soil. A borehole was made in the ground using a manually operated auger. Fresh concrete was deposited at the bottom using tremie pipe and the prefabricated anchor plate and anchor rod was lowered on the concrete followed by concreting the base again. The anchor plate and anchor rod assembly consist of a mild steel plate with a rod of the same material welded on to the plate and extra ribs to strengthen the plate. The concrete cover prevents the corrosion of the anchor plate and gives extra strength to the anchor. The concrete was allowed an initial setting time of 7 days before back-filling the borehole with a mixture of crushed stone material and sand. The granular material mixture charged into the borehole in equal volumes and was compacted using a steel annular hammer with a fixed height of fall and number of blows. A similar procedure has been adopted by different researchers in the field studies without the concrete cover of the anchor plate [5, 8–11].

The installation procedure used by Liu [12] in the field is similar to the installation of the dry vibrated stone column. A steel pipe with a precast concrete toe was driven into the soft soil up to design depth. Concrete is filled at the bottom of the pipe and compacted with the pipe withdrawn upward. A steel bar with end-plate and centralizer is placed on the compacted concrete, and fresh concrete is again poured on the end base, this anchors the steel bar in position. The steel pipe is filled with granular material, and as it is gradually pulled out, the material is compacted using vibration. This process is repeated until the pipe is fully withdrawn.

2 Experimental Studies

2.1 Laboratory Investigation

Kumar [3] investigated the effect of embedment ratio, i.e., the length-to-diameter ratio, number and spacing of the GAP on the uplift load response and ultimate uplift capacity in medium dense sand. The diameter of the GAP was found to have a major role in increasing the ultimate uplift capacity. An embedment ratio of ten times the diameter and spacing of three times the diameter of the GAP was found to be optimum. A study on the oblique pullout capacity of the GAP was carried out by Singh [13]. The study was conducted by varying embedment ratio, number of GAP, and the oblique pullout angles as 30, 45, 60, and 90° with respect to pile axis. The experimental investigation on single and group of GAP in loose to medium dense sand revealed that the ultimate pullout and efficiency was a function of the embedment ratio of the GAP. The ultimate pullout capacity was found to decrease with increase in pullout angle. The rate at which the pullout capacity increased when the embedment ratio increase was found to be a function of the pullout angle with a high rate of increase in case of lower angles and almost constant rate of increase in case of pullout angle 90°. The efficiency of the pile group was observed to increase with embedment ratio and a decrease in spacing between GAP.

Phanikumar [2, 4] and Ibrahim [14] performed laboratory model tests on GAP installed in expansive clay beds to study the influence of length, diameter, and relative density of the GAP and the dry unit weight of expansive soil surrounding it. It was found that that the GAP reinforced expansive clay beds undergoes significantly less heave and the rate of heave improved with respect to the expansive clay bed without GAP. The reduction in the heave increases with an increase in the surface area and relative density of the GAP due to the friction resistance offered by the pile–soil interface. The reduction in heave was found to increase with an increase in the number of GAP and reduction of the spacing between them.

In the pilot tests conducted in the laboratory by Harikrishna [15], model GAP and concrete pile were installed in saturated expansive soil and expansive soil compacted to achieve maximum dry density at optimum moisture content. The tensile load at a constant speed was applied to the GAP and the concrete pile to determine their pullout capacity. It was observed that the pullout capacity of the GAP was three to four times that of a concrete pile. The higher pullout capacity was attributed to increased friction at the pile–soil interface due to the lateral displacement of the granular material of GAP.

Laboratory model tests were performed by Muthukumar and Shukla [16] to compare the heave reduction of expansive clay in a cylindrical due to the installation of GAP and helical pile anchors (HPAs). The tests were conducted by varying the number of GAP and HPA as 0 (unreinforced expansive clay bed), 1, 2, and 3. In the case of HPA, the helices were also varied as 1, 2, and 3. The expansive clay in the tank was slowly inundated with water, and the heave was obtained using dial gauges. It was found that there was a reduction in heave for both the GAP and HPA installed expansive clay beds. There was also a reduction in heave with an increase in the number of helices of HPA. However, the GAP was more effective in controlling the uplift of the foundation than HPA with any number of helices. The GAP was found to have additional uplift resistance due to surrounding clay offering lateral pressure against bulging of the GAP.

Few studies have been conducted to incorporate geosynthetics with GAP to enhance its uplift resistance [4, 6, 16]. In the laboratory scale study carried out by Phanikumar and Rao [4], the effect of placing base geosynthetic above the anchor plate to form an integral part of the GAP was studied. The model study was conducted in expansive soil and two types of geosynthetics were used, i.e., geotextile and geogrid. The diameter of the base geosynthetic was kept larger than the GAP. The confining media used to sandwich the base geosynthetic was varied as black cotton clay—geotextile/geogrid—bottom sand layer, fine sand—geotextile/ geogrid—fine sand, coarse sand—geotextile/geogrid—coarse sand, and metal chips—geotextile/geogrid—metal chips. The interface friction angle between the confining media and the geosynthetic was found out using shear box text by placing the geosynthetic at the failure plane. It was observed that the interface friction angle increased with increase in gradation of the confining soil medium. A higher interface friction angle was also seen in the case of geogrid with respect to geotextile. The expansive soil was first saturated to induce heaving in the soil and after 100% saturation is achieved the anchor rod of the GAP was pulled out with equal increments of load. High heave reduction along with short time was observed in the reinforced expansive soil. The ultimate pullout load of GAP with the base geosynthetic was at least 2.4 times that of the GAP without base geosynthetics. The pullout capacity of the GAP increased with increase in gradation of the confining medium due to the resistance offered by the confining medium and base geosynthetic. The resistance was more in case of the geogrid base than the geotextile base.

Phanikumar [6] investigated the effect of placing geogrid layers inside the GAP at a distance from the anchor plate. The number of geogrid layers (0, 2, 3 and 4), spacing and the location of bottom geogrid from the anchor plate were varied, and the effect on heave and pullout behavior of GAP was studied. The diameter and embedment depth of the GAP were kept constant. The GAP was installed in expansive soil. The GAP was tested in both heave and pullout condition. It was observed that as the number of geogrid layers increases the heave of the clay bed reduced and the time takes to achieve the equilibrium heave also decreased. The heaving also reduces with the reduction in the spacing of the geogrid layers. This reduction was attributed to the confining and interlocking effect of the geogrid layers which resists the bulging and lateral spreading of the GAP material. It was observed that the pullout capacity of the GAP increased as the number of geogrid layers increase and the space between them decreased. The closer the bottom geogrid layer is to anchor plate the better was the pullout capacity. This was also attributed to the confining and interlocking effect of geogrid layers.

Muthukumar and Shukla [16] studied the effect of the geosynthetic encasement and its stiffness on the heave reduction capacity of single and group of GAPs. It was observed that the heave decreased with an increase in the number of GAPs.

The encasement of GAPs leads to more than 50% heave reduction than non-encased GAPs. This was true for any given number of GAPs. The increase in the stiffness of the geosynthetic encasement also reduced the heave. The encasement of the GAP leads to resistance against bulging failure, thus increasing the uplift capacity. The increase in stiffness also helps in providing higher hoop stress to confine the GAP material.

2.2 Field Studies

Kumar [3] conducted an extensive field study in both cohesionless and cohesive soils. A constant diameter of 0.3 m and spacing of 3 times the diameter for a group of GAP have been adopted for the field study. The results of uplift tests on a group of GAP showed that the ultimate capacity of the group was highly dependent on the number of GAPs in the group. The data showed that the ultimate capacities of the two and four number GAP group are almost equal to the ultimate uplift capacity of the single GAP multiplied by the number of GAPs in the group. In the study conducted in a cohesive soil, an embedment ratio of 13 was found to be optimum. The increase in embedment ratio played a minor role in the increase of the ultimate uplift capacity. However, the diameter and number GAP in a group played a significant role in the increase of uplift capacity. The ultimate uplift capacity of the 2 GAP group of embedment ratio 20 in cohesionless was almost 3 times more than in cohesive soil. Similar comparisons also suggest that the GAP system is more effective in cohesionless soil than cohesive soil.

Liu [12] conducted a field research study on the performance of two prototype GAPs installed in a land newly reclaimed from the sea for the construction of sewage pools. Repeated compression and tension cycle tests were carried out on the GAP which was installed in mostly soft soil layers. The length and the diameter of the prototype GAP were 17 m and 0.5 m, respectively. The anchor rod of the GAP was fitted with load cells at top, middle, and bottom portions, and load transferred on to the rod was measured during the compression and tension cycles. The tensile load applied during the test was 50% of the calculated ultimate uplift load, i.e., 150 kN and a compressive load of 180 kN. The test results indicate that the GAP was able to perform similar to the ordinary stone column under compression and at the same time resists tensile loading with undergoing large plastic displacements. The GAP system was later implemented to counter the cycling loading that takes place under sewage pools as they are filled and emptied.

Rao [7] and Phanikumar [5] conducted pullout tests on field scale GAP installed in expansive clay beds of depth 1 m. The clay bed was prepared by compacting expansive clay at a water content of 15% in a pit. The diameter of the GAP was varied as 100, 150, and 200 mm, and the length was varied as 500, 750, and 1000 mm. The expansive clay bed was flooded with water and allowed to heave, and once the saturation was reached, pullout test was conducted. The displacement of the top of the GAP during heaving was noted. The results showed that the heave of the GAP reinforced bed reduced by 70, 87, and 92% for full depth GAP of diameter 100, 150, and 200 mm, respectively, when compared to unreinforced expansive clay bed. The time taken for achieving the full saturation and heave equilibrium reduced drastically in case of GAP reinforced expansive clay beds which were attributed to increased permeability of the GAP material. The pullout load increased with increase in the embedment depth indicating that the uplift resistance depends on the surface area of the GAP. The test was also carried out on a group of GAPs by keeping the diameter and length as 150 mm and 1000 mm, respectively. During pullout test, the center GAP of the group was loaded. When the GAP in the group was tested and compared to a single GAP, it was noted that the GAP in the group performed better. This was due to high lateral swelling pressure and arching action in the group of GAP. Phanikumar [5] studied the undrained shear strength, penetration resistance, and variation of heave with a depth of the reinforced and unreinforced expansive clay beds. Cylindrical samples from 25, 50, and 75 mm from the top of the saturated expansive clay bed were retried and conducted unconfined compression tests to determine the undrained shear strength. The undrained shear strength of the reinforced expansive clay bed was higher because of the increase in density as the heave was controlled in GAP reinforced clay beds. The poor permeability and uncontrolled heave lead to lower undrained shear strength in the unreinforced clay bed. Penetration tests were also conducted using a proctor needle up to 25, 25, 50, and 75 mm from the top of the saturated expansive clay bed. The penetration resistance of the reinforced expansive clay beds was higher than unreinforced clay bed. Thus, penetration tests result verified the results of the unconfined compression tests. The heave of the expansive clay bed was monitored at different depths and radial distances from the center of the GAP.

Rao [11] carried out field-scale plate load tests were conducted GAPs installed in expansive clay beds to study the compressive load response. The dimensions of the GAP were the same as that of the study done by Rao [7] and Phanikumar [5]. The tests were conducted in unreinforced expansive clay beds, reinforced expansive clays beds with single and group of GAP (3 GAPs). Two conditions of loading plate position were tested, one was composite ground position in which plate was placed on GAP as well as the expansive clay and in the other condition GAP alone was loaded. In the composite ground and GAP group study, the diameter and length were kept constant as 150 mm and 1000 mm. The embedment ratio and the diameter of the GAP were varied in the compression test of the single GAP. The compression tests were carried out after full saturation is achieved by flooding the clay bed. The results showed that the stress required for the settlement of 25 mm in the composite ground was more than twice that of the unreinforced bed, and in case of GAP alone, it is more than 3 times. When the group of GAP was tested, an improvement of 65% with respect to unreinforced clay bed was noted. The bulging of the GAP was observed to increase with the increase in the diameter of the GAP for a given length.

Sivakumar [9] performed the study of ultimate pullout capacity of granular anchors constructed in intact lodgement till and ground deposits. Uplift tests were also performed on the cast in situ concrete anchors to compare its performance with that of the GAP. Failure modes of GAP were also studied in detail along with proposing a new method for predicting the ultimate uplift capacity. The tests were carried out in two sites, in the first site the anchors were installed in stiff clayey silty sand with occasional gravel (mean undrained cohesive strength of 55 kPa) that was placed around 50 years ago, the second site soil consisted of stiff to very stiff, brown, slightly sandy clay of low plasticity. The first site was used to compare the cast in situ concrete anchors and GAP. The tests were conducted on the anchor of length 0.5, 1, 1.5 m with diameters 0.07 and 0.15 m. The ultimate uplift capacity of the cast in situ concrete anchors and GAP was found to be almost same but the mode of failure was different. The GAP experienced ductile failure by undergoing a large amount of displacement while the concrete pile failed by sudden pullout failure with very less upward displacement compared to GAP. In short GAPs, the ground appeared to heave around the GAP indicating shaft failure. The longer GAPs failed by localized bulging at the base. The second site was used to understand the failure mode of GAP with respect to the surrounding ground surface. It was observed that short GAPs (length 0.5 and 0.45 m with diameter 0.196 and 0.148 m) failed in shaft resistance and the granular material at top of GAP, and surrounding soil had undergone a substantial amount of heave. In case of long GAPs (length 0.8, 1.47 and 1.62 m with diameters 0.168 and 0.219 m), only marginal heaving was observed on the top of GAP and surrounding soil. The researchers proposed that the uplift load is resisted by shaft friction and localized bulging of GAP at its base. It was suggested that an embedment ratio of lesser than 7 resulted in shaft resistance failure and greater than 7 resulted in localized bulging failure at the base. Some tests were also carried out for double plate GAPs. The results indicated that a double plate anchor will have enhanced uplift capacity if the length of each segment should have an embedment ratio of greater than 7.

Harikrishna [15] carried out field testing to compare the performance of cast in-situ concrete pile and GAP in expansive soil. The pullout tests were conducted in both saturated (after 10 days of wetting the ground) and unsaturated condition. The length was varied as 1 and 1.5 m with diameters 100 and 150 mm. The results of the pullout tests indicated that the GAP had an uplift capacity of more than twice than that the cast in-situ concrete pile in both saturated and unsaturated conditions. This was attributed to the interlocking between the granular material of the GAP and the ideas of the borehole. The pullout resistance of the GAP and cast in-situ concrete pile reduced due to the saturation by 32% and 25%, respectively. It was also noted that for the same area the diameter of the GAP plays a more important role than its length.

Krishna [15] conducted studies on the heave reduction of flooring panels using a combination of CNS layer and GAP. Five flooring panels 2.5 cm thick mortar of 3 m by 3 m size with five types of foundations were constructed in expansive soil ground. Five types of foundation were constructed, they were 0.5 m thick CNS cushion layer, a 9 group granular column of 0.2 m diameter and 0.6 m depth, a 9 group GAP of the same dimension as that of granular column, a combination CNS layer and group of granular column and an untreated 100 mm thick murrum compacted. The constructed flooring panels were cured for 10 days and flooded with water for 100 days. The seasonal movements of flooring panels were monitored for 4 years. It was noted that the GAP foundations performed excellently with respect to reduction of the heave. A heave reduction of 89% could be achieved when the flooring is provided with GAPs. The heave could be reduced to 92% when a combination of GAPs and CNS cushion is used.

3 Numerical Studies

Ismail [17] carried out three-dimensional finite element analyses of a typical double-story building constructed over a system of granular pile anchor foundation system in a reactive soil. Both heave and shrinkage were modeled by applying equivalent volumetric strains to the affected area. An analysis of the building resting on pad footings with and without the GAP was done and the results were compared. The Mohr–Coulomb (MC) model was used to model the reactive soil, and hardening soil (HS) model was used to GAP material and the underlying dense sand. The comparisons were made for the top beams of the central frame of the building. The results in terms of induced deformations, angular distortions, and bending moments due to heave and shrinkage of the reactive soil were analyzed. It was observed that the maximum vertical displacement of 6.7 mm developed in the top central beams of the building due to heaving of the soil has been eliminated due to the use of foundation with GAP. It was also noted that the GAP foundation highly reduced the angular distortion and bending moments developed in the top beams. The results from the finite element modeling concluded that GAP foundation can be potentially used to reduce the destructive effects of expansive soil on the building.

Kranthikumar [18, 19] undertook extensive field scale study on the GAP in cohesionless soil using the finite element analysis software PLAXIS 3D. The studies were conducted on single and group of GAP by varying the length, diameter, relative density of the surrounding soil, the elastic modulus of the surrounding soil, number of GAPs in the group, compaction effects, and the depth of water table. The Mohr–Coulomb (MC) model was used to model the cohesionless soil. It was concluded that the uplift resistance increases with an increase in the length and diameter of the GAP. An economic embedment ratio of 10 was proposed. It was noted that the relative density of the surrounding soil had a considerable effect on the ultimate uplift capacity. The efficiency of the group of GAP was found to decrease with an increase in the number of GAP for constant spacing due to overlapping stresses. The compaction effect on the GAP and surrounding soil during construction if considered during the modeling of GAP led to increases ultimate uplift capacity. The increase in water table level was found to decrease the uplift capacity. The two failure mechanisms identified were shaft failure and bulging failure.

Abhishek and Sharma [20] modeled GAP installed in expansive clay bed to study the length, diameter, elastic modulus of soil and pile, spacing and efficiency of a group of GAP, and effect of GAP construction. The GAP and clay bed were modeled in a laboratory scale with soil and GAP material defined using the Mohr–Coulomb (MC) model. The results from the models showed that the ultimate capacity of GAP increased with an increase in length and diameter. This was attributed to the self-weight of pile and friction developed along the pile–soil interface. The increase in the modulus of surrounding soil leads to increase in uplift capacity but this was not of linear nature. Optimum spacing of 2.5 times the diameter of the GAP was proposed based on the results from the modeling of the group. The efficiency of the group of GAP was found to increase up to 10 times the diameter after which it decreased; thus, it was proposed as an optimum embedment ratio for group pile.

4 Conclusions

The following are the conclusions from the review of the literature on GAP.

- The GAP system is a cost-effective foundation method that can be used in all types of soils to counter uplift forces and achieve heave reduction. The field and laboratory scale studies suggest that GAP is at par or better than currently used tension-resistant foundation techniques like concrete anchor pile and screw pile. The compression tests on the GAP showed that it behaves similar to an ordinary stone column in soft soil.
- The uplift is resisted by the weight of the GAP and the friction developed at the soil–pile interface. The GAP installed in expansive soil uplift is also resisted by the lateral swelling of the soil. The two types of failure observed in GAP were shaft failure and localized bulging failure at the base of the GAP. Shaft failure was observed in short GAP and the localized bulging failure occurred in long GAP.
- The installation technique used in the field was of two types. One method was similar to the installation of the dry vibrated stone column, and the other is similar to rammed stone column installation without vibration. It was noted that during field installation, it is better to give a proper concrete cover to the anchor plate to strengthen it and prevent corrosion.
- The ultimate uplift capacity of GAP depends on its embedment ratio, i.e., the length-to-diameter ratio, relative density, and elastic modulus of the surrounding soil and GAP material, level of the water table and degree of saturation of the surrounding soil.
- An embedment ratio from 10 to 13 is considered as the optimum embedment ratio after which there is no significant increase in ultimate uplift capacity. The diameter of the GAP plays a more important role than the length in affecting the uplift capacity of the GAP. The increase in elastic modulus and relative density of surrounding soil and the GAP material result in an increase of uplift resistance. The increase in moisture content decreases the uplift capacity of the GAP.
- The efficiency of a group of GAP decreases with spacing due to the overlapping of stresses. A spacing of 2.5 times the diameter was found to be optimum in cohesionless soil. Higher heave reduction was noted in the case of a group of GAP installed in expansive soil.

- The uplift capacity of the GAP can be improved by using geosynthetic encasement, geogrid layer inside the GAP and by increasing the number of plates. However, more studies are needed in this regard.
- The performance of the GAP in expansive soil is well-documented. There is a shortage of research literature in the application of GAP foundation under structures that are prone to uplift failure.

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Enhancing Strength Properties of Soft Soil Using Carbon Fiber

Swati Sucharita Rout and Rupashree Ragini Sahoo

Abstract Civil engineers are always ambitious when it comes to new materials that can face many challenges which have cropped up with time in the world. This paper enquires the effectiveness of carbon fiber (CF) as a better reinforcing agent to augment the engineering characteristics of soil, especially the shear strength, unconfined compressive strength (UCS), and California bearing ratio (CBR) using CI-CH soil. Varying percentage of carbon fiber (0.05%. 0.1%, 0.2%, 0.4%, and 0.6% by weight of soil) were included to analyze the gain in relative strength of soil. The research was conducted to assess the shear parameters, unconfined compressive strength (UCS), and California bearing ratio (CBR) of soil with respect to varying percentage of carbon fiber. The analysis revealed that, 0.4% of carbon fiber in the soil mixture resulted in enhancing the strength properties of the soil to the maximum. The soil reinforced with carbon fiber increased cohesion, internal angle of friction, UCS, both soaked and unsoaked CBR by factor 4.1, 1.78, 1.25, 1.56, and 1.52, respectively.

Keywords Carbon fiber (CF) · CI-CH · Shear parameters · UCS · CBR

1 Introduction

Civil engineering constructions over soft soil arise to many cases due to the shortage of good bearing soil. The characteristic properties of soft soil being differential settlement, low shear strength and high compressibility. It is extremely essential to raise the load bearing capacity of soil for constructing high rise buildings over soft soil. Numerous techniques like soil stabilization, soil reinforcement, grouting, and addition of admixtures have been adopted to tackle the issues of soft soil. Explosive cost of materials and lack of accessible resources have encouraged geotechnical

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engineers to innovate and formulate better alternatives. Depending up on the benefits of excellent dispersion and smooth blending of the fiber, a definite quantity of the fiber and soil is blended to achieve a consistent mixture. The wondrous properties of the fiber have put it under careful study of the researchers which has led to its varied applications in the industry and advancements in the reinforcing technology. Polypropylene fiber had been shown to raise the strength properties of soil [1–3]. Random distribution of polypropylene fibers in soil had been intensively studied by researchers [4] who had found it to raise the strength properties of cohesive soils. The effect of polypropylene fiber on the compressive, triaxial shear strength of the cement treated soil had been investigated [5–7]. Mirzababaei et al. [8] had analyzed the swelling properties of clay using waste carpet fiber. Hair, a natural fiber, was used to enhance the shear strength and bearing capacity of soil [9]. Changizi and Haddad [10] had investigated the consequences of the strength and underlying structure of glass fiber reinforced soil.

Carbon fiber is another fiber material containing over 90% carbon. The fiber is obtained from different organic filaments in inert atmosphere which imparts it the properties of high-temperature carbonization and low-temperature oxidation. This fiber is filamentous thus having a several advantages. The fiber has high strength, high stiffness, lightweight, high heat tolerances and resistant, high tensile modulus, low density, high thermal and electrical conductivity, low coefficient of thermal expansion, corrosion resistance, superior fatigue properties, high creep resistance, excellent strength-to-weight ratio, etc. Its appearance is distinctive, one of kind and almost not possible to replicate. In comparison with steel, it is 70% lighter, whereas with aluminum it is 40% less. This unique property finds its place of use in aerospace industry, aircraft industry, automotive industry, military, recreational application, sport equipment, medical equipment, entertainment, wind energy technology and huge numbers of various fields. In civil construction, design and development, carbon fiber finds its application in concrete beams and columns; whereas it strives to improve the compressive strength, tensile strength, shear strength, load capacity and seismic performances [11, 12]. It also helps in strengthening the structures like concrete, steel, cast iron, masonry, and timber. Lavanya et al. [13] had conducted the direct shear test to analyze the angle of interface friction between well-graded and poorly graded gravel and carbon fiber reinforced polymer (CFRP) wrapped concrete specimens to resolve the soil–substructure interaction problem. The test outcomes indicated that there was a considerable reduction in the angle of interface friction by wrapping the CFRP. Gao et al. [14] conducted a progression of unconfined compression tests for clay reinforced with carbon fiber of 9 mm long which was blended into soil with 0.01, 0.02, 0.03, 0.05 0.1 0.15, 0.25, 0.35 and 0.5%. From his investigation, the unconfined compressive strength increased in the starting and showed a gradual decline with further increase of fiber percentage, the increment impact was the most evident when the carbon fiber content was 0.1%. Firoozi et al. [15] had investigated the stabilization of soft soil using carbon nanotubes (CNT). Increase in the amount of CNT can lead to raise durability, reduced brittleness, and increased tensile strength. Wang et al. [16] conducted shear strength and compaction tests of clay soil reinforced with carbon fiber of 3 mm and 6 mm length. The results indicated

Table 1 Physical properties of soil

Properties	Values
IS classification	CI-CH
Specific gravity	2.40
Grain size distribution	
(a) Sand (%) (b) Silt (%) (c) Clay (%)	12.04 41.74 46.22
Liquid limit (%)	49.72
Plastic limit (%)	26.12
Plasticity index (%)	23.60

that the OMC decreased while MDD increased and the internal friction angle, cohesion increased firstly, followed by a decreased. It should be noticed that in the above researches, there has not been a consent concerning the impact of carbon fiber on the CBR value of soil. Hence, the aim of this experiment was to evaluate both soaked and unsoaked CBR values, shear parameters and unconfined compressive strength of soil when mixed with carbon fiber. A sequence of direct shear tests, unconfined compression tests, California bearing ratio (soaked, unsoaked) tests were carried out and the outcomes observed from laboratory tests were collated with the unreinforced soil.

2 Materials and Methods

2.1 Materials

The soil, utilized for experimental work, was obtained at a depth of 3 ft beneath ground level from a site at Sambalpur, Odisha, India. Tables 1 and 2 present various geotechnical properties of soil and engineering characteristics of soil, respectively.

Carbon fiber was obtained from CFW Enterprises, Delhi. The carbon fiber was chopped into small pieces of 12 mm length (Figs. 1 and 2).

2.2 Test Method

Preparation of Sample. The sample obtained from the ground level was air-dried and pulverized. The soil sample was sieved through a 4.75 mm sieve for segregation of vegetative substances. For the test, length of carbon fiber used was 12 mm in five various samples that were made with the mixture of oven dried soil and 0.05%, 0.1%, 0.2%, 0.4%, and 0.6% of carbon fiber depends on the weight of the soil, respectively.

Table 2 Engineering characteristics of soil

Properties	Values
Compaction test	
(a) MDD (g/cc)	1.63
(b) OMC (%)	19.42
UCS (kN/m^2)	77.72
Shear strength parameters	
Cohesion (c) (kN/m^2)	13.73
Angle of internal friction (φ) (°)	18
CBR (%)	
Unsoaked	4.71
Soaked	1.63

Fig. 1 Carbon fiber

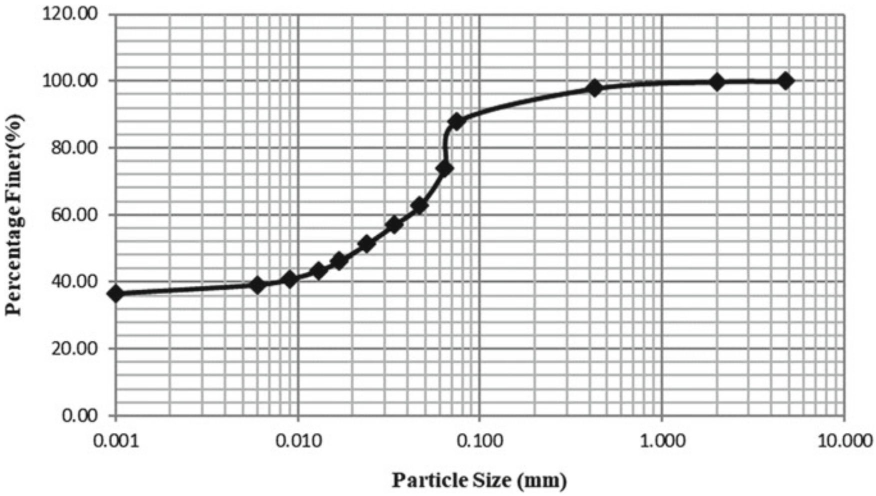

Fig. 2 Particle size distribution curve of soft soil

Then optimal water content achieved from standard Proctor test was mixed and blended until the water diffuses throughout the soil. The soil–fiber was mixed with water homogeneously in an impervious metal tray to prevent water loss. Then the direct shear, UCS, and CBR test specimens were prepared from the mix obtained. The above-mentioned tests were carried out on both unreinforced and reinforced soil specimens to compare the strength of the soil by varying the fiber content.

Direct Shear Test. A sequence of direct shear tests was carried out as per the IS 2720 (part 13) 1986 [17]. The mixture of soil–carbon fiber was compacted in a shear box of size 60 mm × 60 mm × 24 mm by ramming to standard Proctor's MDD in order to get the specimens for the experiments. The specimens had a carbon fiber percentage of 0.05, 0.1, 0.2, 0.4, and 0.6% w.r.t the soil. Three test samples were prepared for each specimen. Each sample was put through a sequence of tests at normal stress of 5, 10, 20, and 50 kPa in an unconsolidated undrained condition. The loading rate was kept to 0.125 mm per second in the tests. Readings from the proving ring dial gauge were recorded with respect to fixed intervals of horizontal dial gauge readings in order to analyze the stress–displacement behavior of both unreinforced and fiber reinforced soil. From the shear stress–normal stress plot, the shear parameters were calculated.

Unconfined Compression Test. To figure out the consequences of embedding carbon fiber with soil on its strength properties, five groups of the specimens were arranged, UCS test was executed on them, and the UCS values of each sample was measured. Based on standard Proctor's MDD, the mixture of soil–carbon fiber was compacted in a cylindrical mold of 50 mm diameter and 100 mm height. The mixture had a percentage of 0.05, 0.1, 0.2, 0.4, and 0.6% carbon fiber for each of the samples. Samples were extracted from the mold for further tests. According to IS 2720 (part 10) 1991 [18], the experiments were modulated at a consistent strain rate of 0.125 mm/min. Three samples of each specimen were studied for each varying proportion. The stress–strain curve was observed, and the UCS value was calculated.

CBR Test. CBR value is the important engineering parameter for assessing the strength of subgrade and sub-base materials for design and construction of pavement. The CBR tests were executed for different percentage of carbon fiber as per IS 2720 (part-16) 1987 [19]. Based on standard Proctor's MDD, the preparation of soil specimens was performed in a cylindrical mold of 150 mm dia and 175 mm height by compaction of the mixture of soil–carbon fiber. The specimens had a carbon fiber percentage of 0.05%, 0.1%, 0.2%, 0.4%, and 0.6%, respectively. Three samples of each specimen were experimented for a variable proportion. Then the samples were immersed in water for ninety-six hours, and again the tests were performed. All experiments were executed at a penetration rate of 1.25 mm/min until a penetration of 12.5 mm was attained. CBR values for both soaked and unsoaked samples were calculated, and the load–penetration curve was plotted for all the specimens.

3 Results and Discussion

Over the entire experimental period, several tests were performed on soft soil without and with carbon fiber reinforcement. The effect of carbon fiber addition on stress–displacement behavior, shear parameters, unconfined compression test and soaked and unsoaked CBR values were examined. The details of the experimental outcomes are presented in the below sections.

3.1 Direct Shear Test

The shear parameters of soil reinforced with varied percentages of carbon fiber of length 12 mm, attained from the direct shear tests which are briefed in Table 3. The relationship between shear stress and horizontal displacement of soil reinforced with fiber is shown in Fig. 3. As observed from Fig. 3, for each specimen the shear stress of carbon fiber reinforced soil was observed to be occurring at larger displacements compared to the unreinforced soil. It was observed that enhancement in normal stress hiked the strength of soil to the topper level after inclusion of carbon fiber and the reason could be such that rising in normal stress caused contact force and interlock between normal stress, contact force, and interlock between soil particles to rise. Increase in carbon fiber percentage of the mixture in turn increases its shear strength. From Fig. 3, the conclusion can be drawn that the inclusion of 0.4% carbon fiber leads to maximum shear stress in comparison with the unreinforced soil. The relation between typical shear stress and normal stress of reinforced soil are shown in Fig. 4. Based on the outcomes, a graph has been plotted with various percentages of carbon fiber which has been represented in Fig. 5. It was obtained from the tests that, cohesion and internal angle of friction varied from 13.73 kPa to 55.90 kPa and 18º to 32º, respectively. These strengths were raised up to 0.4% carbon fiber content, beyond which they started decreasing. Thus, the experiment suggests that the optimal fiber content was 0.4%. Similar outcomes were also recorded by Wang et al. [16]. The observation might be an outcome of interaction between soil and fiber.

Table 3 Shear parameters of carbon fiber reinforced soil obtained from direct shear test

S. No.	Mix proportion	Cohesion (kPa)	Internal angle of friction (º)
1	Soil + 0% CF	13.73	18
2	Soil + 0.05% CF	23.54	22
3	Soil + 0.1% CF	32.36	24
4	Soil + 0.2% CF	37.27	28
5	Soil + 0.4% CF	55.90	32
6	Soil + 0.6% CF	40.20	25

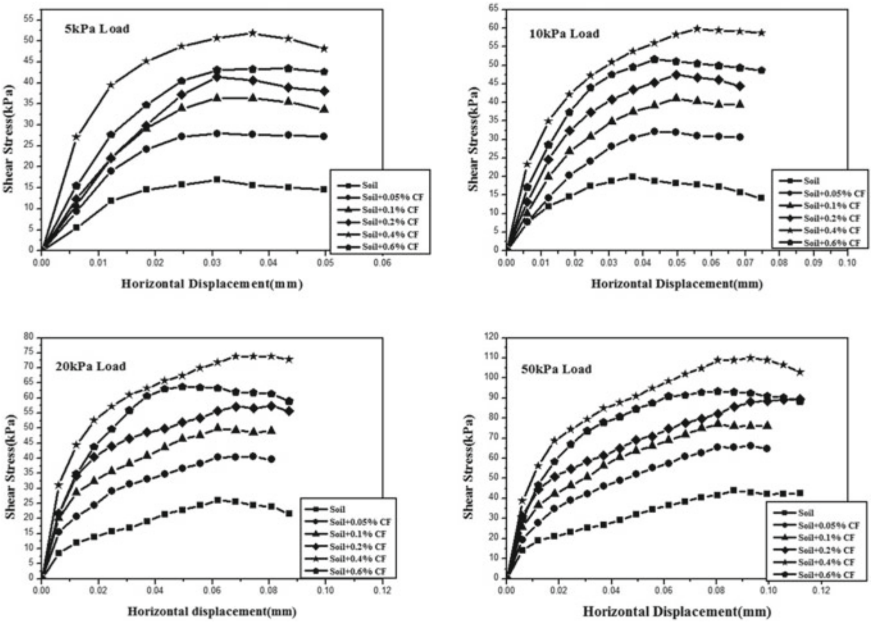

Fig. 3 Stress–displacement curves of soil reinforced by carbon fiber obtained from direct shear test

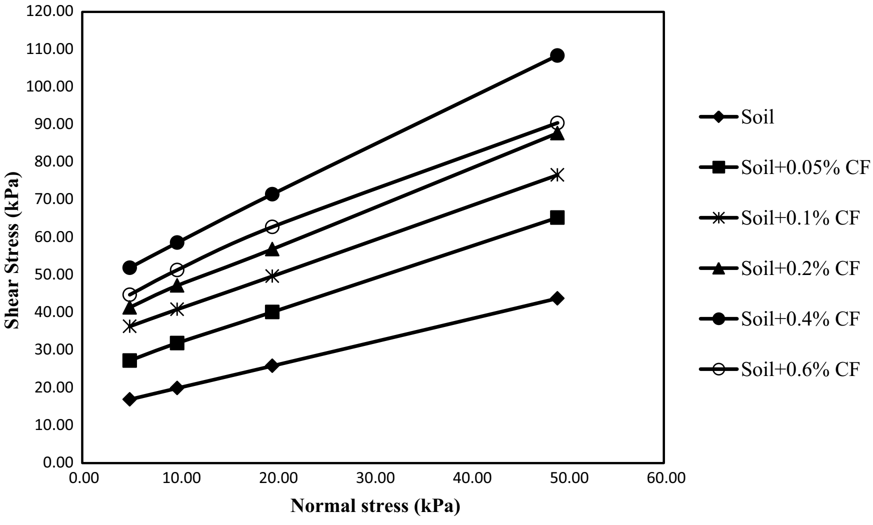

Fig. 4 Typical shear stress–normal stress plots for carbon fiber reinforced soil

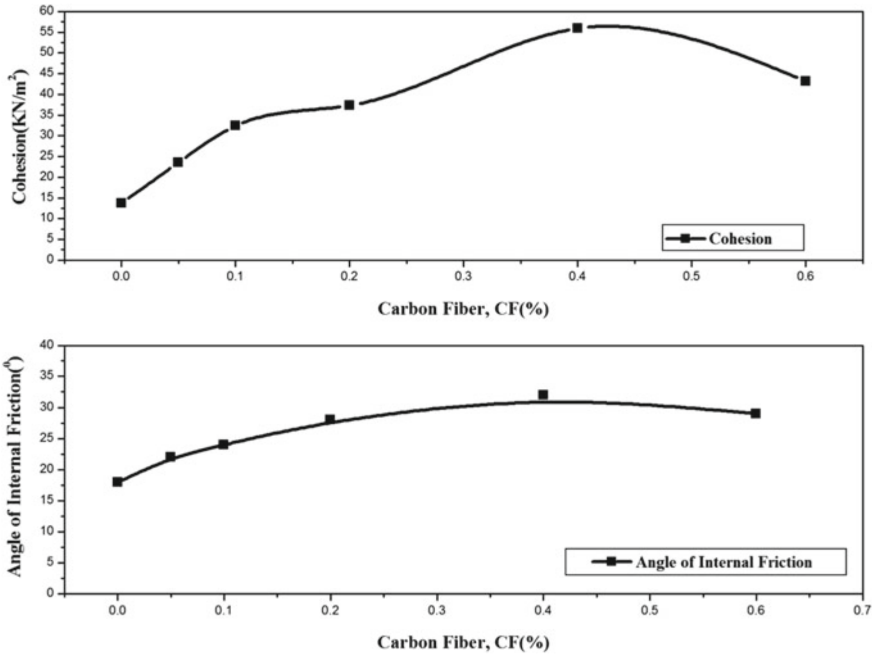

Fig. 5 Effect of carbon fiber on cohesion and angle of internal friction of soil

3.2 *Unconfined Compression Test*

The experimental outcomes of UCS tests are represented by stress–strain curve in Figs. 6 and 7. The conclusion can be attained from the figure that that inclusion of fiber into soil raises the unconfined compressive strength (q_u) and the strain. By increasing the percentage of addition of carbon fiber, the UCS of soil was increased up to 0.4%, further additions of fiber content decreased the UCS value. The carbon fiber effect on the UCS value was marginal when the percentage of fiber was beyond 0.4%. Thus, the maximum percentage of carbon fiber content was noted to be 0.4% and corresponding UCS value was 97.38 kPa. As compare to the native soil, it was observed that the UCS value of all soil specimens reinforced with carbon fiber occurs at maximum strain. Similarly, [14] mixed carbon fiber into soil with ten different percentages and obtained that carbon fiber improved the unconfined compressive strength till a certain strain percentage after which it started decreasing.

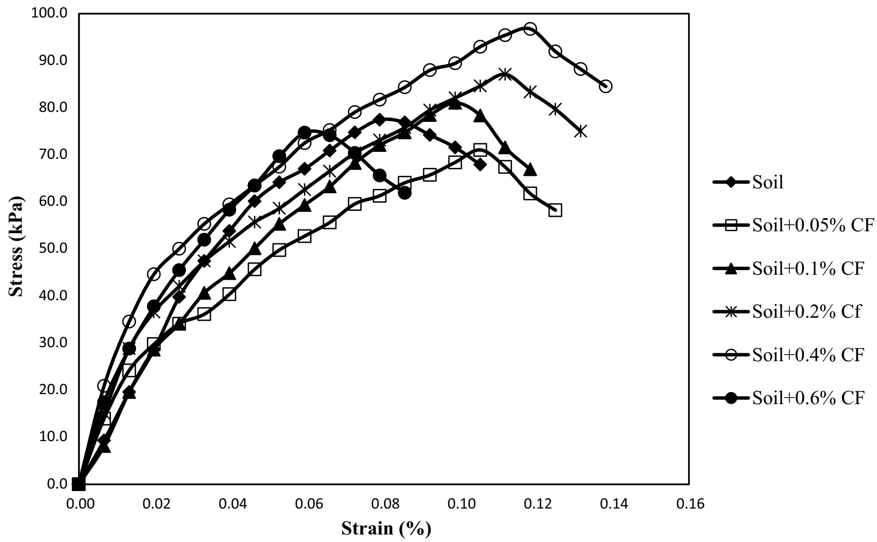

Fig. 6 Stress–strain curve for carbon fiber reinforced soil obtained from unconfined compression test

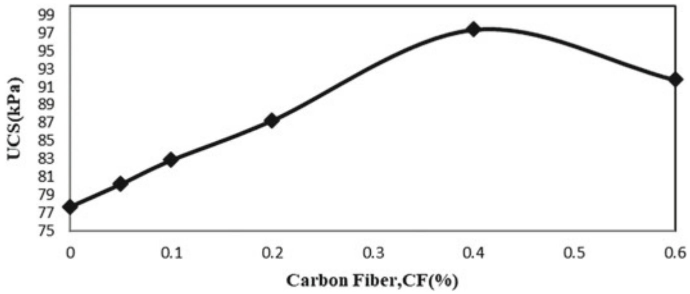

Fig. 7 Effect of carbon fiber on UCS of soil

3.3 CBR Test

Soaked CBR. The load–penetration curve which was plotted from soaked CBR tests for soil sample treated with various proportions of carbon fiber is given in Figs. 8, 9 and 10. The observations can be done from the figure that soaked CBR values of soft soil increase with inclusion of carbon fiber up to 0.4% of fiber content. The CBR value was raised from 1.63% for natural soil to 2.55% for reinforced soil, when 0.4% fiber was added.

Unsoaked CBR. On the basis of the test outcomes, the load–penetration curve and the variation of CBR w.r.t different carbon fiber content were plotted as shown in

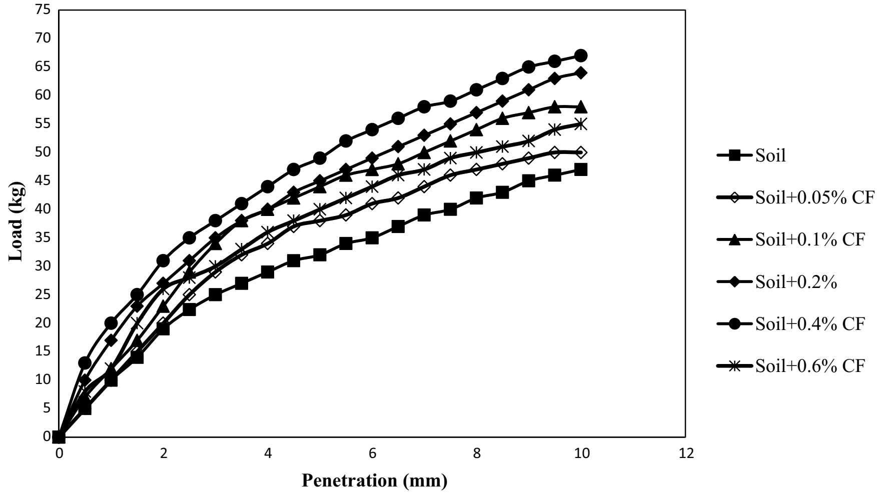

Fig. 8 Load–penetration curve for carbon fiber reinforced soil obtained from soaked CBR test

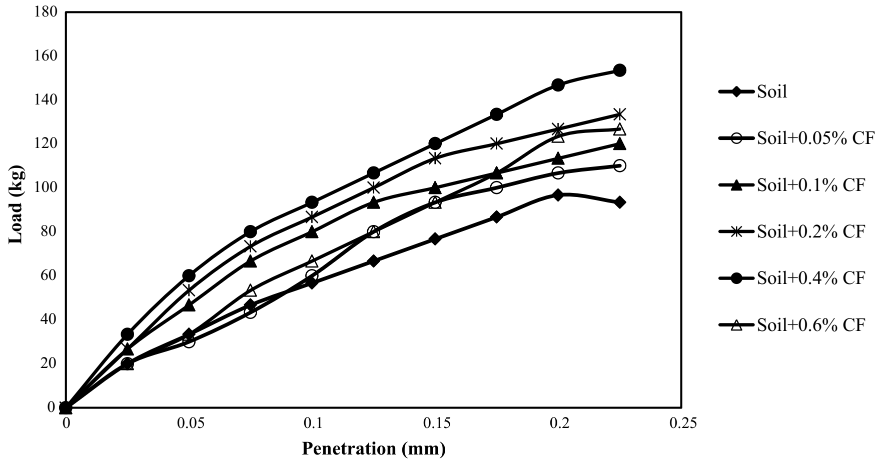

Fig. 9 Load–penetration curve for carbon fiber reinforced soil obtained from unsoaked CBR test

Figs. 9 and 10, respectively. It was clear from the figures that, over the increment of fiber percentage up to 0.4%, the CBR of soil increased, but upon further addition of fiber, the CBR value decreased. The unsoaked CBR of reinforced soil increased from 7.14% to 4.71% of native soil. Here, the optimum value of carbon fiber content was 0.4%.

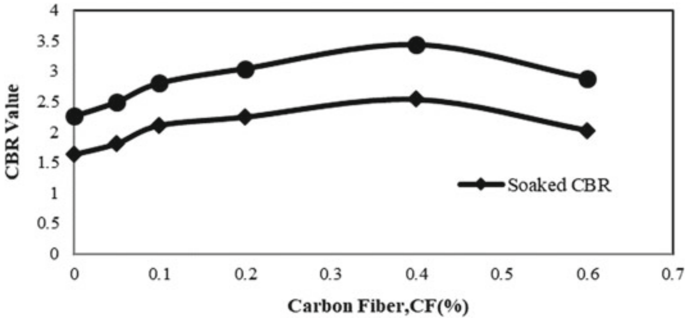

Fig. 10 Effect of carbon fiber on soaked and unsoaked CBR of soil

4 Conclusions

The study was taken to observe the appropriateness of carbon fiber blend as soul reinforcement for soft soil. The accompanying conclusions can be drawn from observations.

1. With increase in the percentage of carbon fiber, the shear strength of compacted fiber reinforced soil was obtained to increase up to a certain point, then decreased. The optimal value of shear strength was achieved for the mixture containing 0.4% carbon fiber.
2. With the increase in the carbon fiber content, initially there was corresponding increase of cohesion and angle of internal friction and then decreased with increasing fiber content. The maximum rise noted at the content of 0.4% fiber and a factor of increase of cohesion and angle of internal friction being 4.1 and 1.78, respectively, as compared to parent soil.
3. With inclusion carbon fiber, the UCS value of soil increased up to 0.4% fiber percentage, and then decreased. So, the maximum UCS value was found at 0.4% carbon fiber content and the value was found with increment of 1.25 factors in comparison with unreinforced soil.
4. Soaked and unsoaked CBR values increased till 0.4% of carbon fiber addition in different proportions to soil. In comparison with unreinforced soil, the both soaked and unsoaked CBR values of 0.4% of reinforced soil were raised to optimum. The value of both soaked and unsoaked CBR was improved by the factor 1.56 and 1.52, respectively.

On the basis of the result obtained, it is recommended that carbon fiber can be utilized as reinforced material and it has significant impact on ground improvement techniques. In line with our study, this technique can be considered as a viable and extremely practical option to bring about a revolutionary change in construction by enhancing the mechanical characteristics of soil of civil projects.

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Settlement Behaviour of Very Soft Soil Reinforced with Stone Columns

Sareesh Chandrawanshi and Rakesh Kumar

Abstract Construction on soft soils has always been a problem to geotechnical engineers. A review of the literature reveals that many techniques and methods have been developed to work with such soils. The technique of soft soil improvement by the installation of stone columns has become popular in recent past and has proven its application to many construction situations in soft soils. Construction of stone columns provides a new composite ground consisting of stiff stone column–soil matrix. Stone column reinforcement in the soft ground increases the bearing capacity and improves the settlement characteristics. They can be used to accelerate the rate of consolidation of soft soil deposits through a well-understood mechanism. They act as vertical drains that provide a shorter drainage path for excess pore water pressure to dissipate rapidly. In the present investigation, an attempt has been made to study the settlement characteristics of soft soil reinforced with the stone column. The main objective of the study was to investigate the settlement time behaviour of the stone column in very soft soil having undrained shear strength (c_u) ≈ 5 kPa under different bearing pressures and to verify the results of the experimental results with the analytical theory on consolidation rate of composite ground. A compactive effort is applied during the construction of stone column, and its average value is equal to 21.98 kJ/m^3. The settlement of the reinforced soft soil bed is reduced by 25.8% when reinforced with stone column diameter of 76.2 mm.

Keywords Soft soil · Stone column · Settlement characteristics · Consolidation rate

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1 Introduction

In the recent past, various techniques of ground improvement have been developed and applied due to the shortage of suitable construction site. Ability to enhance both soft clays and granular soils, by specific solutions suitable to meet any particular settlement or bearing capacity requirements resulted in wide acceptance of stone columns and established it as most widely used form of ground improvement technique [3, 13]. Its applications are widely published in literature which is used to treat various ground conditions successfully, including non-cohesive deposits [20], soft clays [8, 9], reclaimed land sites [18] and fill grounds [21].

Studies on stone columns are carried out extensively and contributed immensely to understanding its strength and settlement behaviour [1, 2, 4, 5, 7, 14, 15, 17, 19].

Stone columns constructed by vibro compaction were employed in grounds with predominantly cohesionless soils, which densify sunder the effect of horizontal/vertical vibrations imparted by the poker. Stone columns in cohesive soils are constructed by vibro replacement method, which includes the formation of a cylindrical void in the cohesive ground with the help of poker, backfilled by well-graded crushed stone aggregates. The laboratory testing programs differ with different researchers and still evolving to capture the load bearing-settlement phenomenon of reinforced clay bed. Rangeard et al. [17] carried out a study on sand column constructed in soft clay under constant consolidation pressure to assess the influence of compaction effort with low replacement ratio and concluded that the settlement reduction is observed significantly with higher compaction efforts.

Current research adopts an experimental laboratory programme on small-scale models for studying the effectiveness of stone columns constructed by vibro replacement technique (adopted with modification) in reducing settlements, formed in reconstituted soil with well-defined properties. Stone columns with varying replacement ratio and uniform compactive effort (applied to the stone aggregates during construction) were investigated.

2 Material Used

Kaolin clay is used to prepare soft ground for experimentation, and stone aggregates were used to construct the stone column.

2.1 Kaolin Clay

Kaolin or China clay is a form of industrial mineral with the chemical composition of $Al_2.Si_{2.}O_{5.}(OH)_4$. It has low shrinkage and swelling capacity, is inert and easy to mix and therefore is a well-established material used in the laboratory modelling. Use of

Table 1 Properties of kaolin clay

Property	Value
Specific gravity	2.62
Clay content, %	27
Silt content, %	73
Liquid limit, w_L, %	27
Plastic limit, w_P, %	18
Plasticity index, *P.I.*, %	9
Classification [11]	CL
Optimum moisture content, *O.M.C.*, %	14.25
Maximum dry unit weight, $\gamma_{d\,\max}$, kN/m^3	18.85

kaolin for experiments ensures reproduction of samples with repeatable properties. Kaolin was used as fine soil. Its basic properties are listed in Table 1.

2.2 *Stone Aggregates*

The stone aggregates of crushed basalt rock were used to form the stone columns. The stone aggregates were washed and sieved to obtain particles between 1.25 and 4.75 mm. Properties are shown in Table 2. Practically in the field, stone columns are constructed in diameters (d) between 0.6 and 1.2 m depending on the prevalent soil conditions [16].

Suitable stones aggregates of varying size (s) 2-75 mm are used, so that the ratio d/s falls in the range of 8 and 550 [12]. In the present research, the dimensions of

Table 2 Physical properties of stone aggregates

Property	Value
Specific gravity	2.72
D_{10}, mm	1.45
D_{30}, mm	1.9
D_{60}, mm	2.6
C_U	1.793
C_C	0.957
Percentage fines ($\leq$0.075 mm), %	0
Classification [11]	SP
Minimum dry unit weight, $\gamma_{\min}$, kN/m^3	14.43
Maximum dry unit weight, $\gamma_{\max}$, kN/m^3	17.86
Minimum size of aggregates, mm	1.18
Maximum size of aggregates, mm	4.75

Table 3 Details of experimental programme

Test series	Test description	Applied pressure	Method of construction of stone column	Compactive effort	Stone column diameter (mm)	Tests
TS-1	Soft soil bed ($c_u = 5$ kPa)	100,150 and 200 kPa	–	–	–	3
TS-2	Reinforced soft soil bed ($c_u = 5$ kPa, with stone column)	150 kPa	RP	E	25.4, 38.1, 50.8, 63.5, 76.2	5
					Total tests	8

RP = Replacement method
E = Compactive effort of 21.98 kJ/m^3 used while constructing stone columns

the stone aggregates are reduced in the same proportion to accurately model the behaviour of stone columns installed in the field; i.e. d/s ratio lies in between 5.3 and 60.7.

3 Experimental Programme

Details of experimental programme are provided in Table 3. A cylindrical test tank having internal diameter of 150 mm and height 230 mm is used to prepare soft clay bed of uniform properties. The soft clay bed was prepared by consolidating kaolin slurry, prepared at a water content of 1.5 times the liquid limit, then applying a pressure of 60 kPa. The laboratory small-scale model tests were carried out on this soft clay bed in unreinforced and reinforced condition. Reinforcement is provided by constructing single stone column of varying diameters from 25.4 to 76.2 mm. Replacement ratios corresponding to five different diameters of stone columns were 2.86, 6.45, 11.46, 17.92 and 25.81%, respectively.

4 Preparation of Clay Bed

The kaolin was mixed with water equal to 1.5 times of liquid limit, then mixed with the help of mechanical mixer and then kept in a sealed plastic bag for 4 h to obtain homogeneous properties. After this period, the soil was filled into the test mould. The bottom of the test mould is porous and before putting the kaolin sample filter paper was applied. This prevents the expulsion of kaolin from tiny holes and properly seals the soil. Filter paper is put between the porous plate and the sample from the top also. And then kaolin is subjected to a consolidation pressure of 60 kPa for 24 h. Kaolin is

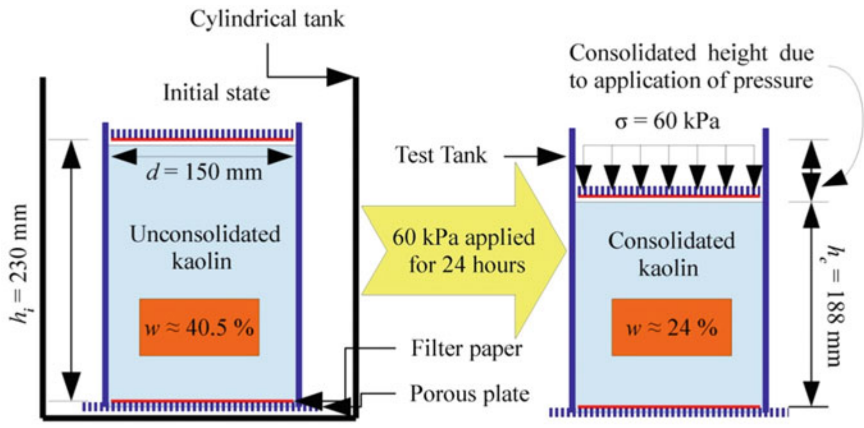

Fig. 1 Preparation of consolidated soft soil bed

Table 4 Physical properties of soft soil bed

Test bed type	Applied pressure (kPa)	Time duration (h)	Soft soil bed's undrained shear strength (c_u, kPa)
Type-1	60	24	5

stabilized under the sustained pressure, and its settlement is less than 0.01 mm/min after 12 h. Schematic representation of getting soft soil bed, type 1; i.e. of $c_u = 5$ kPa is shown in Fig. 1 (Table 4 provides the details of soft soil bed formed). Adopted methodology is taken from Chandrawanshi et al. [6].

5 Construction of Stone Column

A purpose of this study is to understand the settlement characteristics of the reinforced soft clay. The stone columns can be constructed by two methods, viz. replacement and displacement method with varying compactive efforts applied during its construction. In replacement method, soft clay was extruded out and replaced with stone aggregates which were compacted further with compactive effort. Whereas in displacement method the poker displaces the soil and intrudes, the tubular cavity is filled with stone aggregates and compactive effort is applied for achieving higher density. Displacement method gives a true representation of actual field installation technique, as it displaces the treated soil [9], but the technique was difficult to simulate in the laboratory with reduced scale models. In our experimental study, the stone columns were constructed by the displacement method. In the displacement method, the hollow pipe with cone was pushed into the centre of the test bed. It displaces the soil and cylindrical cavity is formed, which was then filled with stone aggregates

Table 5 Compaction specifications for making stone column by the effort, E

Diameter of stone column (mm)	Compacted height (mm)	No. of blows per layer	Drop height (mm)	Compactive effort (kJ/m^3)
25.4	50	1	100	23.21
38.1	25	1	100	20.62
50.8	25	2	100	23.20
63.5	25	3	100	22.28
76.2	25	4	100	20.62
Weight of ramming rod = 0.6 kg			Average	*21.98*

(Table 5 provides the details of compactive efforts applied on various diameters of stone columns).

6 Experimental Results

In test series TS-1, the soft soil of undrained strength 5.0 kPa was subjected to sustained bearing pressures of 150 kPa and their final settlement was noted. The test variable was area replacement ratio of the construction of stone columns. The settlement recorded with the stone column reinforced test beds are compared with the unreinforced bed.

It is noted that the installation of the stone column reduces the settlement. To quantify this reduction in settlement, a dimensionless parameter, settlement reduction ratio (S.R.R.) is introduced. It is defined as:

$$\text{S.R.R.} = \left[\frac{h_{\text{scb}} - h_{\text{rcb}}}{h_{\text{scb}}}\right] \times 100$$

where h_{scb} = settlement of the soft soil bed for a given bearing pressure, and h_{rcb} = settlement of the soft soil bed reinforced with stone column under the same bearing pressure.

Larger the value of S.R.R. better is the improvement in the stone column reinforced soft soil ground. In this chapter, the test results of various tests series are presented and discussed.

Table 6 Settlement of soft soil beds under different bearing pressure

Test number	Undrained shear strength (kPa)	Bearing pressure (kPa)	Initial height of the soft soil bed (mm)	Final height of the soft soil bed (mm)	Settlement (mm)
C1	5.0	100	188	179.29	8.71
C2	5.0	150	188	178.19	9.81
C3	5.0	200	188	177.49	10.51

6.1 Test Series TS-1: Settlement Characteristics of Soft Soil Beds

As given in Table 3, in Test series TS-1, the soft soil beds of kaolin having undrained shear strength $c_u = 5$ kPa were prepared. These beds were then subjected to three different bearing pressures (i.e. 100, 150 and 200 kPa). Settlement values were recorded with time, and the final thickness of the test bed was considered when the settlement rate reduces to less than 0.1 mm/hr. Different tests of test series TS-1 have been numbered as C1–C3. The test results of test series TS-1 are given in Table 6.

Settlement versus time graphs for soft soil bed ($c_u = 5$ kPa) under three bearing pressures (100, 150 and 200 kPa) was plotted and shown in Fig. 2.

6.2 Test Series TS-2: Settlement Characteristics of Reinforced Soft Soil Beds

The test Series, TS-2, was conducted on soft soil of undrained shear strength (c_u) of 5 kPa reinforced with stone columns of various area replacement ratio (a_s) constructed by replacement method. All the tests beds were subjected to constant bearing pressure of 150 kPa. Table 7 provides the details of the stone columns constructed by replacement method with different compactive efforts.

Figure 3 shows the settlement time graph for the five different tests of Test series, TS-2. The settlement reduction ratio (S.R.R.) for the different tests conducted in Test series, TS-2, are given in Table 8. Settlement versus time values obtained from analytical method (proposed by Han and Ye [10]) is plotted and compared with the experimentally obtained graph for stone column diameter of 26.1 mm as mentioned in Table 8 (Fig. 4 shows the comparison of experimental results with the analytical relation).

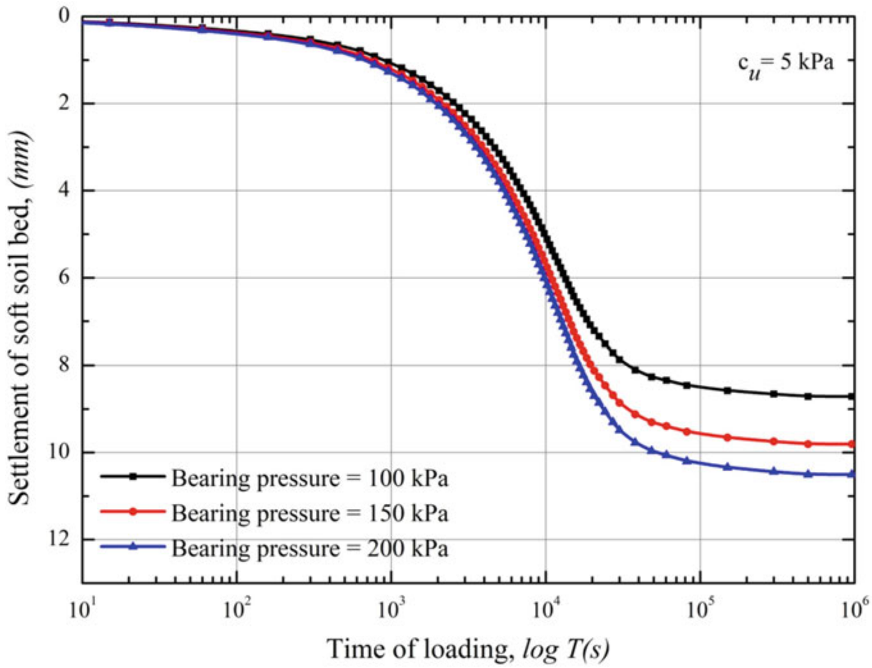

Fig. 2 Settlement versus time of loading for different bearing pressures ($c_u = 5$ kPa)

Table 7 Dimensions of stone columns, TS-2 ($c_u = 5$ kPa, RP, E)

Method of construction of stone column	Test No.	Initial diameter of stone column (mm)	Compactive effort applied (kJ/m^3)		Final diameter of stone column (mm)
RP	T1	25.4	E (21.98)	23.21	26.1
	T2	38.1		20.62	39.0
	T3	50.8		23.20	51.9
	T4	63.5		22.28	64.9
	T5	76.2		20.62	77.8

7 Conclusions

The research work conducted reports the settlement performance of a soft clay bed reinforced with the stone columns with different diameters constructed by constant compactive effort. Following conclusions were drawn:

1. The settlement values of a reinforced soft soil bed under a bearing pressure are dependent on the replacement ratio of stone column. More is the diameter of stone

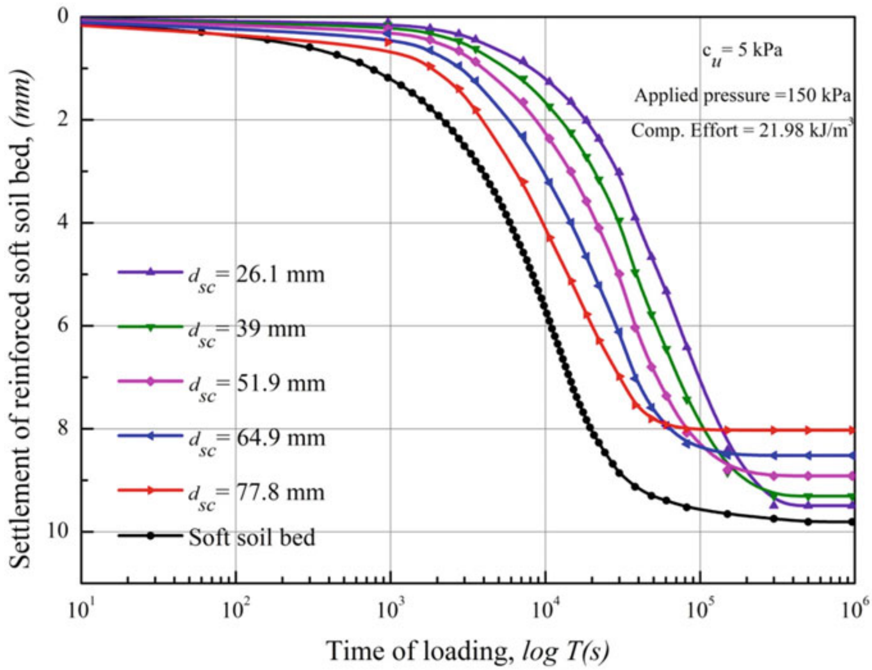

Fig. 3 Settlement versus time of loading for R.S.S. (c_u = 5 kPa, RP, E, 150 kPa)

Table 8 Settlement reduction ratio, (c_u = 5 kPa, RP, 150 kPa, E)

Method of construction of stone column	Test No.	Initial diameter of stone column (mm)	Final diameter of stone column (mm)	Compactive effort applied (kJ/m^3)	Settlement of soft soil bed (mm)	Settlement reduction ratio, S.R.R. (%)
RP	T1	25.4	26.1	E (21.98)	9.49	3.32
	T2	38.1	39		9.31	5.16
	T3	50.8	51.9		8.92	9.18
	T4	63.5	64.9		8.52	13.26
	T5	76.2	77.8		8.03	18.26

column, more is replacement ratio and smaller the settlement of the reinforced soft soil bed.

2. Final diameter of stone column is enlarged due to the application of compactive effort applied. The finalized diameter of 76.2 mm becomes 77.2 mm due to applied compactive effort.

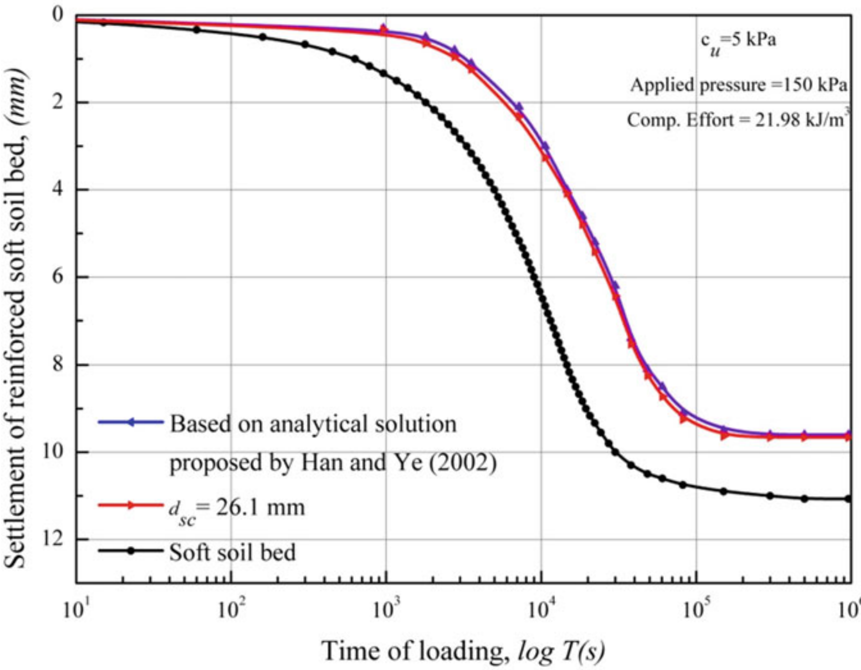

Fig. 4 Comparison done for S.C. of 26.1 mm (c_u =5 kPa, RP, 150 kPa, E)

3. Settlement of the reinforced soft soil bed can effectively controlled using larger stone columns. Reduction in settlement (i.e. SRR) is more for larger diameter of stone column.
4. Due to the compactive effort applied during the construction of stone column the adjoining soft soil structure is disturbed. This zone is named as smear zone. Permeability of the soft soil is modified, and its value is reduced in this zone. The analytical relation used for the comparison accounts for the reduced permeability in the smear zone and is showing good agreement with the experimental results.
5. Least settlement of the reinforced soft soil bed is achieved for the highest replacement ratio of stone column diameter of 76.2 mm that corresponds to 25.8%.

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Strength Improvement of Gandhinagar Soil Using Microfine Cement as Grout

Manank Shah, Manas Kumar Bhoi, and Kaushal Vora

Abstract Grouting technology is a widely accepted soil improvement technique for improving soil parameters like c and ϕ value. The use of microfine cement grout along with sodium silicate can easily deal with the problems associated with ground improvement, toxicity, and costliness of chemical grouts. Grout is injected under pressure into the material to be grouted until it fills the desired void or until the maximum specified pressure is attained and a specific minimum grout flow is reached. The purpose of this study is to investigate the proper grout mix using microfine cement (whose specific area is greater than the normal cement) and silica used in grouting. An attempt has been made to find the optimum grout mix design at various water content (10 and 20%) and cement content (5, 10, 15, 20, and 25%). The paper discusses about strength improvement of loose sandy soil and silty clay soil. The results shows that with the increase in microfine cement content, the strength of soil will increase, and with the increase in the water content, strength of soil decreases. In this study, direct shear test was performed to find shear strength of loose sandy soil. The grouting properties of microfine grout was found to be better as compared to the conventional grout.

Keywords Microfine cement · Grouting · Shear strength · Direct shear test · Sandy soil

1 Introduction

The building of structures on the ground with less structural stability often requires the soil to be improved in order to ensure the safety and the substructure stability of surrounding buildings [1]. Collapses and seepage of water are common phenomenon during the construction of tunnels and other underground works which is caused by soil having poor shear strength or permeability capacity. Phenomenon of liquefaction

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of soil can be observed during the events of earthquake due to the insufficient shear and bearing capacity of the soil [2, 5, 8].

Different methods like external vibration, vibro-flotation, compaction piles, compaction with explosives, replacement of soil, grouting, etc., can be used as the ground improvement techniques for the soil. Grouting is the method with several applications in the field of civil engineering and previously was not widely accepted as it required high skill and efficiency of labors to pursue the tasks. Grouting is a method in which the material is injected under pressure into the cavity until the desired volume of material around the hole is filled or until the maximum specified pressure is attained and a specific minimum grout flow is reached. Various grout slurries like cement-based grout slurry such as cement slurry and cement-sodium silicate slurry to chemical slurry such as lignin, acrylic, and urea or epoxy resins can be used according to the requirements of the project [6, 7].

As an alternative to chemical grouting of fine- and medium-grained sands, the use of grouts prepared with microfine cements has been proposed. Microfine cement is advantageous over ordinary Portland cement as it can provide a larger specific surface area [3]. Due to this larger specific surface area, the grout with microfine cement may provide better grout properties than the grouts with ordinary Portland cement. Due to its high fineness, the cement has extremely good retention when used for purpose of grouting in both the micro-cracks in concrete or rocks or voids in soil.

The grouting reduces pore size while filling the voids in the material and also create bond between soil particles, thereby improving the engineering properties such as strength, stiffness, and reduction of permeability. In this paper, an attempt is made to study the improvement in the strength of grouted loose sand by using microfine cement as grout.

2 Materials Used

The basic material used for soil testing are water, microfine cement (MFC), and sandy soil. MFC is similar to cement which when combined with water exhibits cohesive and adhesive properties that help in holding soil particle together to form a soil mass. In most of the cases, the microfine cement is available in slurry (cement + water) form and also available in powder form. Microfine cement particle is finer than normal cement particle and used to improve soil shear strength. The microfine cement used in this experimental program has following properties: fineness = 6000 cm^2/gm, specific gravity = 3.1, bulk density = 700-800 kg/m^3, particle size $D_{50} < 9$ μm, and $D_{95} < 22$ μm [4]. The soil used here is basically sand and has following properties.

3 Test Methodology and Results

3.1 *Test of Microfine Cement (MFC)*

The initial and final setting time of the microfine cement slurry was found out by the Vicat apparatus, and the results are given in Table 1.

For finding initial setting time, immediately place the test block with the non-porous resting plate, under the rod bearing the initial setting needle. Then, lower the needle and quickly release allowing it to penetrate in to the mold. In the beginning, the needle will completely pierce the mold, and after certain repetition of this procedure over a period of time, the needle fails to pierce the mold for $5 + 0.5$ mm. Record the period elapsed between the time of adding water to the cement to the time when needle fails to pierce the mold by $5 + 0.5$ mm as the initial setting time. For finding final setting time, replace the needle of the Vicat apparatus by the needle with an annular ring and lower the ring and quickly release. This process is repeated until the annular ring does not make an impression on the mold. Record the period elapsed between the time of adding water to the cement to the time when the annular ring fails to make the impression on the mold as the final setting time.

For the given soil with different water–cement ratio, the initial and final setting time obtained is given below in Table 2.

Table 1 Properties of the soil used in this experimental program

Properties	Values
Grain size D_{10}	0.17 mm
Grain size D_{30}	0.22 mm
Grain size D_{60}	0.3 mm
Uniformity coefficient(C_u)	1.76
Coefficient of curvature, (C_c)	0.95
Specific gravity (G_s)	2.62

Table 2 Initial and final setting time for different water–cement ratio

W/C ratio	Initial setting time (min)	Final setting time (min)
0.5	190	265
0.67	220	295
1	290	362
1.3	335	416
1.5	380	460
2	452	590

3.2 Direct Shear Test of MFC and Soil Mixture

Direct shear test has been performed to determine shear strength of soil and done so, using direct shear apparatus. The initial tests were performed on pure soil, and the shear strength was found out. The results obtained were compared with the shear strength of microfine cement silica grouted soil. The direct shear test on soil samples having different proportions of MFC and different water–cement ratio is given below.

A. **Direct shear test on soil sample having 5% and 25% MFC Content (Variation of Initial water content)**

1. An attempt has been made to find out the optimum moisture content for obtaining maximum shear strength of the soil mixture through direct shear test. Here, the 5 and 25% of MFC content are mixed with soil, and direct shear test is done on 7 days and 28 days of curing period. The obtained result of shear strength for 5 and 25% MFC content is given below in Fig. 1a, b, respectively. These results are for 100 kN/m^2 normal load applied in the direct shear test. The 25% MFC content is the optimum mix of our study of improvement of soil. For both the cases of MFC content of 5 and 25%, there is a decrease in shear strength with increase of the water content, and for

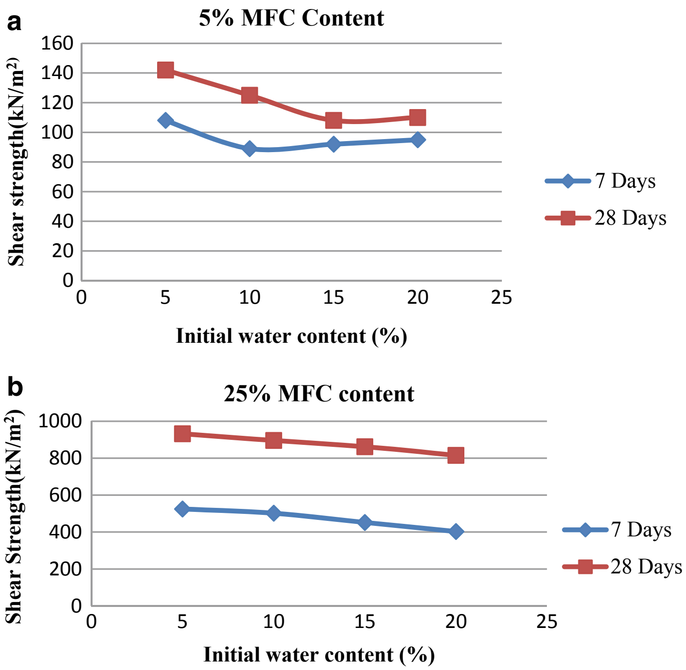

Fig. 1 **a** Graph showing shear strength versus initial water content at 5% MFC content. **b**. Graph showing shear strength v/s initial water content at 25% MFC content

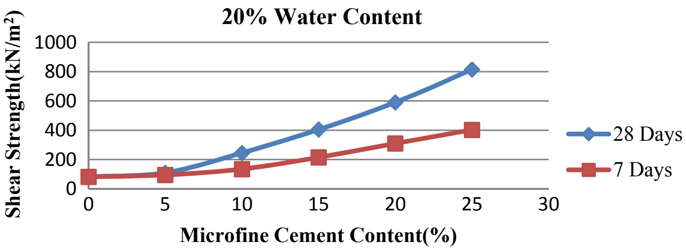

Fig. 2 Graph showing shear strength versus MFC content at 20% water content

both the cases, there is a general increase in shear strength with increase in the curing period. Here, for 5% MFC content, the shear strength decreases from to 110 kN/m^2 for 28 days of curing and for 7 days of curing, the trend is not very clear, and this can be attributed to not gaining sufficient strength in such a short period of time. For 25% MFC content, shear strength decreases from 932 to 815 kN/m^2 for 28 days of curing and from to 403 kN/m^2 for 7 days of curing. So, the 7 days curing period gives a clearer trend for 25% MFC content compared to 5% MFC content.

B. **Direct shear test on soil sample having 20% Initial water Content (Variation of MFC content)**

This graph (Fig. 2) shows the results of the direct shear tests conducted on soil mixture of different MFC content with variation from 0 to 25%, while keeping the water content at 20%. This graph shows the variation of shear strength of soil sample on 7 days and 28 days for different MFC content. This test results is for applied normal load of 100 kN/m^2. The increase in shear strength is steeper in case of 28 days curing (82 to 815 kN/m^2) for different MFC content compared to 7 days curing (82 to 403 kN/m^2), and this can be attributed to the higher molecular bonding strength with the increase in curing period.

C. **Direct shear test on soil samples after curing period of 28 days and for 20% Initial water Content (Variation of applied normal load)**

In this case, the graph (Fig. 3) shows the relative variation between shear strength and MFC content for applied normal load of 50, 100, and 150 kN/m^2. This direct shear test is conducted on mixed soil sample after 28 days of curing keeping the water content at 20%. The shear test is conducted for normal stress of 50, 100, and 150 kN/m^2. The graph clearly shows that the strength increment is steeper (82 to 625 kN/m^2 for 50 kN/m^2 normal stress and 105 to 952 kN/m^2 for 150 kN/m^2 normal stress) as the normal stress applied increased.

D. **In Direct Shear Test, Variation of Shear Stress versus Shear Strain for soil samples with different MFC content (5-25%)**

This figure (Fig. 4) shows the graph of shear stress versus shear strain obtained

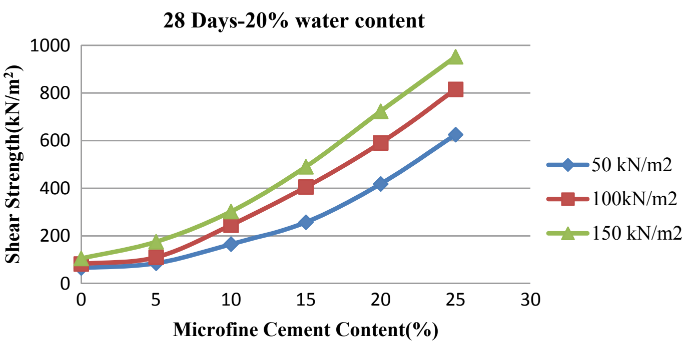

Fig. 3 Graph of shear strength versus MFC (%) at 20% water content and 28 days

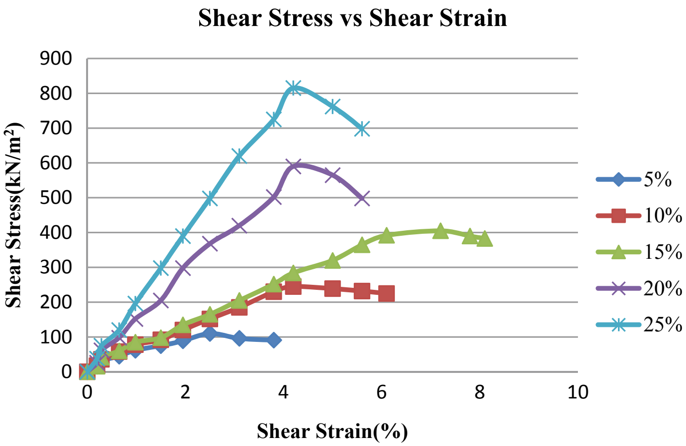

Fig. 4 Graph of shear strength versus shear strain (%) at different percentage of MFC content

from direct shear test conducted on soil samples having different microfine cement contents (5-25%) and water content of 5%. All the direct shear test conducted here is for the normal stress of 100 kN/m^2. The graph indicates the shear stress failure point for various MFC content; starting from 110 kN/m^2 at 5% MFC content to 815 kN/m^2 at 25% MFC content. It is also quite clear that with increase in MFC content, maximum shear strength is achieved at lesser shear strain.

4 Conclusions

The broad conclusion based on the analysis of the experimental results are given in this section. There is over eight times increase in the shear strength which can be seen at the age of 28 days by 25% replacement of soil by microfine cement at a specified W/C ratio. Microfine cement content is directly proportional with the shear strength of the soil (maximum shear strength at 25% MFC). For both the cases of MFC content of 5% and 25%, there is a general trend of decrease in shear strength with increase of the water content, and in both the cases, there is a general increase in shear strength with increase in the curing period. The increase in shear strength is steeper in case of 28 days curing for different MFC content compared to 7 days curing.

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Application of Non-woven Polyester Geotextile for Soil Improvement in Pavements

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Abstract The performance of flexible pavements are greatly affected by the type of subgrade, sub-base, and base course materials; the most important of these are the properties of soil subgrade, as it serves as the foundation for pavement. In India, around 8 lakh km^2 area is covered with poor subgrade soil covering central, some parts of southern region, and along the coastline. The pavement constructed over such soils will lead to greater thickness requirement, and it will also fail prematurely under heavy wheel load. In order to overcome such untoward situation, some ground improvement technique has to be adopted. This paper presents the effect of including non-woven polyester geotextile on the strength behavior of weak subgrade soil. The geotextile sheets are placed in single and multiple layers at various depths of soil subgrade and thereby determination of optimum combination and optimum position of reinforcement based on the California bearing ratio results are done. Greater improvement in CBR is observed for soil samples reinforced with geotextile in upper layers of subgrade as compared to lower ones with a maximum increase of 70% corresponding to double layer geotextile at 0.2 and 0.4 H depth from top of mold. It can be concluded that geotextile sheets can be considered as a good earth reinforcement material.

Keywords California bearing ratio · Compaction · Polyester geotextile · Reinforcement · Subgrade

1 Introduction

Roads are vital to link our communities and sustain the economy and quality of life in society. The overall development of any country cannot be thought off without

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effective road network, connecting hills to planes and cities to villages. India has a total road network of about 60 lakh km of which 80% consists of rural roads. Around 20% land area is covered with soils having low strength and high expansion potential. It is nearly impossible to provide stable construction platform over soft or weak soils. The intermixing of aggregate and fine soil will take place under heavy traffic load, leading to complete disintegration of pavement. Therefore, reinforced earth technique has to be adopted which includes mechanical or granular, chemical and physical methods. Reinforcing the cohesive soil with geosynthetics is the physical method of stabilization. One of the most common geosynthetic materials is geotextile. The World Bank has made it mandatory to use geotextile in construction projects funded by it. The improved performance of pavement reinforced with geotextile is attributed to three mechanisms namely increased bearing capacity, tensioned membrane effect, and confinement or lateral restraint. Many studies have been conducted on use of synthetic fibers [1–25], natural fibers, and geotextiles [26–44] on strength behavior of both cohesive and cohesionless soils. Several investigations have also been conducted on use of synthetic geotextiles [45–52] on granular soils, while limited studies have been found on fine-grained soils.

In the present study, effect of non-woven polyester geotextile on the strength behavior of weak subgrade soil is studied. The geotextile sheets are placed in single and multiple layers at various depths of soil subgrade and heavy compaction, and soaked CBR tests are conducted.

2 Materials

The following section presents the details of materials used in conducting laboratory investigations and their various properties.

2.1 Soil

The soil used in the present experimental tests is obtained from Meja (25.13°N, 81.98°E), Allahabad, Uttar Pradesh, India. The soil sample is collected by digging trial pits at 1 m below ground surface. The soil is air dried, broken into pieces with a wooden mallet and sieved through 4.75 mm sieve in the laboratory. Table 1 shows the various physical and mechanical properties of soil. The soil specimen is classified as clay of intermediate compressibility (CI) as per IS: 1498 (1970). Figure 1 shows the grain size distribution curve of soil.

Table 1 Physical and mechanical properties of soil

Soil properties	Value
Specific gravity	2.71
Grain size distribution	
(a) Gravel (%)	0.33
(b) Sand (%)	9.10
(c) Silt (%)	67.47
(d) Clay (%)	23.10
Atterberg's limits	
(a) Liquid limit (%)	36
(b) Plastic limit (%)	19
(c) Plasticity index (%)	17
Soil Classification (ISCS)	*Clay of intermediate compressibility (CI)*
Water content (%)	16.82
Free swell index (%)	32.54
pH value	7.55
Optimum moisture content (%)	13.60
Maximum dry density (KN/m^3)	18.80
Soaked CBR (%)	3.85

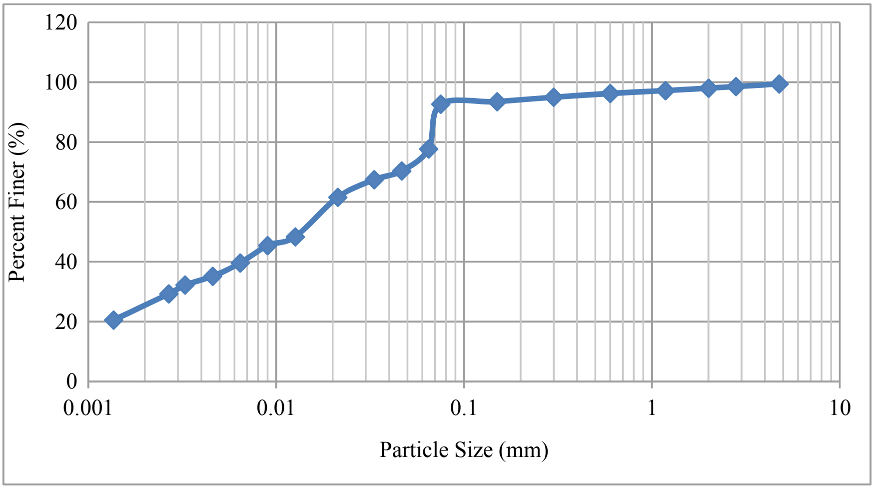

Fig. 1 Grain size distribution curve

2.2 *Geotextile*

A non-woven synthetic polyester geotextile having mass per unit area of 350gsm is used in the present study. The geotextile supplied by ocean non-woven Pvt. Ltd, New Delhi is shown in Fig. 2. The various index and strength properties of geotextile as provided by manufacturer are presented in Table 2.

3 Testing Program

The experimental program is carried out in two parts. First, the physical properties of soil (specific gravity, Atterberg's limits, ISCS classification, etc.) were determined, and then, a series of heavy compaction and soaked CBR tests are conducted in the laboratory based on the standard methods suggested by relevant parts of Indian Standards (IS): 2720 for 'Method of test for soils.' Various positions of geotextile reinforcement in soil subgrade are presented in Fig. 3. The term H signifies the total depth of soil in testing mold which is 12.73 cm in both CBR and compaction test.

Fig. 2 Non-woven polyester geotextile

Table 2 Index properties of geotextile

Properties	Unit	Test standard	Value
Type	–	–	Non-woven
Material	–	–	Polyester fibers
Mass per unit area	g/m^2	ASTM D 5261	350
Thickness	mm	ASTM D 5199	2.9
Breaking strength	kN/m	ASTM D 4595	11
Trapezoidal Tear strength	N	ASTM D 4533	280
CBR puncture strength	N	ASTM D 6241	1800

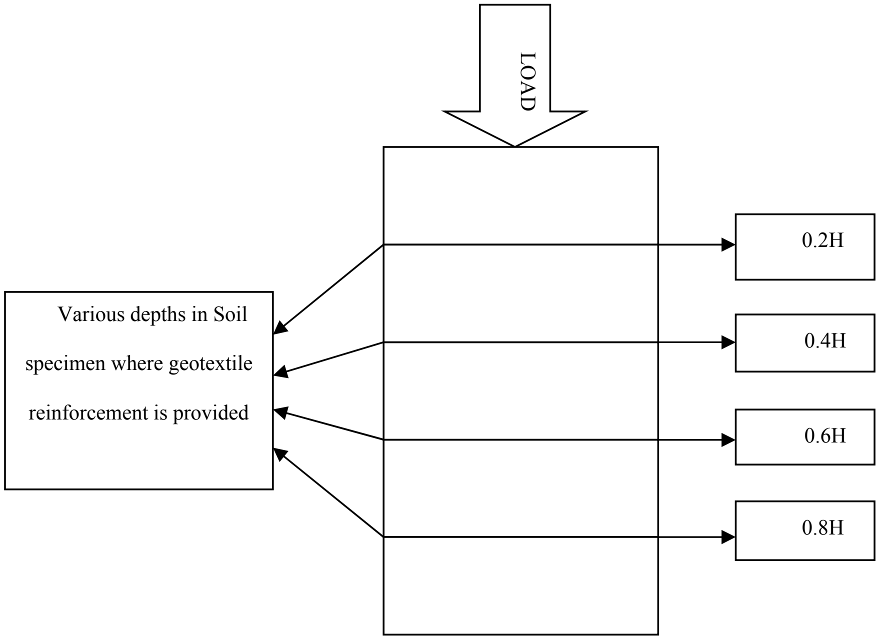

Fig. 3 Position of geotextile layer in soil

4 Results and Discussions

4.1 Heavy Compaction

The results of optimum moisture content and maximum dry density of soil sample reinforced with and without geotextile are shown in Table 3. Increase in MDD corresponding to single and double layer reinforcement as compared to virgin soil specimen is observed. The MDD for unreinforced soil is 18.60 kN/m^3 which increases to 19.40 kN/m^3, 18.61 kN/m^3, 19.44 kN/m^3, and 19.37 kN/m^3, respectively, for single layer of geotextile at 0.2, 0.4, 0.6, and 0.8 H depth from top of mold. These value further changes to 18.72, 19.09, 18.69, and 18.66 kN/m^3 for double layer of geotextile at 0.2 and 0.4 H, 0.2 and 0.6 H, 0.4 and 0.6 H, and 0.6 and 0.8 H depths, respectively. Reduction in MDD is observed for triple and four layer reinforcement which is even below the virgin soil with a minimum value of 18.38 kN/m^3 and 18.31 kN/m^3, respectively. The OMC results shows irregular trend, however, for most of the cases with increase in MDD reduction in OMC is observed and vice-versa. This increase in MDD for single and double layer reinforcement condition is due to greater compactness achieved with geotextile layers resulting in reduction of voids. However, with further increase in number of geotextile layers, this effect is overcome by lower specific gravity of polyester geotextiles as compared to soil as a result of which reduction in MDD is observed.

Table 3 OMC-MDD values of soil reinforced with non-woven geotextile

Depth of geotextile from top of mold	Experimental value	
	OMC (%)	MDD (kN/m^3)
Unreinforced soil	13.60	18.60
0.2 H	13.20	19.40
0.4 H	13.30	18.61
0.6 H	13.30	19.44
0.8 H	12.90	19.37
0.2 and 0.4 H	12.80	18.72
0.2 and 0.6 H	14.15	19.09
0.4 and 0.6 H	13.02	18.69
0.6 and 0.8 H	13.20	18.66
0.2, 0.4 and 0.6 H	13.90	18.50
0.2, 0.4 and 0.8 H	13.80	19.20
0.2, 0.6 and 0.8 H	14.10	18.38
0.4, 0.6 and 0.8 H	13.10	18.52
0.2, 0.4, 0.6 and 0.8 H	13.80	18.31

4.2 Soaked CBR

The California bearing ratio test results of soil reinforced with and without geotextile in various layers are presented in Table 4. The CBR for unreinforced soil is 3.85% which increases to 6.32%, 4.18%, 4.28%, and 6.09%, respectively, for single layer

Table 4 CBR values of soil reinforced with non-woven geotextile

Depth of geotextile from top of mold	CBR (%)
Unreinforced soil	3.85
0.2 H	6.32
0.4 H	4.18
0.6 H	4.28
0.8 H	6.09
0.2 and 0.4 H	6.55
0.2 and 0.6 H	5.28
0.4 and 0.6 H	5.93
0.6 and 0.8 H	5.85
0.2, 0.4 and 0.6 H	4.25
0.2, 0.4 and 0.8 H	4.85
0.2, 0.6 and 0.8 H	3.56
0.4, 0.6 and 0.8 H	4.57
0.2, 0.4, 0.6 and 0.8 H	4.32

of geotextile at 0.2, 0.4, 0.6 and 0.8 H depth from top of mold. These value further increase to 6.55, 5.28, 5.93, and 5.85% for double of geotextile at 0.2 and 0.4 H, 0.2 and 0.6 H, 0.4 and 0.6 H, and 0.6 and 0.8 H depths, respectively. Reduction in strength improvement is observed for triple and four layer reinforced cases as compared to single and double layers. The CBR is 4.25%, 4.85%, 3.56%, and 4.57%, respectively, for triple layer of geotextile at 0.2, 0.4 and 0.6 H; 0.2, 0.4 and 0.8 H, 0.2, 0.6 and 0.8 H and 0.4, 0.6 and 0.8 H depths from top of soil sample. The CBR further decreases to 4.32% for four layer reinforcement. Greater improvement in CBR is observed when geotextile sheets are placed in upper layers of soil subgrade as compared to lower ones. This is because for tensile strength of fabric to come into action certain amount of deformation is required in soil and this will always be more in upper layers of subgrade due to greater traffic load intensity as compared to lower layers.

5 Conclusions

Based on the experiments performed in laboratory to study the effect of non-woven polyester geotextile on the strength behavior of poor subgrade soil, the following conclusions are made. As the number of geotextile reinforcing layer increases, reduction in dry density is observed due to lower unit weight of polyester geotextile. The MDD for reinforced soil ranges from 19.44 to 18.31 kN/m^3. No fixed pattern is reported in OMC values but for majority of cases OMC decreases with increase in MDD and vice-versa. The range of OMC for reinforced soil is 12.90% to 14.15%. Maximum CBR of 6.32 and 6.55% is reported for single and double layer reinforcement when geotextile is placed at shallow depth of subgrade as against 3.85% for unreinforced soil. This is due to greater resistance to penetration of plunger in upper layers offered by geotextiles. With further increase in number of geotextile layers, reduction in CBR is observed. This is due to loss of integrity in soil system due to separation of soil layers completely from each other resulting in formation of more void spaces causing strength reduction. Double layer geotextile at 0.2 and 0.4 H depth from top of specimen is found to be the most optimum position of reinforcement when analyzed on the basis of reduction in thickness and cost of pavement and improvement in CBR. Thus, it can be concluded that use of non-woven geotextile in pavement subgrade results in economical pavement design with reduced structural section, saving costly base and sub-base aggregate materials and reducing frequent maintenance requirements.

These conclusions can be used effectively in locations where locally available soil has very low strength and Civil engineering structures such as pavement and embankment has to be constructed over it. The need for removal and replacement of soil will get eliminated and huge benefits in terms of aggregate saving and environmental protection caused by reduction in aggregate transportation, diesel consumption, noise, and air pollution will occur.

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Review on Suitability of Rice Husk Ash as Soil Stabilizer

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Abstract Modern era is an era of sustainable development. Utilizing industrial waste for the development works is an essential principle of sustainable development. Around 70 million hectare area of central India has black cotton soil, so there is a deficiency of stable construction site in these areas. Making these sites suitable for construction activities is a challenging task for geotechnical engineers. India is the second largest rice producer in the world. Around 24 million tons of rice husk and 4.4 million tons of rice husk ash (RHA) are produced annually in India. The effective disposal of such huge quantity is cumbersome, and thus, their use in some other fields must be looked into. The use of RHA as a soil stabilizer not only intensifies the required soil properties but also provides an effective way of its safe disposal. RHA contains rich amount of silica which have capability to replace the exchangeable ions present in clay minerals, thus reducing the shrinkage and swelling behavior of black cotton soil. The present study describes the available knowledge on use of RHA in soil stabilization purposes. The effect of RHA on various index and engineering properties of soil is also discussed based on previous researches in this field.

Keywords Black cotton soil · Rice husk ash · Solid waste · Stabilization

1 Introduction

Roads are the most vital component for the economic and social development of any country. Its importance further increases if the economy of a country is based on agriculture. India has a total road network of 6 million kilometers of which 79% consist of rural roads [1]. In India, more than 20% land area is covered with soils

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having low California bearing ratio (CBR) and shear strength values [2]. The pavement constructed over such poor subgrade soils will deteriorate significantly under heavy wheel load. So in order to overcome such situations, some soil reinforcement techniques have to be adopted which includes chemical, mechanical, and physical methods [3].

Soil stabilization by addition of RHA and lime is quite suitable for road pavements since it leads to cheaper construction and lesser maintenance costs, prevents environmental degradation, and preserves good quality aggregate materials which are available in limited quantity [4]. India is one of the largest producers of rice in the world. The major use of rice husk is in refractory brick industry, cement, and steel industry as a fuel, which ultimately produces rice husk ash as a solid waste.

This paper reviews and describes the available state of the art knowledge on use of RHA for soil stabilization purpose.

2 Rice Husk Ash

Rice husk is a by-product of the rice milling. About 110 million tons of husk per year are produced across the world. Due to its abrasive character, it is not suitable as animal feed. High lignin and ash content make it unacceptable for paper manufacturing. During milling of paddy, around 75% is obtained as rice and bran and rest 25% as husk [4]. The husk obtained is used as fuel for processing paddy in rice mills and for producing energy through direct combustion. Upon burning, 25% of husk gets converted into ash and remaining is volatile matter. This RHA is a great threat to the environment which can damage surrounding area and land where it is dumped. For its effective disposal, it can be used as pozzolanic material in concrete production, absorbents for oils and chemicals and for soil stabilization [5]. Tables 1 and 2, respectively, show the chemical composition and physical properties of RHA [6].

Table 1 Chemical composition of RHA

Constituent	Percentage (%)
Silica (SiO_2)	87.12
Aluminum oxide (Al_2O_3)	3.27
Ferric oxide (Fe_2O_3)	1.45
Calcium oxide (CaO)	2.79
Magnesia (MgO)	0.63
Loss in ignition	4.50

Source Subrahmanyam et al

Table 2 Physical Properties of RHA

Property	Value
Grain Size Distribution (% Finer than)	
4.75 mm	100.00
2.8 mm	97.21
2 mm	95.45
1.18 mm	93.87
600 μ	89.63
300 μ	79.42
150 μ	46.56
75 μ	17.90
Specific Gravity	2.07

3 Classification of RHA

Based on the carbon content, rice husk ash has been classified into two types [4].

(a) Low carbon RHA
(b) High carbon RHA.

3.1 Low Carbon RHA

The burning of rice husk at high temperature and shorter duration or low temperature and longer duration in controlled atmosphere produces low carbon RHA. Currently, various techniques including fixed bed reactor, fluidized bed reactor, rotary tube furnace, etc., have been developed for burning rice husk under controlled atmospheric conditions. The RHA thus produced contains amorphous silica and low graphite carbon content. Figure 1 shows the low carbon RHA.

3.2 High Carbon RHA

When burning temperature and duration both are high as in case of open heap burning of rice husk, the ash thus produced contains crystalline silica and high graphite carbon content. High carbon RHA is used less as open heap burning is prohibited in various countries due to the production of hazardous gases resulting in severe health issues. Figure 2 shows the high carbon RHA.

The index properties of RHA are shown in Table 3.

Fig. 1 Low carbon RHA

Fig. 2 High xarbon RHA

Table 3 Index Properties of RHA

Particulars	Description
Liquid limit (%)	Non-plastic
Plastic limit (%)	Non-plastic
Maximum dry density (kN/m^3)	6.90
Optimum moisture content (%)	42.20
CBR (%)	16.12
Color	Gray
Shape texture	Irregular
Mineralogy	Non-crystalline
Odor	Odorless
Appearance	Very fine

4 Review of Literature

This section presents the state-of-the art knowledge about use of RHA in soil stabilization.

Rao et al. [7] studied the effect of RHA, lime and gypsum on the index and engineering properties of marine clay. The soil sample was classified as clay of high compressibility (CH) as per Indian standard classification system (ISCS). The soil sample was mixed with 5% lime, RHA (10–40%), and gypsum (2–5%) and tested at various curing periods (4–28 days). The optimum mix was found to be 5% lime +20% RHA +3% gypsum at 28 days curing, where maximum improvement in soil properties is obtained. Maximum improvement in UCS and CBR was 548% and 1350%, respectively. It was concluded that the utilization of industrial wastes like RHA, lime, and gypsum is an alternative to reduce the construction cost of roads particularly in the rural areas of developing countries.

Sabat [8] studied the effect of polypropylene fiber on geotechnical properties of RHA and lime stabilized expansive soil. The soil sample was classified as clay of high compressibility (CH) as per ISCS. As percentage of polypropylene fiber increases in the RHA–lime stabilized soil, MDD decreases and OMC increases. The UCS and CBR of the RHA–lime stabilized soil increased up to 1.5% addition of polypropylene fiber and decreases with further increase in fiber content. The hydraulic conductivity of RHA–lime stabilized soil increases as the percentage of fiber increases, whereas swelling pressure decreases. The optimum proportion of soil: fiber: lime: RHA was found to be 84.5:1.5:4:10 where maximum improvement was reported.

Rao et al. [7] studied the effect of adding potassium chloride (KCI) and RHA on the strength and swelling properties of clayey soil. The soil sample was collected from Amalapuram, East Godavari District, Andhra Pradesh, and was classified as clay of high compressibility (CH) as per ISCS. The KCI content varies from 0 to 3%, whereas RHA content varies from 0 to 16%. The maximum reduction in plasticity index; swell potential; swelling pressure; and improvement in UCS was observed at 1% KCI and 12% RHA content. The UCS of expansive soil has increased by 515% at 28 days curing as compared to unreinforced specimen. It was concluded that the utilization of industrial wastes like RHA is an alternative to reduce the construction cost of roads particularly in the rural areas of developing countries.

Kuity and Roy [9] studied the effect of using waste material and geogrid on the strength behavior of poor subgrade soil. The soil sample was collected from BESUS, West Bengal, and classified as clay of low compressibility (CL) as per USCS. The waste materials; pond ash (PA); and rice husk ash (RHA) and lime as additive was mixed in different proportions with soil. Geogrids are placed at middle of mold in single layer and at one-third height from both top and bottom of mold. It was found that soaked and unsoaked CBR values of mix increased by 1.22 to 3.72 times and 1.16 to 2.06 times, respectively, by adding PA, RHA, and lime. For single layer of geogrid in soaked and unsoaked conditions, improvement varies from 7.76 to 12.84 and 1.91 to 7.88 times, respectively. For double layer of geogrid in soaked and unsoaked conditions, improvement varies from 7.49 to 18.21 times and 2.16 to

9.29 times, respectively, in comparison with virgin soil. It was concluded that use of geogrid in soil subgrade enhanced the CBR value significantly.

Anupam et al. [10] studied the effect of mixing fly ash (FA), bagasse ash (BA), rice husk ash (RSA), and rice straw ash (RSA) on the shrinkage limit, OMC-MDD and soaked California bearing ratio (CBR) value. The soil sample was classified as clay of low compressibility (CL) as per USC system. Waste materials were mixed in increment of 5% upto 35% by part replacing the subgrade soil. It was found that shrinkage limit increased with increase in percentage of waste material. Heavy compaction test showed an increase in OMC and decrease in MDD with increase in percentage of waste material in the mix. Soaked CBR test conducted at different curing periods (i.e., 3, 7, 14, and 28 days) showed an increase in CBR value upto 25% for FA, BA, RHS, and upto 20% for RSA and beyond that it decreased. Thus, there will be considerable reduction in the thickness requirement of pavement leading to cost savings.

Shrivastava et al. [11] studied the effect of adding lime and RHA on various geotechnical properties of black cotton soil. The soil sample was collected from Bilhari area of Jabalpur (Madhya Pradesh). The RHA content varies from 5 to 20%, whereas lime content was kept constant at 5%. The test results indicate significant improvement in California bearing ratio (CBR) and unconfined compressive strength (UCS), whereas differential free swell decreases with increase in RHA content. Maximum improvement of 287.5 and 30% in CBR and UCS, respectively, was reported corresponding to 20% RHA content. It was concluded that silica present in RHA reacts with lime to form a binding material and enhances the soil properties considerably.

Kumar and Preethi [12] studied the effect of RHA and lime addition on strength properties of clayey soil. The soil sample was classified as clay of intermediate compressibility (CI) as per ISCS. The RHA (5–15%) and lime (3–9%) are mixed in soil sample individually and in various combinations and tested for CBR and UCS for different curing periods (4, 7, and 14 days). It was found that 6% addition of lime shows maximum improvement in the CBR and UCS as compared to other combinations for 14 days curing. Maximum increase of 89% in UCS and 378% in CBR with respect to uncured sample is obtained. It was concluded that lime addition changes the soil condition from poor to excellent, thus significantly reducing the thickness requirement of pavement.

Roy [13] studied the effect of RHA and cement on the strength properties of clayey soil (CH). The soil sample was mixed with (5–20%) RHA and 6% cement. It was found that with increase in RHA content, reduction in maximum dry density (MDD) and increase in optimum moisture content (OMC) is reported. The CBR and UCS show maximum improvement of 106% and 90%, respectively, at 10% RHA and beyond that it decreases. It was concluded that silica present in RHA reacts with the lime present in cement, thus forming binder material and improves the engineering properties of poor subgrade soil.

Akinyele et al. [14] studied the effect of adding RHA in various percentages (i.e., 2, 4, 6, 8, and 10%) on the index and shrinkage properties of poor lateritic clayey soil. The soil sample was collected from Buruku, Nigeria, and is classified as clay of low compressibility (CL). It was found that as the percentage of RHA increases, reduction in plasticity index and linear shrinkage is reported. It was concluded that the RHA is an effective stabilizing agent for subgrade in road construction and for backfilling in retaining wall, but the mix should be controlled not to exceed 10%.

Kumar and Gupta [15] studied the effect of adding pond ash (PA), rice husk ash (RHA), polypropylene fibers, and cement on the compaction and strength behavior of clayey soil. The soil sample was collected from Jalandhar, Punjab, and was classified as clay of low compressibility (CL) as per USCS. Pond ash and rice husk ash content vary from 30–45% and 5–20%, respectively, whereas fibers and cement content vary from 0 to 1.5% and 0 to 4%, respectively. To study the effect of curing, specimens are cured for 7, 14, and 28 days. Modified compaction, unconfined compressive strength (UCS), and split tensile strength (STS) tests are conducted. It was found that OMC increased and MDD decreased with increase in RHA content. Fiber addition increased the UCS and STS values with reduction in post-peak strength loss and crack formation. The optimum value of pond ash and rice husk ash content in mixture was found as 40% and 10%, respectively. Thus, it was concluded that admixtures can be used as lightweight fill materials.

Raj et al. [16] studied the effect of RHA addition in various proportions (i.e., 5, 10, 20, 30, 40, 50, and 80%) on the engineering properties of clayey and alluvial soil. As the RHA proportion increases, reduction in liquid limit, free swell index, and OMC is observed, whereas significant improvement in MDD, CBR, and angle of internal friction was reported for both soils. Maximum increase of 160% and 55%, respectively, in CBR was observed for alluvial soil and clayey soil when mixed with 80% RHA.

Prakash et al. [17] studied the effect of adding RHA in various percentages (i.e., 5, 10, 15, and 20%) on strength properties of poor subgrade soil. As the percentage of RHA increases, reduction in liquid limit and MDD was reported, whereas OMC increases. The CBR value increases upto 10% RHA content and beyond that it decreases. It was concluded that silica present in RHA is capable to replace the exchangeable ion present in clay mineral, thus can reduce shrinkage and swelling property of clay minerals.

Ghutke et al. [18] studied the effect of RHA addition on index and strength properties of black cotton soil. The RHA was mixed in various percentages by weight of soil (i.e., 4, 8, 12, and 16%). It was found that liquid and plastic limit first increases upto 4% RHA addition and then start decreasing. Specific gravity and MDD decrease as the percentage of RHA increases. CBR value increases up to 12% RHA content and beyond that it decreases. It was concluded that optimum ash content in soil was 12% where maximum improvement in properties is occurring.

5 Conclusions

The rice husk ash is a pozzolanic material which contains large amount of silica with little or no cementious properties. From the previous research on the potential of

RHA as a soil stabilizer, we can conclude that RHA is generally not used alone as a soil stabilizer, but can be used with small amount of cement or lime. Lime reacts with silica present in RHA and forms cementious compounds and hence enhances the geotechnical properties of expansive soils. The RHA can also be used with small amount of soil reinforcing materials such as polypropylene and waste coir fibers. The optimum amount of stabilizer with the combination of different materials is as follows:

1. 10% RHA with 6% cement gives maximum improvement in CBR and UCS values of soil.
2. 10% RHA with 6% lime results in maximum increase in CBR and shearing properties of the soil.
3. 20% RHA and 10% stone dust results in maximum improvement in the strength characteristics in soil.
4. 8% RHA and 1% core fiber results in maximum improvement in shearing characteristics of soil.

Thus, it can be recommended to use RHA as soil stabilizer which not only reduces construction cost of pavement due to significant improvement in CBR, but also provide its safe disposal, which otherwise may cause substantial environmental degradation. However, more research work is required to fully understand the working mechanism of such agricultural wastes for soil stabilization. Hence, it is time to support rice husk ash for the sake of better environment.

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Analysis of Strength Properties of Lime Stabilized Black Cotton Soil with Phosphogypsum

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Abstract Expansive soils are those soils which pose a very serious problem when they are subjected to moisture variation. Phosphogypsum is one of the industrial waste by-products that can be utilized for the purpose of soil stabilization. In this study, the strength of lime stabilized soil added with phosphogypsum for immediate 7 and 14 days of curing periods is compared with the strength of phosphogypsum amended black cotton soil for immediate 7 and 14 days of curing. From unconfined compressive strength test, it was found that 6% of phosphogypsum is the optimum content, that imparts the maximum strength to the soil that when it is used alone. The combination of 3% of lime and 6% of phosphogypsum gives the maximum strength, and the strength obtained is relatively higher than the strength of the black cotton soil stabilized with phosphogypsum alone.

Keywords Phosphogypsum · Unconfined compressive strength · Soil stabilization

1 Introduction

Black cotton soil, a cohesive soil, is considered as a problematic soil for civil engineers. It has characteristics of swelling during rains and shrinking during summer because of the presence of clay mineral montmorillonite. Soil stabilization is the process or technique of improving the engineering properties of the soil. The properties of soil can be improved either by mechanical stabilization or chemical stabilization. Mechanical stabilization is the methodology which involves the improvement in the properties of soil by changing its gradation without the addition of agents. The methodologies include compaction, blasting, dynamic compaction, preloading, sand drains, etc. [1], and chemical stabilization involves the reaction between the stabilizer and the soil minerals in order to achieve the desired effect. The most commonly used materials for soil stabilization include cement, lime, fly ash, blast furnace slag, etc., and the soil material which is stabilized is having higher strength and lower

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permeability and compressibility. Cement and lime stabilization are the most widely used methods of chemical stabilization. Phosphogypsum is a kind of industrial waste by-product generated from the production of phosphoric acid by treating phosphate ore with sulphuric acid [2]. Nearly about 4.5–5 tons of phosphogypsum is generated per ton production of the phosphoric acid. Large volume production of phosphogypsum poses a very serious problem of disposal. If it is disposed of in open yards, it may cause a threat to the environment [3]. The beneficial use of phosphogypsum like it is used as a raw material in the cement manufacturing industry, as a substitute for mineral gypsum as well as for soil stabilization is more effective in solving the problem associated with the disposal of phosphogypsum [4, 5].

2 Materials and Methodology

The materials which are utilized in this study include natural black cotton soil which is needed to be stabilized, lime and phosphogypsum.

2.1 Black Cotton Soil

The black cotton soil used in this study was collected from Raichur District of Karnataka, India. The properties of the soil were tested according to the Bureau of Indian Standards in the laboratory, and the results that were obtained are given in Table 1

Table 1 Soil properties

S. No.	Properties	Values	Standards
1	Liquid limit	83%	IS 2720 Part 5
2	Plastic limit	37%	
3	Plasticity index	46%	
4	Shrinkage limit	8%	IS 2720 Part 6
5	Specific gravity	2.4	IS 2720 Part 3
6	Maximum dry density	14.95 kN/m^3	IS 2720 Part 7
7	Optimum moisture content	27%	
8	UCC strength	20.6 kN/m^2	IS 2720 Part 10

Table 2 Properties of lime

S. No.	Composition	Percentage
1	$Ca(OH)_2$	91.21
2	Silica	0.96
3	Magnesia as MgO	0.94
4	Aluminium as Al_2O_3	In traces
5	Fe_2O_3	In traces
6	Mesh	250

2.2 Lime

The addition of lime quickly improves the condition of soil during construction and can contribute to the early and late strength of the stabilized soil. Addition of lime can cause three major improvements in the soil.

- Soil drying—Reduction in soil moisture content.
- Soil modification—Reduction in plasticity, improvement in compaction characteristics and gain in early strength.
- Lime stabilization—Increasing long-term strength and reduction in swell potential.

The lime used in this study was obtained from the Mysore agency, Bengaluru, Karnataka. The lime composition as provided by the manufacturer was as follows (Table 2).

2.3 Phosphogypsum

The phosphogypsum essentially a "calcium sulphate" is generated as a waste by-product from the phosphoric acid manufacturing plants by the reaction of rock phosphate with the sulphuric acid. About 100–280 million tons of phosphogypsum are estimated to produce annually worldwide. There is 11 number of phosphoric acid manufacturing units located in 7 states, namely Andhra Pradesh, Gujarat, Kerala, Maharashtra, Orissa, Tamil Nadu and West Bengal. The phosphogypsum generation in India is about 11 million tons per annum [4].

The phosphogypsum used in this study was collected from Vaikash Exim, Tuticorin, Tamil Nadu. It is a grey coloured, fine-grained material and its chemical composition as provided by the manufacturer is as follows (Table 3).

Table 3 Composition of phosphogypsum

S. No.	Composition	Percentage
1	Moisture	10–15
2	CaO	32.8
3	SO_3	45.8
4	Total phosphate as P_2O_5	0.30
5	Water soluble phosphate as P_2O_5	0.08
6	Fluoride	0.46
7	Water of Hydration	19.50
8	MgO	0.10
9	Na_2O	0.10
10	K_2O	0.04
11	Fe_2O_3	0.01
12	Al_2O_3	0.0
13	SiO_2	1.20
14	Cl	0.004
15	Organic matter	0.15

2.4 Methodologies

The soil which was collected from the site was subjected to cleaning, air drying and pulverization, and then it was sieved through designated sieves as per the requirement of the test. The three different trial percentages of phosphogypsum, i.e. 3, 6 and 9%, were selected. Each of this percentage of phosphogypsum was mixed with the soil, and the tests were carried out as per the Bureau of Indian Standards. The compaction test was conducted according to the Bureau of Indian Standards for the combination of Soil + Phosphogypsum and Soil + Lime + Phosphogypsum. The optimum phosphogypsum content was determined by conducting unconfined compressive strength on three different percentages of phosphogypsum like 3, 6 and 9% and the phosphogypsum content which gives the maximum UCC strength is selected as the optimum phosphogypsum content. The optimum phosphogypsum content in enhancing the maximum UCC strength obtained was 6% [6].

Similarly, the optimum lime content was determined by conducting unconfined compressive strength test on three different percentages of lime like 3, 6 and 9% and 3% of lime is obtained as optimum lime content in imparting the maximum UCC strength. The expansive soil was mixed with each of these three different percentages of phosphogypsum by dry weight of the soil, and the specimens for UCC test were prepared according to the maximum dry density and optimum moisture content obtained in the compaction test. The specimens were casted in a steel mould of 38 mm diameter and 76 mm of height [2].

The samples which were prepared were kept for immediate testing, 7 and 14 days of curing in airtight-sealed polythene bags in order to prevent the loss of moisture.

The specimens kept for curing were subjected to unconfined compressive strength test at their specific curing days. The specimens were made by using the combination of optimum lime content stabilized soil, i.e. 3%, lime admixed soil to each percentage of phosphogypsum like 3, 6 and 9%. The specimens were prepared according to their maximum dry density and optimum moisture content obtained in the compaction test. The specimens prepared were kept for immediate testing, 7 and 14 days of curing. The samples which were kept for curing were tested at their specified curing days. The UCC test was conducted at the strain rate of 1.25 mm/min. To determine the performance of the additives, the results of the unconfined compressive strength tests of lime stabilized soil admixed with phosphogypsum are compared with the soil stabilized with the phosphogypsum alone.

3 Results and Discussion

The results of the unconfined compressive strength test conducted for the specimens at their different curing periods and phosphogypsum content were discussed in this section. The effect of phosphogypsum and the combination of lime and phosphogypsum on the unconfined compressive strength of the soil are as shown in Figs. 1 and 2, respectively.

Figure 1 shows the variation of UCC strength of soil stabilized with different percentages of phosphogypsum. From the test, we can observe that the addition of phosphogypsum increases the strength up to 6%, thereafter which there is a reduction in the strength of the soil. The UCC strength of the soil to be stabilized is 20.6 kPa. The strength increases from 20.6 to 49.4 kPa; for 3% of phosphogypsum and for 6% of phosphogypsum, it increases from 49.4 to 50.8 kPa, thereafter which the UCC strength decreases to 27 kPa for 9% of phosphogypsum for 14 days of curing. The similar trend of change in the strength is observed for 7 days and immediate testing.

Similarly, from Fig. 2, it can be seen that the maximum unconfined compressive strength is obtained for the combination of 3% lime stabilized black cotton soil with 6% of phosphogypsum and is of the magnitude 21.6, 68.8 and 73.8 kPa, respectively,

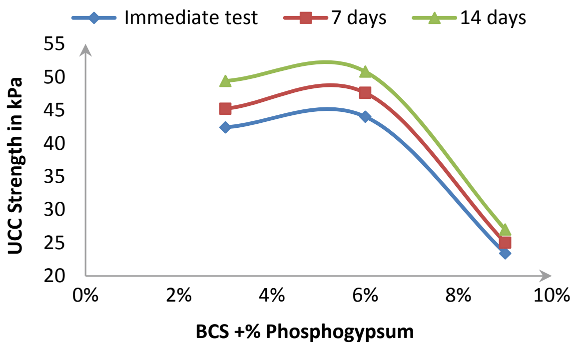

Fig. 1 Variation of UCS of soil with different percentages of phosphogypsum

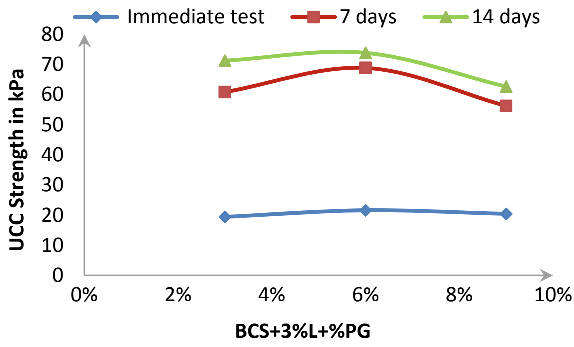

Fig. 2 Variation of UCS of 3% lime stabilized soil with 3 different percentage of phosphogypsum

for immediate testing, 7 days and 14 days of curing. However, the strength achieved is more in the case of lime stabilized soil with phosphogypsum when compared to the soil stabilized with the phosphogypsum alone at their optimum content. The magnitude of UCC strength achieved in the former case is 73.8 kPa and in the latter case 50.8 kPa, respectively, at 14 days of curing.

Figure 3 shows the variation of UCC strength of the phosphogypsum amended soil with different curing periods. The strength increases from 20.6 to 44 kPa on immediate testing, from 44 to 47.6 kPa for 0–7 days of curing and from 47.6 to 50.8 kPa for 7–14 days of curing for 6% of phosphogypsum content. The increase in strength is almost similar for the soil stabilized with 3% of phosphogypsum when compared with the soil stabilized with 6% of phosphogypsum with the increase in curing period. However, the increase in the strength of the soil is observed with the increase in curing periods irrespective of the phosphogypsum content with the increase in curing period.

Figure 4 shows the variation of UCC strength of 3% lime stabilized soil amended with different percentages of phosphogypsum with the increase in curing periods. The graph shows a similar trend of increase in the strength of the soil as that of the phosphogypsum stabilized soil. The strength increases from 20.6 to 21.6 on immediate testing, from 21.6 to 68.8 for 0–7 days of curing and from 68.8 to 73.8 for

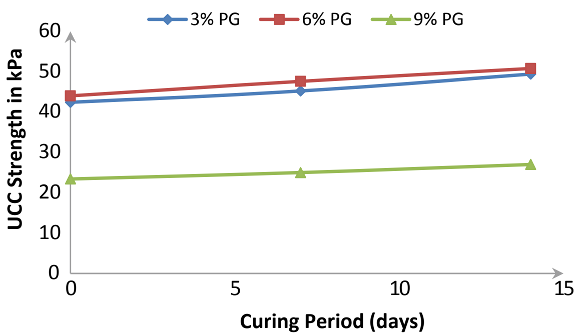

Fig. 3 Variation of UCC strength of phosphogypsum amended soil with the increase in curing periods

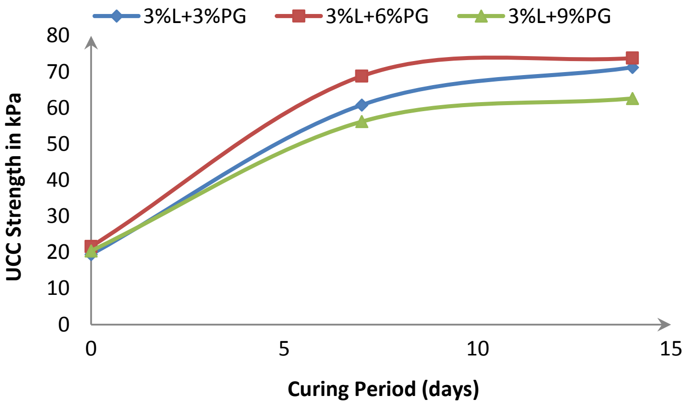

Fig. 4 Variation of UCC strength of 3% lime stabilized soil admixed with different percentage of phosphogypsum with the increase in the curing period

7–14 days of curing, respectively, for optimum lime content stabilized soil amended with 6% of phosphogypsum.

Figures 5 and 6 show the variation of percentage change in the UCC strength for phosphogypsum amended soil and the lime stabilized soil admixed with phosphogypsum with the increase in curing periods, respectively. The percentage increase in the strength for immediate testing for phosphogypsum admixed soil is more, whereas the percentage increase in strength decreases with the increase in curing period. The UCC strength is increased by 100% on immediate testing, after which, the percentage increase in strength decreases to 8% and 6%, respectively, for 7 days and 14 days of

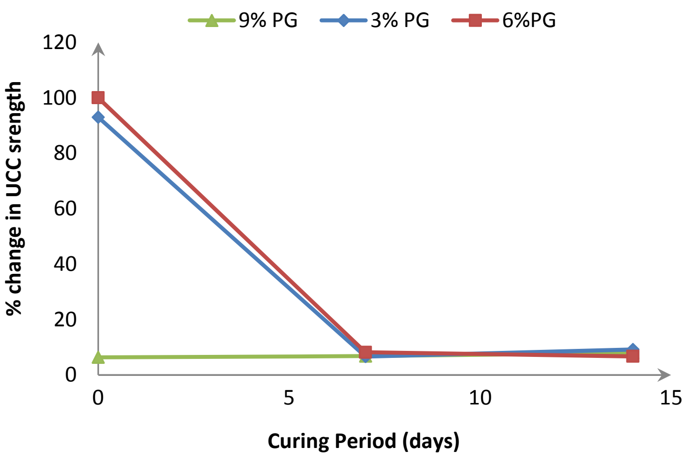

Fig. 5 Variation in the percentage change in UCC strength with the increase in the curing period for the phosphogypsum amended soil

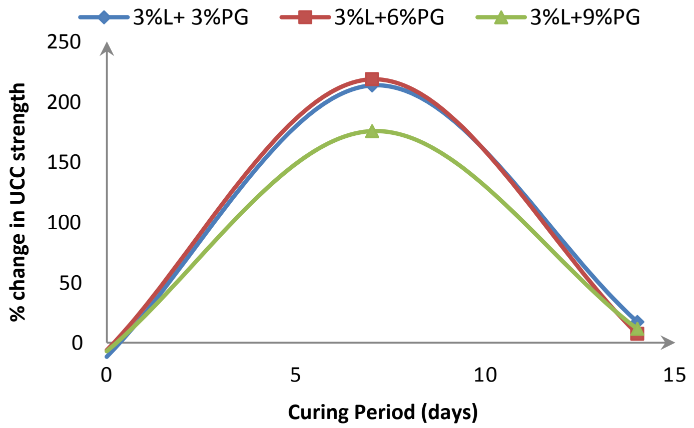

Fig. 6 Variation in the percentage change in UCC strength with the increase in the curing period for the phosphogypsum admixed lime stabilized soil

curing for 6% of phosphogypsum content which is optimum in imparting maximum strength. The similar trend of percentage change in strength is observed for 3% of phosphogypsum content. However, the percentage increase in strength remains relatively constant that is almost remain 7% for 9% of phosphogypsum content with the increase in curing period.

From Fig. 6, it can be seen that the percentage change in strength increases from 0 to 7 days of curing by 225% for both 3 and 6% of phosphogypsum admixed lime stabilized soil, whereas the percentage increase in UCC strength decreases to 7 and 17% from 7 to 14 days of curing period for 3 and 6% phosphogypsum admixed lime stabilized soil. The similar trend of percentage change in strength is observed for 9% of phosphogypsum admixed lime stabilized soil. The percentage increase strength is more for initial curing period, and hence from this, we can conclude that phosphogypsum is responsible for the early strength gain of the soil.

3.1 Mechanism of Increase in the Strength of Phosphogypsum Admixed Soil and Lime Stabilized Soil with Phosphogypsum

The earlier strength development in phosphogypsum stabilized soil can be attributed to the increase in the supply of calcium ions by phosphogypsum, and hence, the accelerated pozzolanic reactions occur [7]. Formation of ettringite may also be the one of the reasons for the early strength gain by the phosphogypsum amended soil. High pH environment is found to be favourable for the formation of ettringite and in this case that is enabled by the presence of lime. Phosphogypsum is consumed by forming ettringite with the increase in the curing period. The formations of ettringite fill up the pores, which is in turn responsible for the increase in the strength of the soil. However if once, the phosphogypsum content goes beyond the optimum content may result in large cluster formation of ettringite and that may lead to the decrease in the strength of the soil.

4 Conclusions

The study here centred towards understanding and comparing the effect of phosphogypsum on black cotton soil with the effect of phosphogypsum on the lime stabilized black cotton soil. From the test results, the following points can be concluded.

- The addition of phosphogypsum to the lime stabilized soil resulted in high early strength gain when compared to the early strength gain of phosphogypsum admixed soil.
- The strength achieved by the soil at optimum content of phosphogypsum and lime amended soil is more when compared to the strength achieved by optimum content of phosphogypsum amended soil.
- The strength of both lime stabilized soil with phosphogypsum and the phosphogypsum stabilized soil increases with the increase in the curing period. However, the percentage increase in strength is more in the former case than the latter one, respectively.
- Phosphogypsum is one of the industrial waste materials that can be utilized effectively in construction of pavement rather than dumping.

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Strength Improvement of Sand by State-of-the-Art Microbially Induced Carbonate Precipitation (MICP) Technique

Vishal Khanna, Umesh Chandra Sahoo, and B. Hanumantha Rao

Abstract Micro-biological geotechnics is a relatively young and dynamic field where microbiological methods are employed to address geotechnical issues. Microbially induced carbonate precipitation (MICP) is one such sustainable method, which enables cementation in loose sandy mass through calcium carbonate precipitation. Among a series of possible mechanisms (i.e. photosynthesis, sulphate reduction, de-nitrification, iron reduction and urea hydrolysis) to attain MICP, urea hydrolysis associates with greater efficacy and ease of practice. In the present study, widely accepted urease positive microorganism was employed as a source of urease enzyme which helps in biocementation process. Additionally, the effectiveness of MICP technique on sand stabilization and the role of particle sizes on the development of cementation bonds were investigated. Unconfined compressive strength (UCS) and hydraulic conductivity (k) tests were performed on samples treated with 1 M urea-calcium chloride cementation solution. To further endorse cementation of sand particles, microstructure analysis such as scanning electron microscopy (SEM) was performed. The detailed analysis showed that MICP has the potential to bind the particles through bio-mineralization which was further warranted by microstructure analysis. SEM images clearly disclosed mesoscopic and microscopic semblance of calcium carbonate precipitation on sand particles, resulting in the stabilization process.

Keywords MICP · Microbial treatment · Bio-cementation · Strength improvement

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1 Introduction

The entire load of a structure is ultimately taken by subgrade, whose properties may vary considerably depending upon its origin. In some cases, the properties might not change spatially, while in others, they may significantly vary from one point to another within a short distance [1–3]. It is obvious that where ever the local soil lacks desired properties, which is deemed necessary for engineering application, the best remedy is to stabilize or alter the soil with a suitable foreign high strength materials, which are compatible [4–6]. However, such modifications are associated with cost and time [7].

Some of the ground improvement techniques that commonly employed are soil replacement, mechanical stabilization by compaction, drainage by soil consolidation, chemical treatment (using cement, lime, calcium chloride etc.), vacuum-assisted pre-consolidation, thermal treatment, stone/sand columns, dynamic compaction by heavy tamping, vibro-flotation and deep mixing. Among many afore-mentioned methods, cement and lime due to their ease in availability, application and cost effectiveness are ubiquitously used worldwide. However, their harmful impact on environment cannot be turned down. Researchers have documented that one metric ton production of ordinary Portland cement liberates around one metric ton of carbon dioxide and similarly one metric ton of lime releases around 0.86 metric ton of carbon dioxide [8, 9]. The obvious consequences of CO_2 release into the atmosphere are well known in the form problems like global warming, while other negative aspects include ground water contamination, particulate particle emissions, etc. [10].

On the flip side, high consumption of natural resource material causing excessive exploitation of nature, leading to depletion of natural resources, environmental degradation and enhanced cost of cementing materials [11]. The non-availability of quality natural materials and incurrence of exorbitant prices on procurement of cementing materials have led the research fraternity to devise novel techniques for the soil stabilization, which are both economical as well as environmentally benign [12].

There has been a significant increase in the research to explore the possible biological technologies that can be applied in the construction industry and can work as a carbon sink. MICP is one such technology, which when applied in civil engineering projects enables the confluence of microbiology, chemistry and civil engineering disciplines. Out of the various possible mechanisms (i.e. photosynthesis, sulphate reduction, nitrogen reduction, iron reduction and urea hydrolysis) to attain MICP, urea hydrolysis associates highest efficacy and ease [13]. MICP has shown promising results in the stabilization of sand, wastewater treatment, strengthening of concrete and enhanced oil recovery [14–16]. MICP is sometimes also referred as "Bio-Grouting" where bio-based suspension is injected in a granular material in order to catalyze biochemical reactions and further facilitate calcium carbonate precipitation [17, 18]. In the present study, authors aim to cater some of the geotechnical issues using MICP technique. Effect of void space and efficiency of MICP along the sample depth is also evaluated.

2 Mechanism of Calcite Precipitation Through Urea Hydrolysis

Formation of calcite by urease positive bacteria is highly substrate dependent. These bacteria release urease enzyme which hydrolyze urea (the substrate) by utilizing two moles of water (H_2O) and leads to formation of carbonate ($CO_3{}^{2-}$) and ammonium ions (NH^{4+}) as per Eqs. 1, 2:

$$CO(NH_2)_2 + 2H_2O \rightarrow 2NH_4 + CO_3^{2-} \tag{1}$$

$$HCO_3^- \rightarrow CO_3^{--} + H^+$$

$$Ca^{++} + CO_3^{--} \rightarrow CaCO_3 \downarrow \tag{2}$$

The above process tends to increase pH of the solution and as soon as the pH value increases 8.5, precipitation of calcite begins in the presence of a calcium source. This precipitation can be delayed by the use of different buffer at different concentration [19]. The progress of the above process is governed based on various chemical, environmental and geotechnical parameters. The scope of the present study is limited to geotechnical parameters.

3 Materials

3.1 MICP Recipe

A gram-positive bacteria strain was used in this study and BHI solution mixed with 20 gm/l of sterile filtered urea solution was used as an inoculation medium. The prepared mixture was kept in an incubator cum orbital shaker (at 37 °C) for 24 h at 160–180 rpm. Bacterial cell harvesting was done through centrifugation at 4600 × g for 10 min (Hermile Centrifuge Z 326 K) and an OD600 of 0.8–1.2 was adjusted. The selection of time was made after different trials, ensuring stable cell pellet. After harvesting, the supernatant was removed and cell pellets were resuspended in buffer.

3.2 Sand

For the present study, a sand sample was collected from the banks of river Mahanadi in the state of Odisha, India. This sample is designated as Mahanadi River Sand (MRSM). The same sand was further sieved and subcategorized as coarse MRS

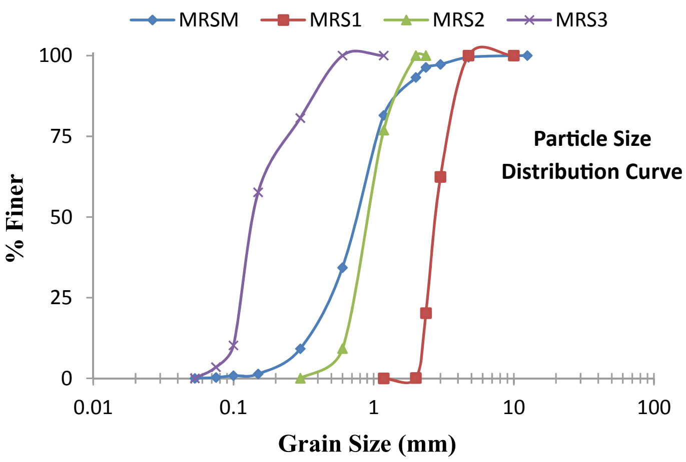

Fig. 1 Gradational characteristics of different sand samples used in the study

Table 1 Particle size characteristics of the sans samples used in the study

Type of soil	D_{10}	C_u	C_c
MRSM	0.314	3.046999	1.147849
MRS1	2.18	1.361814	0.975304
MRS2	0.62434	1.742448	0.899866
MRS3	0.099	1.704814	1.11592

(MRS1), medium MRS (MRS2) and fine MRS (MRS3). The gradational characteristics of the sand samples are shown in Fig. 1. The sand used in the study does not exhibit requisite engineering properties for most of the pavement and well as geotechnical applications. A summary of particle size characteristics is provided in the Table 1.

4 Experimental Methodology

4.1 Sample Preparation

Several identical test specimens of approximately 38 mm diameter and 76 mm height were prepared by pouring the sand adopting to rainfall technique. Sand was filled in five separate layers to achieve desired relative density of 82%. Grouting was done in two different phases. First phase involved injection of 0.5 void volume (vv) bacterial suspension (a mixture of 50 mM Tris buffer, Bacterial suspension, 3 g/L BHI broth), while the second phase, which begun after 6 h, involved injection of cementation solution (a mixture of 1 M Urea-Calcium Chloride) through the

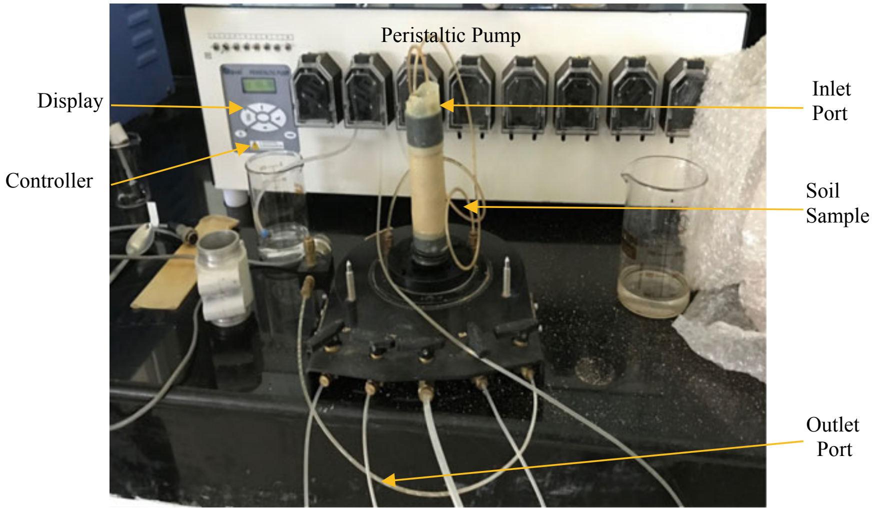

Fig. 2 Typical test set-up used for bacterial and cementation injection

top port of triaxial chamber with the help of peristaltic pump (10 mL/min) (see Fig. 2). The rate was decided after calibrating flow with soil permeability and to ensure that no impounding occurs. Prior to the above phases, 1 vv autoclaved/de-ionized water was down flushed to remove excess ions, entrapped air and to saturate sample. Moreover, to increase bacterial retention 0.3 M calcium chloride solution was injected immediately after first phase. The intermediate time provided between the two phase was to give microorganism chance to adhere to the soil grains and to avoid instantaneous flush out of bacteria solution. About 5–8 times cementation solutions were injected to ensure dense precipitation at about 12 h interval.

4.2 $CaCO_3$ Content (CCC)

Acid wash method was adopted to determine calcium carbonate content. The calcium carbonate percentage is expressed as mass of $CaCO_3$ divided by mass of soil as given in Eq. 3 [20].

$$\mathrm{CaCO_3\ content}(\%) = \left(\frac{W_{\mathrm{treated}}(W_1) - W_{\mathrm{acid\ washed}}(W_2)}{W_{\mathrm{acid\ washed}}(W_2)} * 100\right) \tag{3}$$

where W_1 is the mass of treated soil and W_2 is the mass of the soil recorded after acid wash.

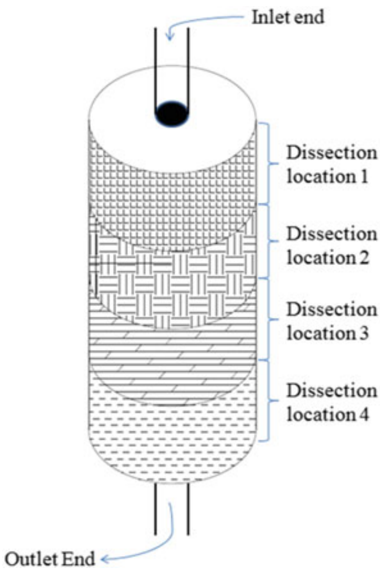

Fig. 3 UCS sample representing different dissection locations

In the present study, change in bio-mineralization (i.e. CCC) as a function of depth of sample was evaluated. Samples for the same were carefully prepared by dissecting the specimen into four layers as shown in Fig. 3 (DL1-DL4).

4.3 Permeability

Permeability is one of the prime factors governing the behaviour of a material. A highly porous material with high permeability restricts the occurrence of excess pore water pressure and vice versa. In the present study, permeability test on the untreated and bio-mineralized sample was conducted in a laboratory using constant head method in accordance with the Indian standard code of practice [21]. All the treated and untreated samples were completely saturated prior to permeability test with 1L water to remove the entrapped air.

4.4 Morphological Characteristics

Micro-level analysis was performed to evaluate the morphological, mineralogical, characteristics of precipitation and degree of bonding between the soil grains and were ascertained using MERLIN compact field emission scanning electron microscope (FE-SEM) with EDS unit attached. Scanning electron microscopy (SEM) and energy dispersive X-ray spectroscopy (EDS) images were captured of both treated

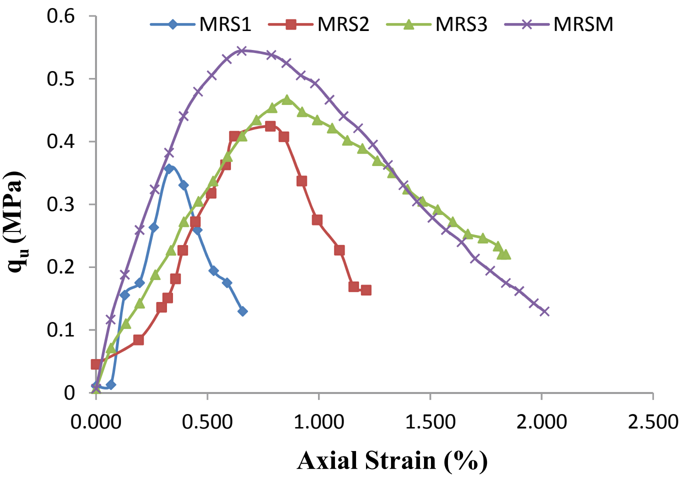

Fig. 4 Stress–strain response obtained from different sand samples

and untreated samples. The change in precipitation at different depths was also analyzed. For the preparation of powder sample, 2–3 g of over dry sample passing sieve 75 microns was used. Prior to sample evaluation in the FESEM, samples were coated (sputtered) with gold using Q150R ES Sputter Coater (make, Quorum, UK).

5 Results and Discussion

5.1 Unconfined Compressive Strength

After 5 to 8 injections, the bacterial treated samples were left for 7 days curing. UCS tests conducted on different MRS treated samples revealed that a maximum strength of about 544 kPa can be attained with 1 M urea-calcium chloride concentration on well graded sand samples (see Fig. 4). Results obtained from the study are similar to the findings of [22–25].

5.2 Calcium Carbonate Content (CCC)

In the present study, four different graded sands, fine, medium, coarse and a mixture of the trio, were used. All soils are quite distinct in their permeation characteristics, which is one of the key for ensuing the uniform calcite precipitations across as well as along its length. To affirm the fact that the precipitated compound is calcium carbonate and that it is uniform from top to the bottom of sample, acid wash technique to determine CCC was performed. An average CCC precipitated at different dissection locations was also estimated to correlate it to the degree of precipitation and contribution to strength development. Figure 5 shows the variation of precipitation intensity estimated at different dissection locations and the same values are also listed in Table 3. A simple observation of results reveals relatively low CCC precipitations (2.87%) in the case of MRS1 vis-à-vis with that of MRSW soil (6.26%). One of the reasons for low precipitation in MRS1 may be low bacterial cell density or retention in the sample as the major bacterial suspension was found to get discharged with the cementing injections. It can also be discerned from Fig. 5 that the pattern of CCC precipitations in all the four grades of sand are almost similar at different dissection locations, though the values differ significantly. It was observed that CCC decreased with an increase in depth, except for the last layer (Table 2). The increase in CCC at DL4 may be due to the placing of filter paper at the outlet end, which might have facilitated the retention of more bacterial cells resulting in higher CCC precipitation

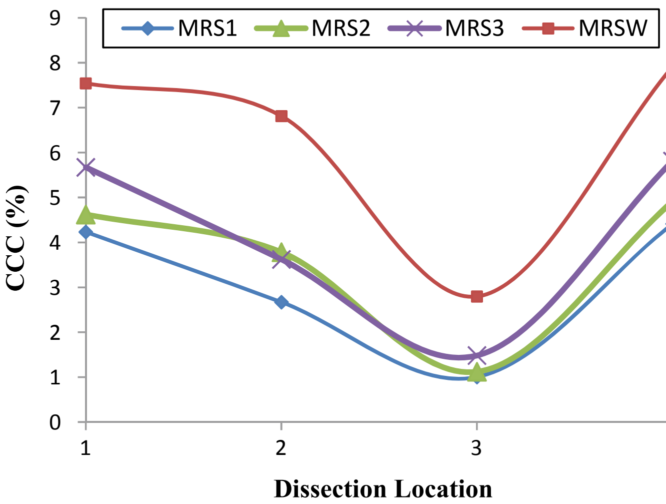

Fig. 5 CCC precipitation intensity at different dissection locations of MRS sands

Table 2 Value of CCC measured before and after treatment of MRS at different dissection locations

Sample dissection location	MRS1	MRS2	MRS3	MRSW
Untreated	0.0273	0.0586	0.0392	0.0922
DL1	4.232	4.622	5.672	7.5396
DL2	2.671	3.781	3.623	6.8119
DL3	1.004	1.122	1.482	2.7972
DL4	4.412	4.912	5.8122	7.9114

Table 3 Value of permeability measured on bacterial treated LSO and MRS soil with different concentrations of CCL

Sand sample	k (cm/s)	
	Untreated	Treated
MRS1	6.067×10^{-3}	2.932×10^{-3}
MRS2	5.692×10^{-3}	2.536×10^{-3}
MRS3	4.823×10^{-3}	1.426×10^{-3}
MRSM	5.6×10^{-3}	0.816×10^{-3}

(refer to Fig. 4). As such, results in Fig. 5 clearly demonstrate a fact that permeability, void space and bacterial retention play an important role in accentuating the cementation effect within the soil sample.

5.3 Permeability

It is interesting to note from the above results that there is a contrasting effect between permeability and precipitation. Understandably, these two properties are highly interdependent. While the former property is crucial in ensuing the latter one up to a certain limit (till it does not lead to the free flow of bacterial suspension), which at the later stage of testing plays a decisive role of making the medium an impervious. This highlights a fact that resorting to MICP technique fetches multiple advantages such as strength enhancement concurrently diminishing permeability characteristics of the media. With this in mind, permeability tests were also conducted on bio-cemented MRS soils. The process of MICP can offer significant increase in strength, maintaining the permeability of the material to such an extent that the negligible pore water pressure is generated. The values of permeability measured on various bio-cemented samples are presented in Table 3. The results indicate a significant decrease in permeability with bio-treatment. It was found that permeability reduced by 48–85% based on the pore space or void space. Similar findings have been reported by Van Paassen et al. [26] and Harkes et al. [27], who reported a reduction in permeability within the same range.

6 Morphological Analysis

SEM analysis was performed to examine the microscopic semblance and portray the presence of calcium carbonate precipitation within the materials. From Fig. 6, a distinct layer of precipitation can vividly be seen, as untreated particles (Fig. 6a) have smooth surface and that of treated ones coated with minerals.

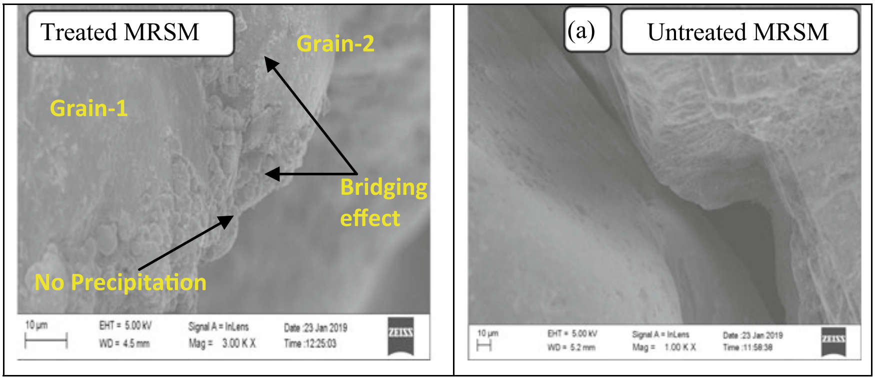

Fig. 6 SEM images of untreated and bacterial treated MRSM soils showing the bridging and cementation effect

Based on the images presented in Fig. 7, it can be substantiated that the improvement in compressive strength in MRS soil occurred mainly because of bridging action together with the precipitation and cementation between the particles, in particular at grain contacts.

Therefore, in order the soil to exhibit greater binding action or compressive strength, precipitations should happen maximum at the contact junction of particles or on the grain surface, apart from the pore space, which might associate with an increase in pore water pressure leading to decrease in effective stress. Such precipitation, importantly also, widens the applicability of MICP technique to develop the impervious barriers.

SEM images in Fig. 7 exhibits a progressive calcite precipitation in the MRS soil based upon the number of injections. It can be inferred from the images that precipitations might be happening concurrently on the grain surface and particle contact junctions.

7 Conclusions

Based on the extensive experimental investigations carried out in the present work, the following salient conclusions can be drawn:

- The various results demonstrate that MICP is an effective means of stabilizing poor soils by calcite mineralization.
- It has been observed that the UCS value of MRSM sample increased from 0 to 0.544 MPa when treated by resorting to MICP technique.
- The morphological analysis clearly revealed that bacterium is efficient in cementation by bridging mechanism. It also portrayed that not only voids are the place of precipitation, rather binding also occurred on the grain surface.

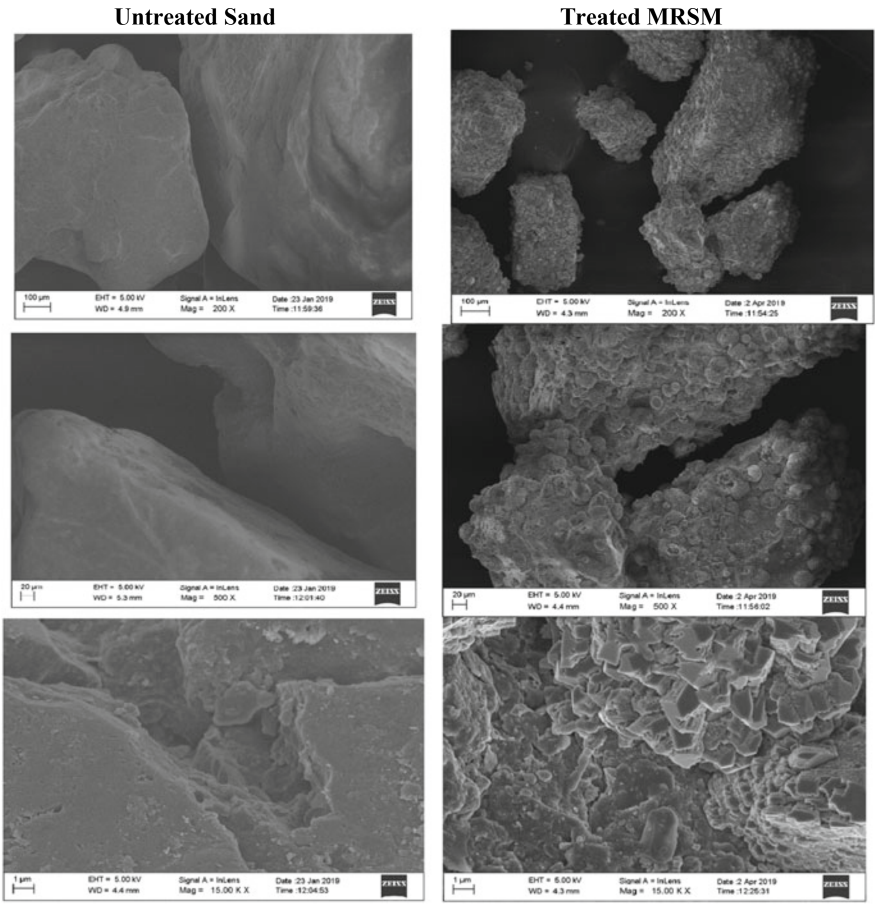

Fig. 7 SEM images revealing progressive calcite mineralization on MRS soil

- The results also highlight that particles size and pore spaces are major factors governing the efficiency and applicability of the process.
- Results pertinent to dissection analysis disclose the variable intensity of carbonate precipitations along the depth of the sample.

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An Experimental Study on Effects of Non-plastic Fines on Engineering Properties of Sand–Silt Mixture

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Abstract Sandy soil as a foundation material largely depends on the engineering properties of a soil governed by its physical properties and behaviour, containing less or more amount of fines obtained through dredging operation. As IS: 6403-1981 and NAVFAC depicts approximate correlations between SPT *N*-value, angle of internal friction, relative density and dry unit weight for cohesionless material. But, the effect of a different range of percentage of fines on different sand gradations is not well covered. Hence, an attempt has been made to establish a relationship for different gradations of sand with different amount of fines. An experimental study on the sand along with silt under different conditions was conducted. The reconstituted well-graded samples containing different amount of silt, i.e. 0%, 5%, 10% and 15% were subjected to a vibration table for relative density to obtain maximum and minimum dry density. For the shear strength and consolidation parameter, samples were subjected to direct shear test and consolidation test at three relative density, i.e. 30, 60 and 90% at the displacement rate of 0.25 mm/min under the dry and saturated state. Laboratory results depict that up till transition fines content the angle of internal friction increases and a further increase in fines it decreases. Also, the compressibility increases for different relative density. From the results, it is observed that the percentage of fines alters the engineering behaviour of sand.

Keywords Sand-Silt · Relative density · Direct shear test · Dry and saturated condition · Consolidation test

1 Introduction

Traditionally, sandy soil as foundation material has been widely in practice since long time. The stability and safety of structure resting on sandy soil depend on shear strength and deformation of soil structure under the stresses. Moreover, large

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numbers of factors affect the shear strength of soil such as particle shape, particle size distribution, presence of fines (non-plastic/plastic), denseness of soil, drainage conditions, etc. But during the reclamation process of sand from the sources, contain small or large amount of fines. So, it becomes necessary to investigate on effect of non-plastic fines on these parameters of sandy soil.

A large number of existing studies [1–12] in the broader literature have examined the different aspects of clean sand behaviour through extensive and comprehensive studies and experiments. Additionally, few studies [1–3] have focused on up till what and how much degree the fines content affects the various parameter of sand. In granular soil, the arrangement of particles is referred as packing of particles. The packing of particles is strongly influenced by particle shape and its distribution. Moreover, on rearrangement or variation in packing of particles affect the engineering properties of soil. So, to improve the engineering properties of soil, changes in the packing of particles can be done. This can be achieved by the introducing the fines (i.e. below 75 micron and non-plastic) to fill up the voids. For optimum grain size distribution of voids in between the bigger size particles needs to be filled up with smaller size particle. Increase in shear strength mostly depends on particle contact and it is related to the porosity and void ratio.

An experimental study has been conducted to understand the phenomenon of packing of particles and how the fine content affects the minimum void ratio and maximum void ratio Lade and Yamamuro [1]. By performing few experiments, study has shown the theoretical relationship between the void ratio and percentage fines by the schematic diagram (see Fig. 1).

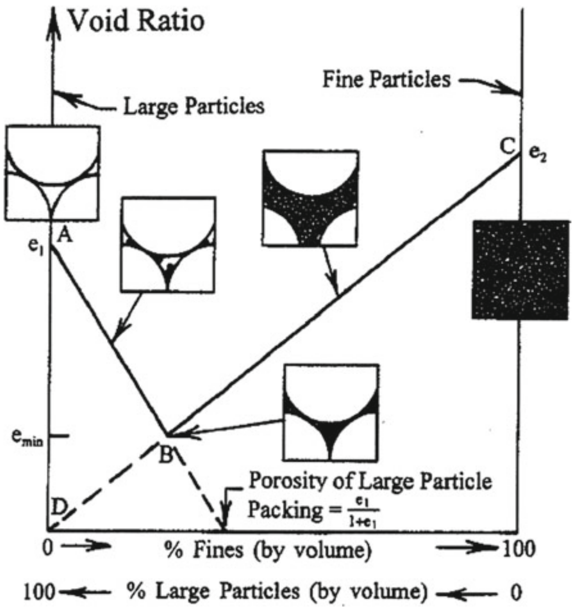

Fig. 1 Theoretical relationship between void ratio percentage fine content

Prior research [2–4] recommends, the strength of soil is expected to be constant until voids are completely filled with fines. When fines get in between the granular particles, strength rapidly reduces. At this stage amount of fines is known as "Transition Fine" content and it differentiate the dominating behaviour of mix.

Addition of non-plastic fines at low content is not significant to participate in force transfer mechanism in sand–silt mixture, due to their nature, position and size. Thus space occupied by fines should be considered as void space, and the stated problem can be simplified by introducing the term "Inter-granular void ratio (e_g)". The simple formulation suggested by Thevanayagam [3] for e_g of the mixture with low fine content can be determined by.

$$e_g = \frac{e + f_c}{1 + f_c} \tag{1}$$

where f_c is fines content and e is the global void ratio.

However, inter-granular void ratio is not applicable to mixture with high fine content, Thevanayagam and Yang et al. [3, 5]. Additionally, the concept of "Equivalent-granular void ratio (e_{ge})" [6, 7] has been considered used to account the contribution of higher fine content and this approach requires an parameter "b" that resembles the fraction of fines participating in force structure of soil skeleton, e is global void ratio and f_c is fine grain content.

$$e_{ge} = \frac{e + (1 - b)f_c}{1 - (1 - b)f_c} \tag{2}$$

2 Methodology

For this study, sample was prepared by locally available Class-I and normal silica sand, i.e. coarse sand (C)—4.75–2 mm, medium sand (M)—2–0.425 mm and fine sand (F)—0.425–0.075 mm. To prepare the silt in laboratory, natural sample procured from the Gulf of Khambhat, Gujarat, was washed on 0.075 mm IS sieve.

3 Preliminary Investigation

On blending coarse, medium and fine sand, three well-graded sand samples were prepared mechanically, i.e. C+M, M+F and C+M+F with the characteristics shown in Table 1 and Fig. 2. The silt was prepared in the laboratory by hydrometer analysis as per ASTM D 422-63 [8] on the Khambhat sample after treating it with dispersing agent, i.e. sodium hexametaphosphate, sodium carbonate for the deflocculation of

Table 1 Sieve analysis data of remoulded sample

Sr. No.	Sample	D_{60} (mm)	D_{30} (mm)	D_{10} (mm)	C_u	C_c
1.	Coarse + medium sand	3.10	1.30	0.51	6.08	1.07
2.	Medium + fine sand	0.60	0.28	0.10	6.00	1.31
3.	Coarse + medium + fine sand	1.19	0.40	0.12	9.90	1.10

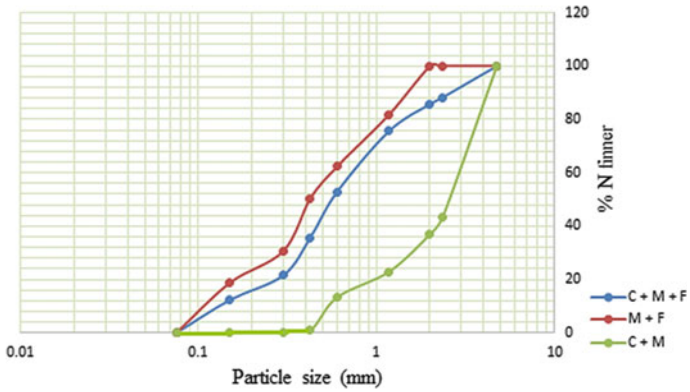

Fig. 2 Grain size distribution curve

Table 2 Index properties of silt sample

Sample	Specific gravity (G)	Liquid limit	Plastic limit	Plasticity index
Silt	2.68	21.6	–	NP

fine particles. Atterberg limit tests on sample showed low liquid limit and plastic limit, thus as per Indian Standard Soil Classification System (ISSCS), sample was classified as non-plastic fines (silt) Table 2.

3.1 Experimental Program

In order to explain the effect of fines on the behaviour of sand–silt mixture, a series of specific gravity, relative density, direct shear and consolidation test was conducted on sand–silt sample. To determine the maximum and minimum densities of each reconstituted samples having 0%, 5%, 10% and 15% of silt, respectively, were subjected to relative density as per the IS:2720-Part XIV [9, 10]. Strain-controlled direct shear tests were conducted on sand–silt specimen at a strain rate of 0.25 mm/min under dry and saturated state at relative density of 30%, 60% and 90%, respectively. The same specimens were also subjected to oedometer test as per IS: 2720-Part XV.

4 Results and Discussion

On the basis of above experimental study, maximum and minimum void ratios were computed analytically as shown in Table 3. These analytical values simulate the theoretical concept (see Fig. 1) given by the previous study [11–15]. Through the obtained data, it has been observed that void ratio decreases as the fine content increases, shown by the graphical representation (see Fig. 3, 4 and 5).

Further their influence on shear strength under dry and saturated conditions at different relative densities has been experimentally studied by Table 4 and 5. Result shows that angle of internal friction is higher in dry state as compared to saturated state. In case of C+M+F, the shear strength increases up to 5% fines both in dry and saturated states due to complete filling up of voids. On further addition of fines, they get in between the coarse particles and shows decreases in shear strength.

Compressibility of the reconstituted samples prepared on the basis of dry density obtained from relative density seems to be increase on addition of fines, Table 6.

5 Concluding Remarks

This study clarifies the effect of non-plastic fines on the engineering properties of sand–silt mixture through analytical and experimental program. Results acquired from the tests have been discussed above and the following conclusion is drawn:

- On increase in fine content, increase in dry density is observed.
- Void ratio decreases as the fine content increases up till the transition fine content.
- In case of C+M+F sand, the angle of internal friction increases up to 5% under both dry and saturated conditions as all the voids are nearly filled up and further on addition, it decreases for 10% and 15% non-plastic fines, as its gets in between the coarse particles.
- In case of C+M and M+F specimen, the angle of internal friction increases up to 15% fines in dry state but in saturated state, it increases up to addition of 10% fines and on further addition, it decreases.
- Coefficient of volume compressibility obtained from oedometer goes on increasing for each specimen on addition of fines from 0% to 15%.
- For the M+F specimen, coefficient of compressibility decreases on addition of 10% and 15% of fines.
- This shows that on addition of non-plastic fines the compressibility of soil sample increases for 30%, 60% and 90% relative density.

Table 3 Laboratory tests result

Sample	Silt (%)	Specific gravity	Minimum density (g/cc)	Maximum density (g/cc)	Dry density (g/cc)	Minimum void ratio	Maximum void ratio	Void ratio (e)	Relative density (%)
C + M	0	2.740	1.74	2.02	1.82	0.356	0.575	0.505	30
					1.9			0.442	60
					1.99			0.377	90
	5	2.740	1.77	2.08	1.85	0.317	0.548	0.481	30
					1.94			0.412	60
					2.04			0.343	90
	10	2.721	1.84	2.19	1.93	0.242	0.479	0.410	30
					2.04			0.334	60
					2.15			0.266	90
	15	2.721	1.88	2.22	1.97	0.226	0.447	0.381	30
					2.07			0.314	60
					2.18			0.248	90
M + F	0	2.685	1.69	1.99	1.77	0.349	0.589	0.517	30
					1.86			0.444	60
					1.96			0.370	90
	5	2.667	1.72	2.03	1.8	0.314	0.551	0.482	30
					1.89			0.411	60
					1.99			0.340	90
	10	2.667	1.73	2.07	1.82	0.288	0.542	0.465	30
					1.92			0.389	60
					2.03			0.314	90
	15	2.667	1.72	2.12	1.82	0.258	0.551	0.465	30
					1.94			0.375	60

(continued)

Table 3 (continued)

Sample	Silt (%)	Specific gravity	Minimum density (g/cc)	Maximum density (g/cc)	Dry density (g/cc)	Minimum void ratio	Maximum void ratio	Void ratio (e)	Relative density (%)
					2.07			0.288	90
C + M + F	0	2.703	1.71	1.98	1.78	0.365	0.581	0.519	30
					1.86			0.453	60
					1.95			0.386	90
	5	2.685	1.76	2.04	1.84	0.316	0.526	0.459	30
					1.92			0.398	60
					2.01			0.336	90
	10	2.685	1.75	2.1	1.84	0.279	0.534	0.459	30
					1.94			0.384	60
					2.06			0.303	90
	15	2.685	1.79	2.11	1.88	0.273	0.500	0.428	30
					1.97			0.363	60
					2.07			0.297	90

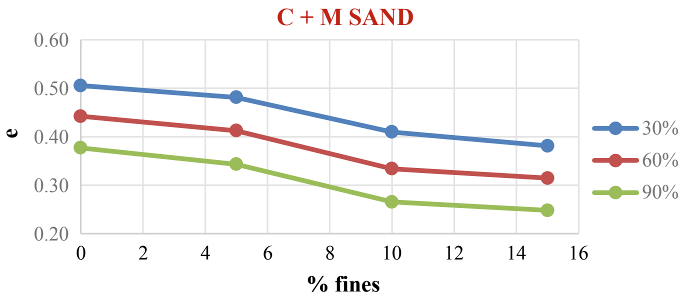

Fig. 3 Relationship between void ratio and percentage fine for C+M sample

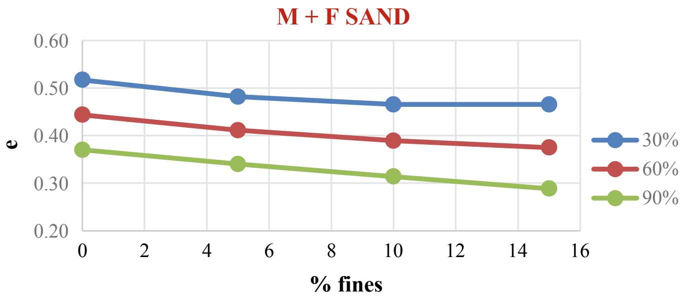

Fig. 4 Relationship between void ratio and percentage fine for M+F sample

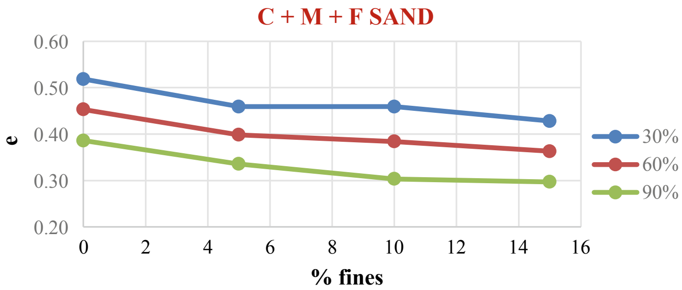

Fig. 5 Relationship between void ratio and percentage fine for C+M+F sample

Table 4 Angle of internal friction data of reconstituted sample under dry state

Soil samples	Angle of internal friction (ϕ)											
% Silt	0%			5%			10%			15%		
Relative density	30	60	90	30	60	90	30	60	90	30	60	90
C+M	43.55	45.42	47.24	45.4	46.17	46.41	46.62	47.59	48.13	47.67	48.8	48.9
M+F	33.38	37.78	42.13	35.82	38.38	43.04	37.45	41.41	46.63	38.49	45.46	47.32
C+M+F	37.63	39.72	43.92	40.95	42.09	45.60	39.80	41.43	45.14	37.73	39.69	42.04

Table 5 Angle of internal friction data of reconstituted sample under saturated state

Soil samples	Angle of internal friction (ϕ)											
% Silt	0%			5%			10%			15%		
Relative density	30	60	90	30	60	90	30	60	90	30	60	90
C+M	42.35	43.42	44.11	41.84	44.12	45.40	42.33	44.83	46.31	36.96	39.02	43.54
M+F	32.37	35.82	38.18	34.90	38.00	39.97	35.21	40.31	41.18	33.71	34.15	35.90
C+M+F	34.00	37.15	40.65	38.07	39.70	41.98	37.64	38.39	41.26	34.14	36.16	37.33

Table 6 Coefficient of volume compressibility data on increases in effective stresses

Soil samples	m_v (cm^2/kg)											
% Silt	0%			5%			10%			15%		
Relative density	30	60	90	30	60	90	30	60	90	30	60	90
C+M	0.0048	0.0047	0.0041	0.0056	0.0048	0.0041	0.0059	0.0051	0.0043	0.0075	0.0067	0.0045
M+F	0.0048	0.0041	0.0034	0.0049	0.0042	0.0034	0.0050	0.0035	0.0043	0.0043	0.0035	0.0028
C+M+F	0.0041	0.0040	0.0040	0.0050	0.0042	0.0042	0.0056	0.0049	0.0042	0.0059	0.0059	0.0043

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Effect of Vertical Reinforcement on Settlement and Displacement in Reinforced Soil Under a Three-Dimensional Framed Structure

P. Manjunath, H. M. Rajashekhar Swamy, and Nayana Patil

Abstract Soil reinforcement is a technique for improving the mechanical properties of soil. In recent years, the use of reinforced soils has increased widely due to its satisfactory performance and cost effectiveness. Many studies have been carried out on reinforced soil with conventional horizontal reinforcement. The main disadvantage of horizontal alignment of reinforcement is that it requires large-scale excavation of soil, which destroys the strength of soil developed over the years and is also expensive. In this research, studies have been carried out on soil reinforced with vertical reinforcement by considering soil–structure interaction. For this purpose, a four-storey three-dimensional frame structure with isolated footing resting on both un-reinforced and reinforced soil has been considered. Soil has been reinforced with HYSD bars of Fe 500 grade and reinforcement is provided only below footings. The frame section and soil continuum have been modelled and analysed using finite element-based software program SAP2000. The size of the soil mass considered is 153 m $\times$ 95 m $\times$ 20 m. Parametric studies have been carried out by varying reinforcement length and reinforcement spacing. The study revealed that the displacements in soil can be reduced by the inclusion of vertical reinforcement. Settlement is reduced in the range of 4.45–16.79%. Horizontal displacement along longitudinal and transverse direction is reduced in the range of 7.37% to 26.31% and 8% to 33.24%, respectively. Differential settlement in reinforced soil is reduced by 30.34% when compared with that of un-reinforced soil.

Keywords Reinforced soil · Settlement · Soil–structure interaction · SAP 2000 · Vertical reinforcement

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1 Introduction

Soil–structure interaction is an interdisciplinary field which involves the study of structural engineering, foundation engineering and geotechnical engineering [1]. Soil–structure interaction (SSI) is a process in which the response of soil influences the motion of the structure and the motion of the structure influences the response of the soil. In this case, neither the structural displacements nor the ground displacements are independent from each other. When SSI is taken into consideration, the soil properties, travel path and the geometry of the soil medium influence the ground motions imposed on the foundation of the structure [2].

Reinforced soils have been widely used in geotechnical structures as a result of their satisfactory performance and cost effectiveness [3]. Reinforced soil is most commonly used in embankments and retaining walls. The main disadvantage of horizontal alignment of reinforcement is that it requires large-scale excavation of soil, which destroys the strength of soil developed over the years. Further the soil has to be compacted after placing reinforcement [4].

Numerous studies have been carried out on the effect of SSI under static and dynamic loading and also on reinforced soil foundations and retaining walls. The study on behaviour of interaction of plane frame on elastic foundation with shear and normal moduli of subgrade reactions was carried out by Aljanabi et al. [5]. Simplified approach for soil–structure interaction analysis was developed and studied for 2D skeletal RC frame resting on isolated footing with different soil types by Al-Shamrani and Al-Mashary [6]. Improvement of bearing capacity of loose sand using flexible reinforcement was carried out by Puri et al. [7]. The study on soil reinforced with multilayer horizontal and vertical reinforcement was carried out by Zhang et al. [8]. The effect of SSI on 3D space frame with pile foundation and embedded in clayey soil was studied by Chore et al. [9]. The interactive and non-interactive analysis of a space frame-raft foundation-soil system was carried out by Thangaraj and Ilamparuthi [10]. The interactive behaviour of the 3D frame with isolated footing which is resting on un-reinforced soil was studied by Rajashekhar Swamy et al. [11]. The relevance of interface elements in SSI of 3D frame with raft foundation resting on unreinforced soil was studied by Rajashekhar Swamy et al. [12]. Studies on reinforced soil–structure interaction analysis of 3D space frame resting on soil reinforced with horizontal geogrids was carried out by Nayana [13].

The above literature review reveals the number of studies carried out on SSI and very few studies carried out on soil reinforced with conventional horizontal reinforcement. However, the studies on structure resting on reinforced soil with vertical reinforcement by considering the soil–structure interaction have not yet been carried out. Therefore, it is necessary to study the effect of vertical reinforcement in reinforced soil by considering SSI.

2 Problem Definition

The present study is carried out on the structure shown in Fig. 1. The structure under consideration is selected from the literature Rajashekhar Swamy et al. (2011). The details of the structure and properties of the materials are given in Table 1. As the soil is semi-infinite, the size of the soil mass considered is 153 m × 95 m × 20 m as shown in Fig. 1.

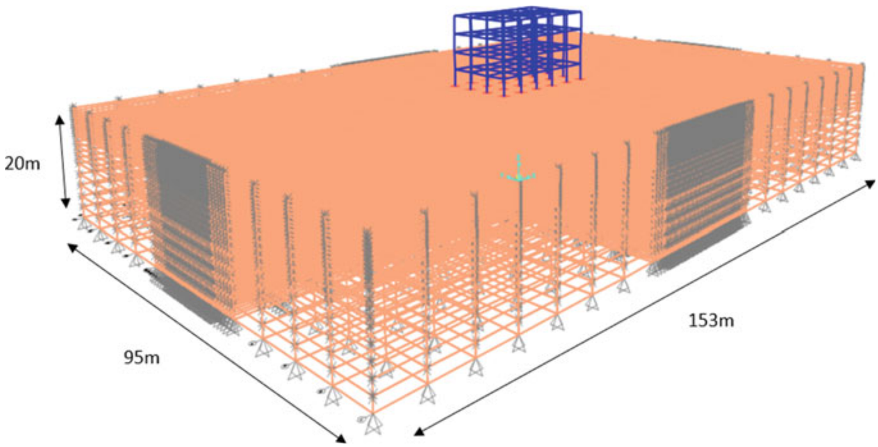

Fig. 1 3D view of space frame resting on unreinforced soil

Table 1 Details of structure and material properties (Rajashekhar Swamy et al.)

S. No.	Structure	Component	Details	Unit
1	Frame	Number of storeys	4	m
		Number of bays	5 × 3	
		Storey height	3.5	m
		Bay width	5	m
		Beam size	300 × 600	mm
		Column size	400 × 400	mm
2	Footing		2.0 × 2.0 × 0.2	m
3	Soil mass		153 × 95 × 20	m
4	Elastic modulus of soil		1.33×10^4	kN/m^2
5	Poisson's ratio of soil		0.45	
6	Elastic modulus of concrete		2.73×10^7	kN/m^2

Table 2 Details of element type

S. No.	Component	Element type	Figure
1	Beams, columns and reinforcement	Two-nodded beam element with six degrees of freedom per node	
2	Isolated footing	Four-nodded plate element with five degrees of freedom per node	
3	Soil mass	Eight-nodded brick element with three degrees of freedom per node	

3 Modelling and Formulation

Finite element method is adopted to study behaviour of the frame-isolated footing-reinforced soil system. Soil has been vertically reinforced with 25 mm diameter HYSD bar of Fe 500 grade and reinforcement is provided only below footings. Modelling and linear analysis of superstructure along with supporting system and soil are done in finite element-based software program SAP2000. Table 2 gives the details of element types used for modelling.

3.1 Super-Structure

Beams and columns of the superstructure frame are modelled using three-dimensional two-nodded beam element with six degrees of freedom per node.

3.2 Sub-structure

Isolated footing is modelled using plate elements with five degrees of freedom per node, i.e. three-translational degrees of freedom and two rotational degrees of freedom. Soil is modelled using eight-nodded brick element with three-translational degrees of freedom per node. Reinforcement in the soil is modelled using three-dimensional two-nodded beam element with six degrees of freedom per node.

4 Parametric Studies on Single Flexible Isolated Footing

The parametric study has been carried out on single flexible isolated footing of size 2.0 m × 2.0 m × 0.2 m for two parameters, (i) Reinforcement length (U) and (ii) Reinforcement spacing (S). The soil boundary is assumed to be six times the width of the footing with the properties mentioned in Table 1.

A non-dimensional parameter ‘U/B’ where *U* is the length of reinforcement and *B* is the width of footing has been considered to determine the optimum length of reinforcement. The analysis is carried out for different U/B ratios of 0.5, 0.75, 1, 1.25, 1.5, 1.75, 2, 2.25 and 2.5 for reinforcement spacing of 250, 500 and 1000 mm. Reinforcement details are given in Table 3. A total 30 combinations of single flexible isolated footing resting on reinforced soil were developed and analysed with area load of 187.5 kN/m^2 applied on the footing top. A graph of differential settlement v/s reinforcement spacing for different U/B ratios are plotted and shown in Figs. 2 and 3.

From the plot between differential settlement and reinforcement spacing, it was observed that the values of differential settlement decreased with increase in reinforcement spacing for all the U/B ratios of 0.50, 0.75 and 1.00.

Further, the study is carried out by performing static analysis on 3D space frame structure resting on reinforced soil for U/B ratios of 0.50, 0.75 and 1.00 for reinforcement spacing of 250, 500 and 1000 mm.

Table 3 Reinforcement details

S. No	Material	Diameter (mm)	Spacing mm()	U/B ratio
1	Fe 500 grade HYSD bar	25	250	0.5, 0.75, 1, 1.25, 1.5, 1.75, 2, 2 25 and 2.5
			500	0.5, 0.75, 1, 1.25, 1.5, 1.75, 2, 2.25 and 2.5
			1000	0.5, 0.75, 1, 1.25, 1.5, 1.75, 2, 2.25 and 2.5

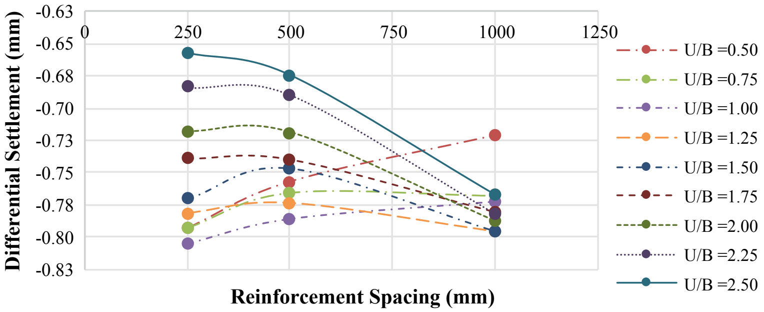

Fig. 2 Differential settlement versus reinforcement spacing

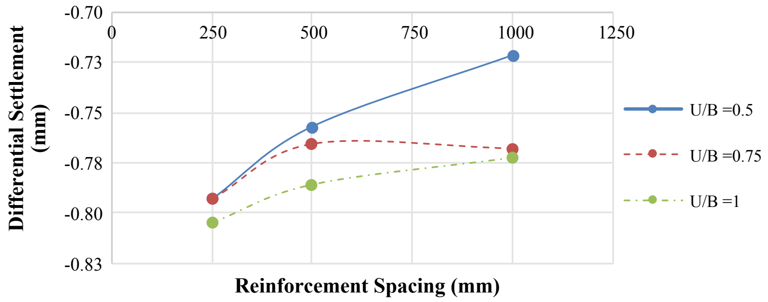

Fig. 3 Differential settlement versus reinforcement spacing for U/B ratios of 0.5, 0.75 and 1.00

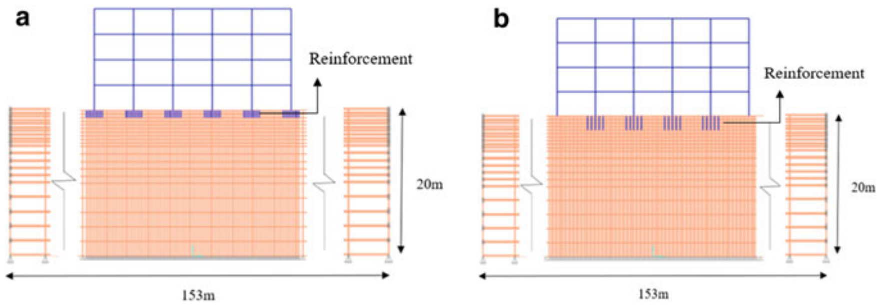

Fig. 4 **a** Front view of space frame resting on reinforced soil (reinforcement below all footings). **b**. Front view of space frame resting on reinforced soil (reinforcement below internal footings only)

5 Analysis of Space Frame Resting on Unreinforced and Reinforced Soil

To understand the effect of vertical reinforcement, the study is carried out on space frame resting on unreinforced and reinforced soil under static load of 31 kN/m. A total nine combinations of space frame resting on reinforced soil were developed and analysed for reinforcement spacing of 250, 500 and 1000 mm and the U/B ratios of 0.5, 0.75 and 1.00 (obtained from parametric study) under two cases by providing (i) Vertical reinforcement below all footings as shown in Fig. 4a and (ii) Vertical reinforcement only below internal footings as shown in Fig. 4b.

6 Results and Discussions

The structure resting on unreinforced soil is analysed initially and these results are taken as reference to compare the results of structure resting on reinforced soil for all the cases. The vertical settlement obtained is plotted against X/L in the longitudinal direction. The maximum vertical settlement in unreinforced soil is 115.81 mm with differential settlement of 17.61. The maximum horizontal displacement in unreinforced soil along longitudinal direction is 3.90 mm and across transverse direction is 4.82 mm.

6.1 Effect of Vertical Reinforcement on Vertical Settlement and Horizontal Displacements in Soil when Reinforcement is Provided Below all Footings

Figures 5, 6 and 7 show the plot of vertical settlement below the structure in unreinforced and reinforced soil for reinforcement spacing of 250, 500 and 1000 mm.

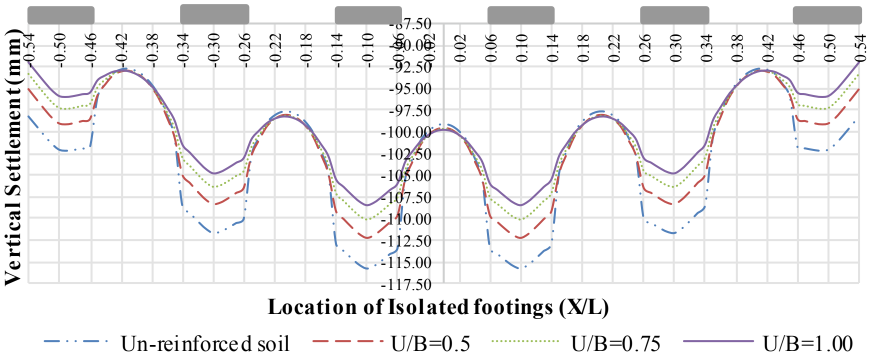

Fig. 5 Plot of vertical settlement in unreinforced and reinforced soil for $S = 250$ mm

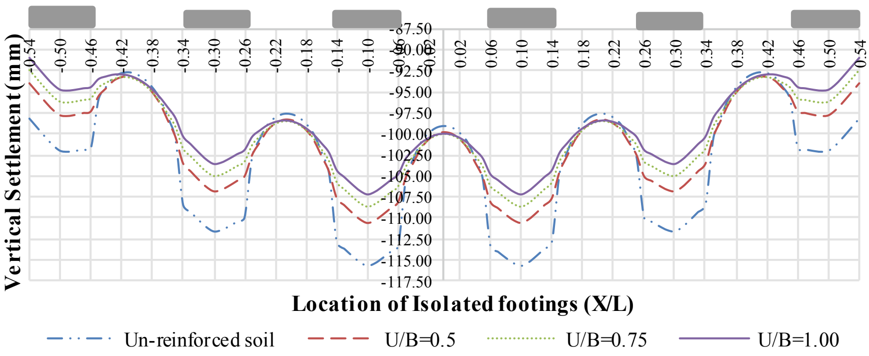

Fig. 6 Plot of vertical settlement in unreinforced and reinforced soil for $S = 500$ mm

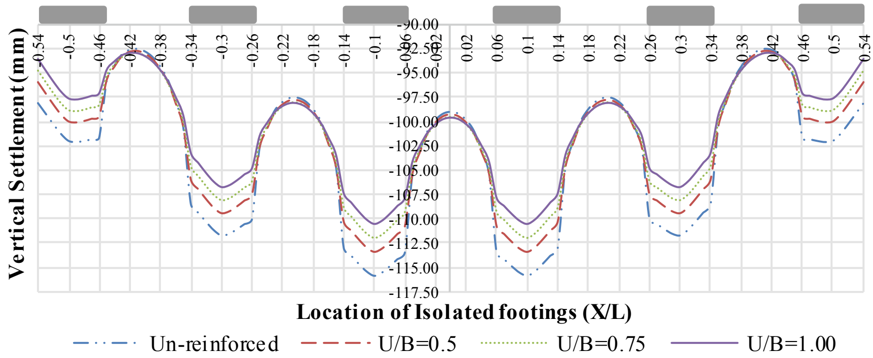

Fig. 7 Plot of vertical settlement in unreinforced and reinforced soil for $S = 1000$ mm

The comparison of maximum vertical settlement and differential settlement in unreinforced and reinforced soil is given in Table 4. The maximum horizontal displacement along longitudinal direction and transverse direction has been compared between unreinforced and reinforced soil and is tabulated in Table 5.

6.2 Effect of Vertical Reinforcement on Vertical Settlement and Horizontal Displacement in Soil when Reinforcement is Provided only Below Internal Footings

Figures 8, 9 and 10 show the plot of vertical settlement below structure in unreinforced soil and reinforced soil for reinforcement spacing of 250, 500 and 1000 mm. The comparison of maximum vertical settlement in unreinforced and reinforced soil is given in Table 6. The maximum reduction in both vertical settlement and differential settlement occurred for U/B ratio of 1.00 and for reinforcement spacing of 250 mm. The maximum horizontal displacement along longitudinal direction and transverse direction has been compared between unreinforced and reinforced soils. Table 7 gives the details of comparison of horizontal displacements in unreinforced and reinforced soil.

7 Conclusions

The following are the conclusion drawn:

- Inclusion of vertical reinforcement in soil reduces the vertical settlement and horizontal displacements in soil.

Table 4 Comparison of max. vertical settlement and differential settlement in unreinforced and reinforced soil

S. No	Spacing (mm)	U/B ratio	Max. vertical settlement (mm)			Differential settlement (mm)		
			Unreinforced soil	Reinforced soil	% Difference	Unreinforced soil	Reinforced soil	% Difference
1	250	0.50	115.81	110.66	−4.45	17.67	16.74	−5.26
		0.75		108.75	−6.10		16.41	−7.13
		1.00		107.14	−7.49		16.20	−8.31
2	500	0.50		112.18	−3.13		17.16	−2.86
		0.75		110.22	−4.84		16.81	−4.86
		1.00		108.52	−6.29		16.51	−6.56
3	1000	0.50		113.49	−2.01		17.48	−1.07
		0.75		112.02	−3.27		17.28	−2.20
		1.00		110.61	−4.49		17.08	−3.33

Table 5 Comparison of horizontal displacement in unreinforced and reinforced soil

S. No	Horizontal displacement (mm)	U/B ratio	Un-reinforced Soil	Reinforced soil					
				Spacing = 250 mm	% Difference	Spacing = 500 mm	% Difference	Spacing = 1000 mm	% Difference
1	Max. horizontal displacement in longitudinal direction	0.50	3.90	3.25	−16.79	3.49	−10.72	3.62	−7.37
		0.75		3.05	−21.95	3.29	−15.81	3.51	−10.18
		1.00		2.87	−26.31	3.11	−20.42	3.35	−14.02
2	Max. horizontal displacement in transverse direction	0.50	4.82	3.69	−23.41	4.13	−14.39	4.44	−8.0
		0.75		3.43	−28.68	3.85	−20.17	4.23	−12.30
		1.00		3.22	−33.24	3.61	−25.15	4.03	−16.44

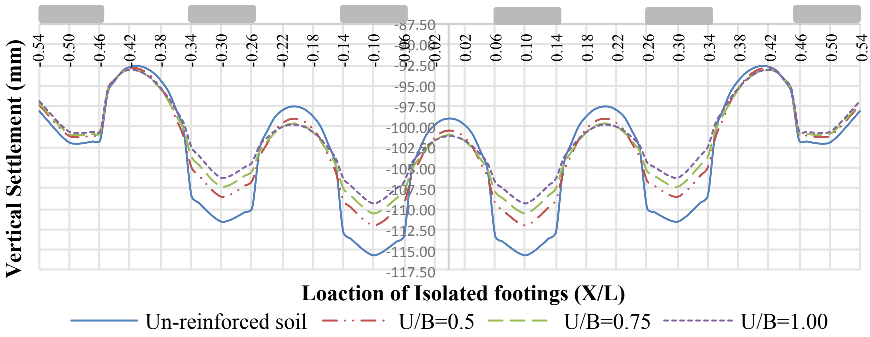

Fig. 8 Plot of vertical settlement in unreinforced and reinforced soil for $S = 250$ mm (reinforcement below internal footings)

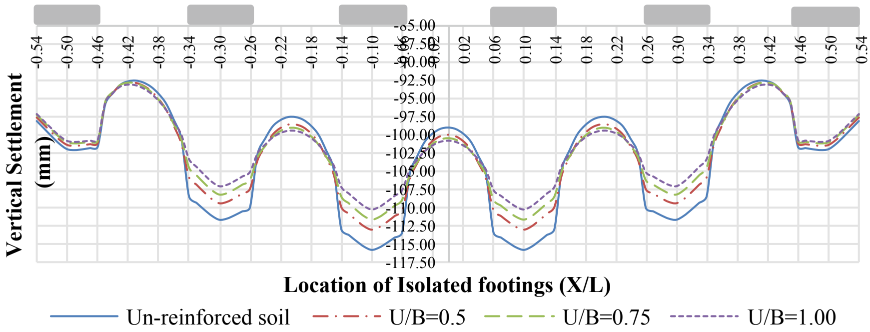

Fig. 9 Plot of vertical settlement in unreinforced and reinforced soil for $S = 500$ mm (reinforcement below internal footings)

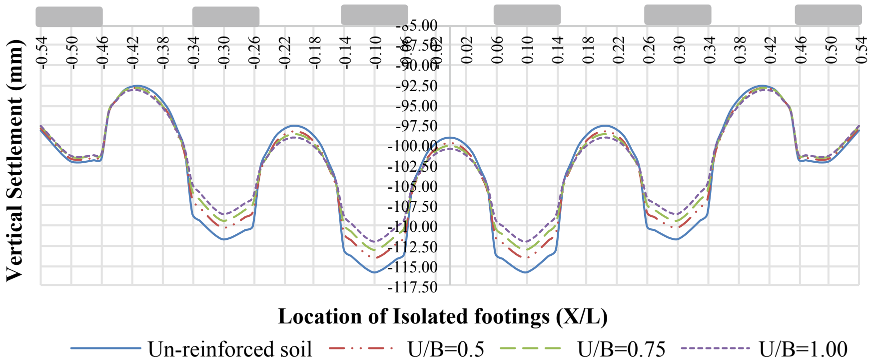

Fig. 10 Plot of vertical settlement in unreinforced and reinforced soil for $S = 1000$ mm (reinforcement below internal footings)

Table 6 Comparison of max. vertical settlement and differential settlement in unreinforced and reinforced soil (reinforcement below internal footings)

S. No	Spacing (mm)	U/B ratio	Max. vertical settlement (mm)			Differential settlement (mm)		
			Unreinforced soil	Reinforced soil	% Difference	Unreinforced soil	Reinforced soil	% Difference
1	250	0.50	115.81	112.01	−3.28	17.67	14.56	−17.6
		0.75		110.55	−4.54		13.36	−24.39
		1.00		109.28	−5.64		12.31	−30.34
2	500	0.50		113.16	−2.29		15.51	−12.24
		0.75		111.69	−3.56		14.31	−19.07
		1.00		110.39	−4.68		13.23	−25.12
3	1000	0.50		114.13	−1.45		16.29	−7.81
		0.75		113.05	−2.38		15.41	−12.79
		1.00		111.99	−3.30		14.53	−17.77

Table 7 Comparison of horizontal displacement in unreinforced and reinforced soil

S. No	Horizontal displacement (mm)	U/B ratio	Un-reinforced soil	Reinforced soil					
				Spacing = 250 mm	% Difference	Spacing = 500 mm	% Difference	Spacing = 1000 mm	% Difference
1	Max. horizontal displacement in longitudinal direction	0.50	3.90	3.63	−7.11	3.72	−4.76	3.79	−3.02
		0.75		3.51	−10.18	3.60	−7.88	3.71	−5.07
		1.00		3.40	−13.01	3.49	−10.7	3.62	−7.37
2	Max. horizontal displacement in transverse direction	0.50	4.82	3.78	−21.63	3.92	−18.72	4.44	−8.15
		0.75		3.50	−27.43	3.80	−21.21	4.15	−13.95
		1.00		3.12	−35.31	3.45	−28.47	3.87	−19.76

- Reduction in vertical settlement increases with decrease in reinforcement spacing and increase in U/B ratio.
- The maximum reduction in vertical settlement is 7.49% for a reinforcement spacing of 250 mm and for U/B ratio of 1.00 when reinforcement is provided below all footings. Whereas the reduction is 5.64% when reinforcement is provided only below internal footings.
- The maximum reduction in the differential settlement when vertical reinforcement is provided below all footings was found to be 8.31% for 250 mm reinforcement spacing and U/B ratio of 1.00.
- The maximum reduction in the differential settlement when vertical reinforcement is provided only below internal footings was found to be 30.34% for 250 mm reinforcement spacing and U/B ratio of 1.00.
- Horizontal displacement along longitudinal direction is reduced in the range of 7.37–26.31% when reinforcement is provided below all footings and 3.02–13.01% when reinforcement is provided only below internal footings.
- Horizontal displacement along transverse direction is reduced in the range of 8.00–33.24% when reinforcement is provided below all footings and 8.15–35.13% when reinforcement is provided only below internal footings.
- Reinforcement spacing of 250 mm and U/B ratio of 1.00 were observed to be the optimum reinforcement spacing and length.

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A Review on Soil Liquefaction Mitigation Techniques and Its Preliminary Selection

Punit Bhanwar and Trudeep Dave

Abstract Soil liquefaction is a phenomenon where a saturated cohesion-less soil substantially losses its strength as a result of reduction in effective stress and/or increase in the pore water pressure due to sudden change in stress condition, causing soil to behave like a liquid. Liquefaction may cause detrimental effects on infrastructures, loss of life and lifeline systems, which was historically observed in numerous earthquakes with major manifestations in 1964 Niigata, Japan, 1964 Alaska and recently in 2001 Bhuj, India, earthquake. In order to mitigate liquefaction effectively, knowledge about prevailing site conditions, subsurface stratification, project constraints, ground water table fluctuation, details of past seismic events, etc., and thorough technical knowledge of various liquefaction mitigation techniques is required. This article provides a concise summarization of various soil liquefaction mitigation techniques in current state of practice. Based on the mechanism of soil improvement, methods were categorized as (a) hydraulic modification, (b) soil structure densification and (c) reinforcement of soil. Finally, each of the listed methods was evaluated by generating a feasibility index through rated score analysis. These ratings were established on the basis of available literature while considering equal weightages to technology selection parameters. It is intended that the calculated feasibility index will serve geotechnical professionals by eliminating least feasible methods for given site conditions during initial stage of project.

Keywords Soil liquefaction · Preliminary selection · Feasibility index

1 Introduction

Soil liquefaction involves loss of strength and stiffness of saturated cohesion-less soils during dynamic loading or sudden change in soil's initial stress condition. Conceptually, it can be elucidated from fact that development of pore water pressure reduces effective stress, thereby decreasing shear strength of such soil. Soils having

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particle size in range of 0.002–2.36 mm (silt and sands) are found to be susceptible to this hazard [1]. The devastating effects of soil liquefaction on infrastructure are quite apparent from long prevailing history, i.e., Alaska and Nigata earthquake (1964), El Sentro earthquake (1979), Kobe (1995), to recent ones in Kocaeli (1999) and Bhuj (2001). The manifestations of above natural hazards were observed in manner such as settlement or tilting of building rested on liquefiable soil, rupture and cracking at joints of buried utilities, lateral spreading of slopes and tilting of quay walls. Studies were done to examine liquefaction susceptibility and characteristics of potentially liquefiable soils. In the past, simplified procedures and vivid criteria were developed for tests focused at evaluating liquefaction resistance [2]. Researches have analyzed field liquefaction observations from past major earthquake of century to obtain more detailed insights on soil liquefaction [3, 4]. Mitigation of soil liquefaction in an effective way which shall minimize costly repercussions and damages caused by it is currently sought by researchers and geotechnical professionals.

2 Mitigation of Soil Liquefaction

It is important to identify critical soil properties which directly affect the soil performance in an event of liquefaction. In an elementary study, these properties were found to be drainage characteristics, state of packing and reinforcement compatibility of soil [4, 5]. Several methods and techniques have been developed to improve these properties of soil, and few of them are listed as below (see Table 1).

Among this mentioned techniques, few are conventional while others are novel in nature. Geotechnical professionals are expected to select most appropriate ground improvement technology under prevalent site and project-specific conditions for ensuring adequate and cost-effective treatment. In absence of such approach remediation may damage structure to an unacceptable state as well as cost high [6]. However,

Table 1 Classification of soil liquefaction mitigation techniques

Techniques	Mechanism of improvement		
	Improvement of drainage characteristics	Densification of soil structure	Reinforcement of soil structure
	Induced partial de-saturation [20]	Vibro-compaction [11]	Permeation grouting [21]
	Earthquake PVDs [22]	Vibro-replacement [23]	Deep soil mixing [24]
	Electro-osmotic consolidation [25]	Dynamic compaction [26]	Jet grouting [27]
		Compaction grouting [28]	Passive Site remediation [16]
		Blasting compaction [6]	Microbial-induced calcite precipitation [29]

plenty of literature is already available, unraveling working mechanism and design considerations of this method. But, they are not structurally organized to deliver exact information on feasibility of selecting concerned methods. Although, previous studies [7] have suggested a preliminary selection process of such techniques but, recent researches and developments in soil liquefaction mitigation technology have fetched us with more sustainable and novel methods which, otherwise were absent in foresaid selection process. Thus, in purview of unavailability of such updated selection model, this article attempts to address the problem by detailing conventional as well as new methods on the basis of certain critical factors. Additionally, a feasibility index is also generated which shall be elaborated in this paper.

3 Site and Project Constraints

The selection of most effective technique for a particular site needs a comprehensive understanding of pivotal factors like site and project specific constraints [7]. Constraints from site condition and geometrics may comprise of limitations imposed by site accessibility, depth of water table, soil type and depth of treatment. While economical aspect, time for remediation, environmental aspect, validation of remediation and ease of operation are included in project specific constraints. It must be noted that parameters like site accessibility, depth of water table, soil type, depth of treatment, environmental aspect, validation of remediation and ease of operation are qualitative in nature whereas economical aspect and time for remediation are quantitative in nature. Both of them are quantified and rated suitably as illustrated hereafter.

4 Quantification of Site and Project Constraints

Site and project constraints identified in previous sections were qualitatively assessed to screen out least appropriate methods [7]. However, in order to arrive at a measurable judgment for technique selection, it is necessary to convert all qualitative parameters in a quantifiable sense, so that each technique can be valued comprehensively. Also, this approach may further seek to optimize each constraint to arrive at best method among given choice. To accomplish this, a quantified rating score is proposed to be evaluated. This quantified rating is ultimately expected to furnish 'Feasibility index (F.I.)'.

Initially to begin with, total response scored is calculated by summing up suitability of technique in terms of 'True' or 'False' for each governing criteria. As linguistic term 'True' and 'False' are denoted by 'T' and 'F' respectively and are assumed to fetch numeric value one and zero in succeeding rating analysis. This being divided by maximum possible score yields efficacy scored in percentage. After which

Table 2 Gradation and rating scheme for qualitative parameters

Efficacy scored (E.S %)	Gradation	Performance level	Quantified rating
$\frac{\text{Total response score} \times 100}{\text{Maximum score}}$	E.S = 0%	*Poor*	*
	0% < E.S ≤ 40%	*Below average*	**
	40% < E.S ≤ 60%	*Average*	***
	60% < E.S < 100%	*Good*	****
	E.S = 100%	*Excellent*	*****

Table 3 Techniques considered for analysis with respective abbreviations

Abbreviation	Techniques	Abbreviation	Techniques
IPS	Induced partial saturation	CG	Compaction grouting
PVDs	Earthquake pre-fabricated vertical drain	BC	Blasting compaction
EOC	Electro-osmotic consolidation	PG	Permeation grouting
VC	Vibro-compaction	DSM	Deep soil mixing
VR	Vibro-replacement	JG	Jet grouting
DC	Dynamic compaction	R&R	Removal and replacement
MICP	Microbial-induced calcite precipitation	PSR	Passive site remediation

conversion of efficacy scored into performance level and further into quantifiable rating is worked out by preset gradation as tabulated below (see Table 2).

Soil liquefaction mitigation techniques under consideration for above analytic assessment are also tabulated with their respective abbreviation, which shall be used several times in article (see Table 3).

4.1 Quantifying 'Depth of Water Table' Effect

Effect of depth of water table refers to suitability of a technique for existing and forecasted water table at that particular site. Fully or partially loose saturated soils generally located at shallow depth, i.e., less than 15 m are most susceptible to liquefaction. The high depth of water table in such scenario can critically affect effectiveness of ground improvement technique employed. To ascertain foresaid effectiveness, suitability of considered techniques subjected to possible depths of water table is evaluated on the basis of empirical guidelines from literature by [7–9], etc. Further, these are translated to corresponding quantified rating with methodology proposed in earlier sections (see Table 4).

Table 4 Evaluation of suitability to various depth(s) of water table

Techniques	Suitability to depth(s) of water table				Efficacy scored (%)	Performance level	Quantified rating (Q.R.[1])
	0–2 m	2–3 m	3–6 m	6–12 m			
IPS	T	T	T	T	100	*Excellent*	*****
PVDs	T	T	T	T	100	*Excellent*	*****
EOC	T	T	T	T	100	*Excellent*	*****
VC	F	F	F	T	25	*Below average*	**
VR	T	T	T	T	100	*Excellent*	*****
DC	F	F	T	T	50	*Average*	***
CG	T	T	T	T	100	*Excellent*	*****
BC	T	T	T	T	100	*Excellent*	*****
PG	T	T	T	T	100	*Excellent*	*****
DSM	T	T	T	T	100	*Excellent*	*****
JG	T	T	T	T	100	*Excellent*	*****
R&R	F	F	T	T	50	*Average*	***
MICP	T	T	T	T	100	*Excellent*	*****
PSR	T	T	T	T	100	*Excellent*	*****

4.2 *Quantifying Economic Consideration*

Economical consideration reflects cost per cubic meter of using technique in field conditions excluding mobilization cost and labor charges, which may vary regionally. The economical aspect plays a key role in selection of any soil improvement technique. It is always desirable to select remediation technique having least cost of treatment from available choices. To achieve same, the quantified rating for all considered techniques is directly extracted on the basis of devised scheme for cost levels (see Table 5). In proposed scheme of rating, the treatment cost per cubic meter is segregated on five cost levels, which are established from cost analysis referenced from [9]. It must also be noted that least treatment cost range is being attributed to

Table 5 Devised scheme of rating techniques on economical aspect

Treatment cost per m^3 ($\$/m^3$)	Qualitative level	Quantified rating (4)
0–20	*Excellent*	*****
20–50	*Good*	****
50–100	*Average*	***
100–200	*Below average*	**
More than 200	*Poor*	*

Table 6 Evaluation of quantified rating for cost of remediation

Techniques	Treatment cost per cu. meter ($/m^3)					Performance level	Quantified rating (Q.R.^{4})
	0–20	20–50	50–100	100–200	More than 200		
IPS	T					*Excellent*	*****
PVDs	T					*Excellent*	*****
EOC		T				*Good*	****
VC	T					*Excellent*	*****
VR		T				*Good*	****
DC	T					*Excellent*	*****
CG					T	*Poor*	*
BC			T			*Average*	***
PG					T	*Poor*	*
DSM				T		*Below average*	**
JG					T	*Poor*	*
R&R		T				*Good*	****
MICP	T					*Excellent*	*****
PSR				T		*Below average*	**

excellent qualitative level as well as maximum corresponding rating. This is done to assure that techniques with least cost of treatment are rated as best.

In the proposed scheme, evaluation is done by entering response as 'T', i.e., true corresponding to that cost group and finally converted into corresponding quantified rating (see Table 6).

As far as quantification of remaining constraints is concerned instead of lump sum evaluation, they have been simply elaborated on respective governing premise required for consequent establishment of ratings (i.e., **Q.R^2**, **Q.R^3**, etc.)

4.3 Quantifying Suitability to Soil Type

Soil type relates compatibility of a technique with soil present at site. Amongst different soil types, slightly cohesive and loose cohesion-less soils which may be partially or fully saturated are mostly prone to liquefaction in seismic event. However, each liquefaction mitigation technique has its own suitable soil type where it functions effectively. The effectiveness of remedial technique corresponding to medium and highly liquefiable soil types, i.e., (GM—Silty Gravel; SP—Poorly graded sand; SW-Well graded sand SM—silty sand; SC—clayey sand; ML—silt) [7] is ascertained on basis of literature sourced from [4, 10–12].

4.4 Quantifying Suitability to Depth of Treatment

Depth of treatment refers to practical applicability of a particular technique for given liquefiable layer of soil, where treatment has to be implemented. The overburden depth below which soil is likely to liquefy was roughly estimated to be around 25 m [2]. This provides limit to the possible depth of treatment and its further split into depths (0–3 m; 3–6 m; 6–12 m; 12–25 m; more than 25 m) where liquefaction is expected to occur. The techniques employed for liquefaction mitigation may express their working limitations for particular depths of treatment and henceforth their suitability is evaluated to screen out best among them. The background for such evaluation is attributed to work of [9, 13, 14].

4.5 Quantifying Time of Remediation

Time for remediation expresses amount of time required by a particular technique to remediate one cubic meter of soil. Time constraints are important from project execution point of view as it is seen that time directly relates to cost of project. It is always intended to select technique consuming lowest time to remediate any problematic construction activity. To achieve same for mitigation of soil liquefaction, the quantified rating for all considered techniques is directly extracted on the basis of devised scheme for time duration levels (see Table 7). In proposed scheme of rating, time required to remediate per cubic meter is split on five duration levels, and these levels are setup from past empirical findings of [9, 15–17].

It must also be noted that least treatment time range is being considered as excellent qualitative level and is followed same by corresponding quantified rating. This ensures that techniques with least duration of treatment are graded as best. In the proposed scheme, evaluation is done by entering response as ‘T’, i.e., true if technique furnishes that particular duration level, which is then finally reformed to corresponding quantified rating.

Table 7 Devised scheme of rating techniques for time of remediation

Time of remediation per m^3 (Days/m^3)	Performance level	Quantified rating
0–5	*Excellent*	*****
5–10	*Good*	****
10–20	*Average*	***
20–50	*Below average*	**
More than 50	*Poor*	*

4.6 Quantifying Site Accessibility

Site accessibility includes dimensional freedom to work, i.e., available headroom and planar space at site, as well as troubles caused by remote location and terrain profile of working site. Site accessibility factor is generally an intuitively important issue in construction industry. This cannot be ignored considering mitigation of soil liquefaction is bound to face variety of construction constraints [18]. The selection of particular technique for given liquefiable soil is effectively aided if we evaluate its suitability to anticipated site accessibility factors. To accomplish this, each technique is analyzed in terms of suitability with respect to site accessibility constraints defined as Low Headroom; clearances; Low working planar space; Near-by Infrastructure Lifelines; All ground Profiles; and Remote location of Site.

4.7 Quantifying Suitability to Environmental Considerations

Environmental impact represents effect of using particular technique, to nearby environment in terms of sustainability, i.e. disturbance to nearby structures, lifelines, health of concerned manpower employed, effect on ecosystem involved, etc. The environmental concern of selecting certain mitigation technique cannot be disregarded in era of sustainable development. Each method may have their own environmental impact [19], which is proposed to be assessed by calculating suitability to environment on five identified parameters. Agreement to these parameters such as No Disturbance to nearby Structure; No Disruption of Ecosystem; No waste/spoils generation; Promotes Sustainability is recorded as True and False as mentioned earlier. The technique selection considering environmental valuation is surely expected to inspire confidence in governmental agencies, non-profit organizations and environmental regulatory, etc., clientele.

4.8 Quantifying Suitability to Ease of Operation, Validation of Remediation

Ease of operation is defined as ease with which technique can be actually used in the field considering skills and expertise of manpower employed as well as technological limitations in remediation of geotechnical problem. Validation of remediation reflects add-on facilitation to supervising authority (Quality Assurance personnel) for maintaining quality control by sampling and real-time monitoring of ongoing remediation process. Specifically, this can be ascertained by evaluating suitability of technique in regard to No Specialized Manpower requirement; No Advanced Technological Requirement; Supports Sampling and Quality Control; Real-Time Monitoring in

Table 8 Proposed scheme of weights for levels of expectation and feasibility index

Levels of expectation	Weightage	Feasibility index
Extremely important	1	$\frac{\text{Q.R}^1 \times \text{Weightage}^1 + \ldots + \text{Q.R}^n \times \text{Weightage}^n}{n}$
Highly important	0.8	
Moderately important	0.6	
Less important	0.4	
Not important	0.2	

Note 'n' express expandable nature of index which is meant for inclusion of more parameters in future, for current study 'n' is limited to 8

terms of true and false. However, evaluation on such parameters must be backed up by extensive case studies, which was available for routine methods from [9].

5 Formation of Decision Matrix

After performing explicit quantification of all sites and project constraints, a decision matrix for all considered techniques is formed. Concept of weighted rating was added to the decision matrix to obtain feasibility index. The feasibility index so calculated attempts to summarize efficacy of applying any particular technique in possible liquefaction mitigation scenario and helps to zero down to most appropriate technique. Higher value of feasibility index suggests more suitable technique. This feasibility index is based on weightage schemes (see Table 8) devised to cover various levels of expectations from technique selection authority. The levels of expectations mentioned below, query selector how important is that technology selection parameter in selection process and are weighed accordingly.

6 Case Study

To demonstrate effectiveness of proposed evaluation, a case study has been incorporated from literature [7]. The problem is defined as follows:

- **Location**: Liquefied natural gas plant in Delta, British Columbia, Canada
- **Type of Structure**: Shallow reinforced concrete raft foundation
- **Performance Objective**: To densify the foundation soil and to minimize the liquefaction induced settlement in soils below the foundation
- **Depth of liquefiable layer**: 22 m
- **Depth of water table**: 1–2 m below G.L
- **Area to be remediated**: 12×12 m^2 in foot print of Foundation

Table 9 Extracted weightages from problem definition

Selection parameter	Level of expectation	Weighs
Depth of water table—$Q.R^{(1)}$	Extremely important	1
Soil type—$Q.R^{(2)}$	Moderately important	0.6
Depth of treatment—$Q.R^{(3)}$	Extremely important	1
Economic consideration—$Q.R^{(4)}$	Highly important	0.8
Time of remediation—$Q.R^{(5)}$	Extremely important	1
Site accessibility—$Q.R^{(6)}$	Extremely important	1
Environmental aspect—$Q.R^{(7)}$	Extremely important	1
Ease of operation and performance validation—$Q.R^{(8)}$	Moderately important	0.6

- **Project Constraints**: (I) 2 m of headroom clearance available for construction, (II) potential damage to the existing settlement sensitive utilities near construction site (III) an accelerated construction schedule, (IV) adjacent existing utilities that are sensitive to ground movement and disturbance.

During analysis, either equal weightage can be assigned to each critical parameter or moderated as per project requirements (see Table 8). After extraction of corresponding weightages (see Table 9), they are fed into decision matrix to evaluate feasibility index of techniques under consideration for that particular site (see Table 10).

7 Concluding Remarks

It can be observed from the decision matrix that IPS, MICP and PG can be prioritized as recommended technique for site remediation. Authors of the said literature [7] recommended permeation grouting (PG) technique for the above-mentioned site conditions. However, it should be noted that IPS and MICP are very novel techniques still under research and field scale application is still underway. Hence, it can be concluded that the proposed evaluation approach can be applied at site by judicious weighing of critical parameters before finalizing a particular technique. The proposed selection method can be sought as preliminary guidance to geotechnical professionals and researchers for screening out least appropriate technique for mitigation of soil liquefaction. However, a more rigorous approach is recommended to further optimize the selection process.

Table 10 Summarization of rated analysis and evaluation of feasibility index

Quantified performance rating	Decision matrix for soil liquefaction mitigation techniques													
	IPS	PVDs	EOC	VC	VR	DC	CG	BC	PG	DSM	JG	R&R	MICP	PSR
$Q.R^{(1)}$	*5*	*5*	*5*	*2*	*5*	*3*	*5*	*5*	*5*	*5*	*5*	*3*	*5*	*5*
Weigh	1	1	1	1	1	1	1	1	1	1	1	1	1	1
$Q.R^{(2)}$	*5*	*5*	*4*	*4*	*4*	*4*	*4*	*4*	*4*	*4*	*4*	*4*	*5*	*5*
Weigh	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
$Q.R^{(3)}$	*4*	*4*	*2*	*4*	*5*	*4*	*4*	*3*	*5*	*4*	*5*	*2*	*5*	*5*
Weigh	1	1	1	1	1	1	1	1	1	1	1	1	1	1
$Q.R^{(4)}$	*5*	*5*	*4*	*5*	*4*	*5*	*1*	*3*	*1*	*2*	*1*	*4*	*5*	*2*
Weigh	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
$\mathrm{Q.R}^{(5)}$	*5*	*1*	*2*	*3*	*5*	*1*	*1*	*5*	*4*	*4*	*3*	*5*	*5*	*1*
Weigh	1	1	1	1	1	1	1	1	1	1	1	1	1	1
$Q.R^{(6)}$	*4*	*2*	*4*	*2*	*2*	*2*	*4*	*2*	*4*	*2*	*4*	*2*	*5*	*2*
Weigh	1	1	1	1	1	1	1	1	1	1	1	1	1	1
$Q.R^{(7)}$	*5*	*5*	*4*	*2*	*1*	*2*	*4*	*1*	*4*	*4*	*3*	*1*	*2*	*5*
Weigh	1	1	1	1	1	1	1	1	1	1	1	1	1	1
$Q.R^{(8)}$	*2*	*4*	*3*	*5*	*4*	*5*	*4*	*2*	*4*	*3*	*3*	*5*	*3*	*4*
Weigh	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Feasibility index	3.9	3.3	3.05	2.8	3.25	2.675	2.95	2.75	3.45	3.1	3.125	2.7	3.85	3.125

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Geopolymer Stabilization of Soft Clays—An Emerging Technique

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Abstract Large tracts of soft clay deposits are present in many world nations especially along their shore lines and estuaries. These deposits are characterised by their high compressibility with low shear strength making them unsuitable to serve as foundation bed. However, in view of enormous economic activity along the coasts, large-scale infrastructure development becomes inevitable. In view of this, the soft clay deposits are to be improved by suitable methods of stabilization. In this direction, stone columns, preloading with or without vertical drains, deep lime or cement mixing and electro-osmosis have been popularly used across the world. In the recent years, efforts are being made to use geopolymer technology as an alternative to lime/cement mixing as an attempt to reduce the carbon footprint. Besides carbon footprint reduction, several researchers (Duxson et al. in J Mater Sci 42(9):2917–2933, (2007) [1]; Majidi in Mater Technol 24(2):79–87, (2009) [2]; Neupane et al. in Mech Mater 103:110–122 (2016) [3]) reported the technical advantages of high early strength, extraordinary durability, resistance to chemical attack and ability to immobilize toxic atoms for geopolymer compared to conventional lime/cement. Keeping in view these recent trends in geopolymer technology, an attempt is made to study the influence of ground granulated blast furnace slag (GGBS) binder with different molarity of activator, the sodium hydroxide (NaOH). The soft clay is simulated by preparing the clay paste at 0.75, 1.0 and 1.25 times the liquid limit water content. At these initial clay consistencies, the influence of GGBS and NaOH is studied. From this study, it is revealed that the unconfined compressive strength of stabilised clay

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increases with increase in activator to binder (A/B) ratio and curing period for any binder content. Increased molarity of activator has little influence on the strength gain. The strength gain is observed to be higher at higher initial consistency of clay.

Keywords Soft clay · Geopolymer stabilization · Activator to binder ratio · Molarity of activator · Unconfined compressive strength

1 Introduction

Large tracts of soft soils are present along the coast lines of many world nations. These deposits are characterized by high natural water content coupled with low shear strength making them unsuitable to support any civil engineering structures [4]. However, essentially the major infrastructure such as transportation routes, ports and harbour structures are to be built over such deposits in view of high economic activity in the coastal regions [5]. In order to build the structures in these deposits, several ground improvement techniques such as preloading with vertical drains, stone columns, electro-osmosis and in the recent times, the deep soil mixing were promulgated all over the world.

These techniques are suitable for a specific ground condition depending upon the available time and resources. All these improvement techniques except deep soil mixing require considerable time before the desired degree of improvement are attained. In view of short duration improvement of soft soils by deep soil mixing, this technique gained global prominence for its wide use. For deep soil mixing, lime and cement have been the most commonly used binders so far [6–9]. In view of environmental concerns with these binders, a great deal of research has been taken up by several investigators to develop alternative binders such as geopolymers [10–13].

Geopolymers are cementitious binders produced by combining industrial by-products and waste products having high amorphous alumina and silica contents, such as flyash, ground granulated blast furnace slag, metakaolin, etc., with a liquid alkaline activator (like sodium/potassium hydroxide and sodium/potassium silicate), rich in soluble metals, like sodium and potassium [10, 14]. The geopolymerization is a fast chemical reaction which involves four main stages (i) dissolution of solid reactants in an alkaline solution releasing silica and alumina atoms, (ii) diffusion of the dissolved species through the solution, (iii) polycondensation of the alumina and silica complexes with the added alkaline activator and the formation of gel, (iv) hardening of the gel that results in the final polymeric product [10–12]. The geopolymeric product thus formed would be a calcium sodium aluminosilicate hydrate, C–N–A–S–H [2, 12, 14]. Further, it is reported that geopolymers are more durable than cement [1–3, 14, 15]. In view of these developments, an attempt is made to stabilize the soft soils using geopolymer.

Table 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 (gm/cc)	1.54
Specific gravity, Gs	2.65
IS Soil classification	CH
pH	7.4

Table 2 Chemical composition of GGBS

Oxide	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	Na_2O
Composition (%)	30.1	13.4	5.7	45.8	6.1	0	0.2

2 Materials and Methodology

2.1 Materials

Soft clay. The locally available black cotton soil (Table 1) mixed with potable tap water at water content close to and slightly more than liquid limit is used.

Sodium hydroxide (*NaOH*). Sodium hydroxide, also known as Caustic Soda, is purchased from Fisher Scientific in pellets form with density 2.1 g/cm^3, mass of 39.9971 g/mol and solubility in water as 111 g/100 ml (20 °C).

Ground granulated blast furnace slag (GGBS). The GGBS powder is obtained from Vizag Steel Plant, Visakhapatnam, Andhra Pradesh, India. Chemical composition is given in Table 2.

2.2 Methodology

The soil slurry at the desired consistency is prepared by adding the corresponding water content. The binder (B) and activator (A) are also mixed together in the required

A/B ratio. The activated binder slurry is mixed with the soil slurry and the test specimens are prepared by filling the mixture in the moulds. These specimens are cured for the required curing period by keeping the moulds in polyethylene bags and covering them with wet gunny bags for 3 days. Then the specimens are removed from the moulds and kept in the same polyethylene bags to continue the curing. At the respective curing periods of 7 and 28 days, the specimens are tested for their unconfined compressive strength as per ASTM D1633 [16]. The durability tests for wetting and drying are also carried out on the specimens after 28 days curing period as per the procedure given by ASTM D559/559 M [17].

3 Results and Discussion

The results obtained from experimental investigation are presented in the following tables and figures.

3.1 Unconfined Compressive Strength

It can be observed from Fig. 1 and Table 3 that for the water content equal to 0.75 times the liquid limit ($0.75w_L$), the unconfined compressive strength is increasing with the binder content for any A/B ratio at activator concentration of 8 molarity. There is a steep increase in strength upto 20% binder content and thereafter, the rate of increase in strength gain is reduced. The reduction in rate of strength gain beyond 20% binder content could be attributed to the presence of unreacted binder particles producing geopolymeric gels which form flocculated crystals that grow larger with time causing internal forces that lead to non-uniformity and minor bond breakages. Further, for the activator concentration beyond 10 M, the strength reduction is noticed and it could

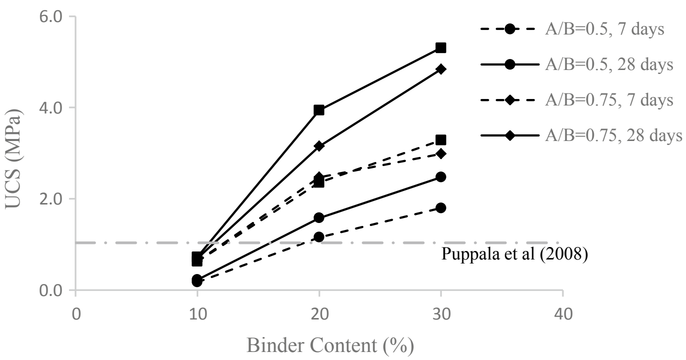

Fig. 1 Variation of UCS with binder content, A/B ratio and curing time at $0.75w_L$ and 8 M

Table 3 Variation of unconfined compressive strength with initial water content, molarity of NaOH, A/B ratio and binder content

Initial water content	Molarity	A/B ratio	UCS (MPa)		
			Binder content (%)		
			10	20	30
$0.75w_L$	8	0.5	0.227	1.578	2.474
		0.75	0.698	3.153	4.836
		1	0.724	3.941	5.304
	10	0.5	0.252	1.928	2.730
		0.75	0.742	3.366	4.891
		1	0.795	4.059	5.416
	12	0.5	0.266	1.919	2.712
		0.75	0.729	3.246	4.359
		1	0.774	3.367	4.834
w_L	8	0.5	0.121	1.022	1.521
		0.75	0.396	2.421	3.627
		1	0.446	3.002	4.081
	10	0.5	0.148	1.168	1.671
		0.75	0.420	2.499	3.576
		1	0.489	3.109	4.188
	12	0.5	0.144	1.474	1.834
		0.75	0.413	2.203	3.289
		1	0.478	2.533	3.659
$1.25w_L$	8	0.5	0.101	0.723	1.174
		0.75	0.229	1.714	2.659
		1	0.299	2.009	3.203
	10	0.5	0.128	0.846	1.348
		0.75	0.298	1.900	2.587
		1	0.356	2.123	3.241
	12	0.5	0.112	0.820	1.310
		0.75	0.242	1.685	2.618
		1	0.329	1.900	3.122

be due to faster rate of reactions that result in non-uniformity in the strength gain. However, the target strength of 1.034 MPa [18] for deep mixing could be obtained for binder content of 20% for A/B ratio greater than or equal to 0.75. Further, beyond the A/B ratio of 0.75, the strength gain is nominal which could be attributed to the undesirable morphological changes at higher activator content. For the water content equal to liquid limit (w_L) and 1.25 times liquid limit ($1.25w_L$) also the patterns of strength gain are similar to that at $0.75w_L$ except the variation in magnitude of the

UCS values (Table 3). It can be seen from the table that the UCS values are decreasing with increasing initial water content for any binder content and A/B ratio which could be attributed to the reduced activator concentration and presence of water voids in the cemented material.

3.2 Durability

From Fig. 2, it can be observed that there is a reduction in volume of the specimens for each cycle of wetting and drying. It can be observed from Fig. 3 that there is increase in percentage of soil-binder loss for each cycle of wetting and drying. The increase of mass loss upto the third cycle maybe due to the leaching of dissolved

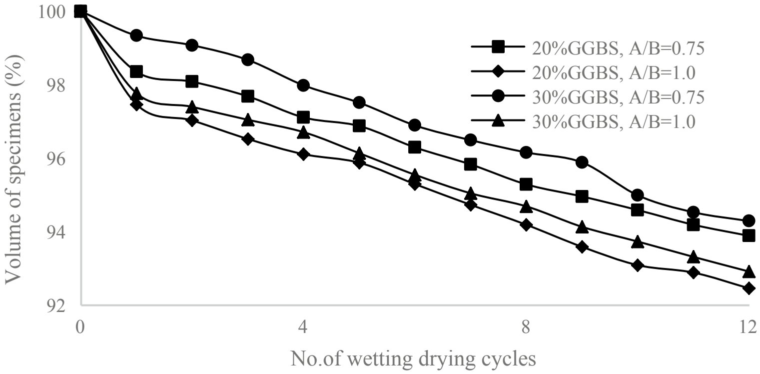

Fig. 2 Volume of specimens (%) for 12 cycles of wetting and drying of geopolymer specimens at w_L and 8 M for binder content 20 and 30% and A/B ratio of 0.75 and 1.0

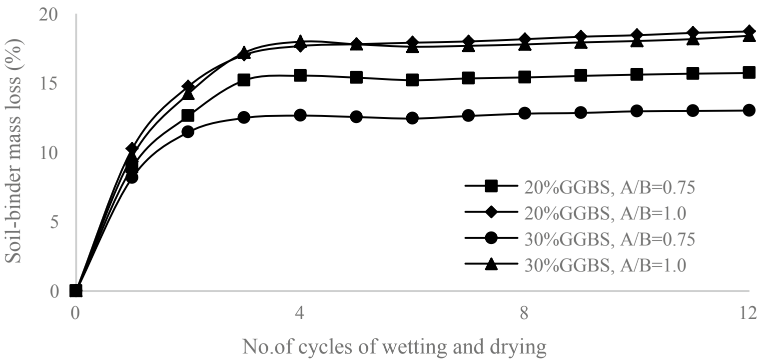

Fig. 3 Soil-binder mass loss (%) for 12 cycles of wetting and drying of geopolymer specimens at w_L and 8 M for binder content 20 and 30% and A/B ratio of 0.75 and 1.0

Si, Al and Ca. However, after three cycles, the geopolymeric network was strongly formed such that mass loss was not much affected by wetting and drying cycles. It can also be observed that the soil-binder specimens show better durability characteristics like lower volume change and lower mass loss at A/B ratio of 0.75 for 20 and 30% binder contents after 12 cycles of wetting and drying.

4 Conclusions

The following conclusions are drawn based on the experimental study carried out in this investigation.

- UCS values increased with the binder content for any molarity of activator. For a given molarity, there is a steep increase in strength upto 20% binder content and thereafter the rate of strength gain is slightly reduced.
- For a given binder content and molarity, the UCS values increased with increase in activator to binder ratio from 0.5 to 1.0. However, for A/B ratio greater than 0.75, the rate of strength gain is reduced.
- For the initial consistencies investigated, strength is observed to be increasing with increase in concentration of NaOH from 8 to 10 M and beyond this the strength reduction is observed. For any combination of binder and activator contents, the strength depends on its initial consistency.
- The target strength of 1.034 MPa could be obtained for all initial consistencies of clay using 20% binder and A/B ratio greater than or equal to 0.75. For cement stabilization, cement content required to attain this target strength is found to be 30%.
- The durability of geopolymer stabilized samples is found to be superior to cement stabilized samples.
- This study revealed that the UCS and durability criteria for soft clay stabilization could be achieved using geopolymer stabilization and hence it can be a promising alternative to traditional stabilizers like cement.

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Slope Stability Analysis for an Airport Runway in North-East India

Yamini Grover, Pranjal Mandhaniya, and J. T. Shahu

Abstract The development of infrastructure in hilly regions requires cutting of slopes and applying slope stabilisation techniques to prevent landslides. In this paper, the results of slope stability analysis are presented for a newly constructed airport in north-east India. This study was conducted on an upcoming airport runway in a hilly region. The slope stability analyses for the retaining structures on the side of the runway are performed using computer simulation. These simulations are used to analyse static and pseudo-static stability of slopes. The stability of slopes is checked using Morgenstern-Price method, which is based on limit equilibrium. An RCC cladding wall supports the cut slopes. Cable anchors and rock bolts support this wall. Further, self-drilling anchors (SDAs) are used on the open ground slopes to resist local failures. At some places, cable anchors are used in combination with self-drilling anchors to support steep slopes. The study shows that long anchors with shear reinforcement are the best solution to stabilise steep ground slopes.

Keywords Slope stabilisation · GeoStudio · Reinforcement · Slope/W · Morgenstern price method · Hilly terrain · Airport · Runway

1 Introduction

The world is rapidly growing and getting smaller day by day. Transportation is a necessity for a fast-growing population. Travel by air is the fastest way to travel as it is also getting cheaper by the day. Airport construction facilitates the need

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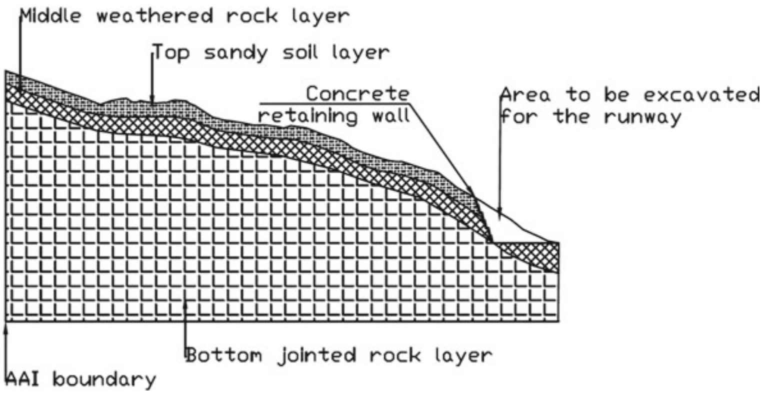

Fig. 1 Schematic diagram of hill slope to be stabilised

for transportation, but it has its hurdles. From environmental regulations to noise pollution, airport authorities face a daunting task before constructing an airport.

In this study, one such case is taken where the airport is being built at the foothills of Himalaya Mountains. The primary concern for designing this kind of airport is slope stability. Cutting natural slopes to make way for a runway and landing requires meticulous analysis. Figure 1 shows a specific section of a proposed airport site and the part to be excavated. The stabilisation is needed to be done in the land boundary of Airport Authority of India (AAI).

In this paper, SLOPE/W (A part of GeoSlope software package) is used to analyse the factor of safety (FoS) of 18 sections of cut slopes running parallel to the airport runway.

2 Methodology

Morgenstern-Price method in GeoSlope (Slope/W) considers both normal and shear inter-slice forces and satisfies both force and moment equilibrium. This method allows for a variable relationship between the inter-slice shear and normal forces. Limit equilibrium method allows calculating the FoS by the ratio of average shear strength along a critical shear surface to the average equilibrium shear stress mobilised along the same surface. GeoSlope enables users to regulate soil properties, pore-water pressure conditions, and loading conditions as well as slip surface profiles to generate the desired output. The aim is to determine reinforcement requirements of all slope sections economically and feasibly to attain global slope stability. In this paper, an attempt is made to provide stability solutions to 18 slope sections achieving a minimum FoS for static and pseudo-static analysis as per standard codes with the help of Slope/W.

Table 1 Geotechnical properties of layers

Layer	Material model	Angle of internal friction (ϕ, degrees)	Cohesion (kPa)	Unit weight (kN/m^3)
Overburden undrained	Bilinear	Refer to Table 2		18
Highly weathered saturated rock	Bilinear			24
Jointed rock	Mohr–Coulomb	36	44	28
Side wall concrete	Mohr–Coulomb	0	3000	25

Table 2 Geotechnical properties of layers

Top soil σ-τ data (large direct shear test)		Weathered rock σ-τ data	
Normal stress (kg/cm^2)	Shear stress (kg/cm^2)	Normal stress (kg/cm^2)	Shear stress (kg/cm^2)
0	0	0	0
0.77	0.598	1.2	1.048099
1.5	1.077	2.4	1.522851
2.256	1.42	3.6	1.894832

2.1 Material Properties

Properties of materials are identified by laboratory testing of data [1]. Bore logs and SPT data collected from the site divide it into three distinct layers. The top layer is a sandy soil, the middle one is a weathered rock layer, and the last layer is a jointed rock mass. Geotechnical properties of soil for a section are taken from the borehole data nearest to the section.

Tables 1 and 2 shows the properties of soil layers required for software input.

2.2 Reinforcement Properties

The following types of reinforcement systems are allowed to be installed for the global stability of slope sections.

1. Self-drilling anchors (SDAs): Long/medium nails which are bolted into loose soils to stabilise them [2, 3].
2. Rock bolts: These are smaller in length and diameter from SDAs. These are used to transfer the loads from weak to strong stratum [2, 3].

Table 3 Design material parameters of systems for global stability analysis

Parameter	Self-drilling anchors	Cable anchor	Rock bolts
Length (m)	3–8	Bonded = 7 Overall = 20, 25 and 30	4
Out of the plane spacing (m)	2.5 or/and 3.0	3	1.5
Bond diameter (mm)	100	125	25
Pullout resistance (kPa)	60	490	60
Tensile strength (kN)	220	900	415
Shear strength (kN)	110	0	207

3. Cable anchors: This is long, complex cables, more flexible than nails and generally used as a post-tension member, unlike nails, which are pretensioned [2].

The properties given in Table 2 are used in slope stability analysis for these three types of reinforcements.

The pullout resistance of SDAs and rock bolts is taken as 60 kPa based on a previously performed material testing by the supplier. Pullout resistance of reinforcement is the function of friction between soil and the outer layer of the reinforcement. Field pullout tests performed on the reinforcement members render the values given in Table 3.

3 Numerical Analysis in Slope/W

Material properties given in Tables 1 and 2 are assigned to the layers, as shown in Fig. 1 using Mohr–coulomb and bilinear material model available in Slope/W interface. All the other properties, such as suction, are kept as default in the property definition.

Reinforcements are applied using reinforcement loads module of Slope/W. Cable anchors [2] are modelled using anchor configuration as it asks for a development/bond length. Rock bolts and SDAs are modelled using nails module [2, 3] as it considers the complete length to be bonded. Properties of the above reinforcements are defined as shown in Table 3 [4, 5].

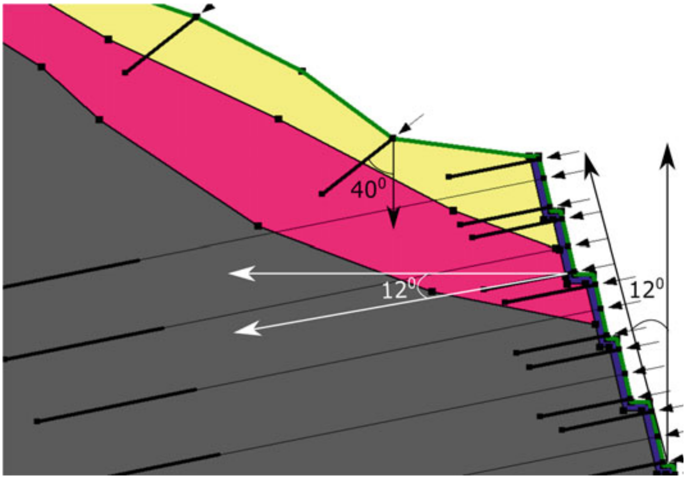

Fig. 2 Schematic diagram of a reinforced hill slope in Slope/W

A total of 18 slope sections is stabilised for 1 km long airport runway. AutoCAD cross-section of these slope sections was made available by a classified consulting firm. These sections are preprocessed and imported into Slope/W.

To incorporate the effect of the groundwater table, the ratio of pore water pressure to vertical pressure (R_u) is specified in Slope/W. R_u of 0.25 for the top two layers are used to assimilate the effect of groundwater table in the analysis.

For pseudo-static loading, K_h (horizontal coefficient for seismic acceleration) is taken as 0.15 as the site of construction comes under seismic zone 4 [6]. K_v is taken as 0.10, i.e. two-thirds of the K_h value.

The wall is inclined at 12° from the vertical, as shown in Fig. 2. Figure 2 also indicates that the cable anchors and rock bolts applied on the wall are perpendicular to the wall hence inclined at 12° from horizontal. The SDAs and cable anchors used on the ground slope are inclined at 40° from the vertical, as shown in Fig. 2 [7, 8].

The following features are incorporated to check global slope stability in the software:

1. Modelling of slope profiles for detailed stability analysis (Geometry and property assignment for slope layers and reinforcements).
2. Analysis under both static and pseudo-static conditions to achieve safe design (Incorporating K_h and K_v).
3. Modelling of groundwater table according to available borehole data from field testing (Incorporating R_u).
4. Optimising the quantity and quality of reinforcements to accomplish required minimum FoS.

4 Grid Convergence Study

For Slope/W analysis, grid and radius approach are chosen. In this approach, a grid is drawn, which represents the centre of cutting circle. The radius is then taken from the drawn radius domain. The iterations are then run for each cut to check the factor of safety. To select a grid and radius mesh, three combinations are tried keeping the same area of interest. For these grid-radius combinations, the minimum factor of safety and time of analysis are recorded. Grid of 30 by 30 and radius increments of 30 are chosen for which the analysis time and FoS are optimal.

5 Results

Consideration of site-specific conditions like high disturbances of the rock mass, heavy rainfall in the area, and the importance of the project plays a major role in determining the stability criterion of slopes. Table 4 describes the stability criterion for this particular case of slopes [9–11] (Table 5).

For a better perspective, four critical sections (sections 6, 11, 13 and 1) with corrections to stabilise the slope are given in Table 6 and Figs. 3, 4, and 5.

Section 1 was failing due to a steep slope which is out of the AAI boundary. The reinforcements were allowed till the AAI boundary was extended out till the end of the critical steep slope. This increased the number of SDAs drastically with densification at steep part (out-of-the-plane spacing is reduced to 1.5 m for SDAs). Six cable anchors were also added to provide stability to the steep slope. The initial and final configurations are shown in Fig. 3.

Section 9 was encountering a local failure at a slope transition and after some iterations with 8 m SDAs and 20 m cable anchors on the slope. The latter correction was applied for the stabilisation as shown in Fig. 4.

Table 4 FoS for different grid-radius configuration

Mesh size (grid, radius)	Computational time (sec)	Minimum FoS
15, 15	11.88	1.710
30, 30	70.28	1.734
50, 50	312.07	1.697

Table 5 Minimum FoS requirements for global stability analysis

Loading conditions	Minimum desired factor of safety
	Long term (Undrained behaviour)
Static loading	1.4
Seismic loading	1.05

Table 6 Initial and final configurations

	Section-9	Section-11	Section-13	Section-1
Initial configuration				
Rock bolts	14	18	24	8
Cable anchors	5 of 25 m 2 of 20 m Total = 7	7 of 25 m 2 of 20 m Total = 9	2 of 30 m 8 of 25 m 2 of 20 m Total = 12	2 of 25 m 2 of 20 m Total = 4
SDAs*	191	191	172	117
Final configuration				
Rock bolts	14	18	24	8
Cable anchors	2 of 20 m 5 of 25 m 3 of 20 m on slope Total = 10	7 of 25 m 2 of 20 m Total = 9	2 of 30 m 8 of 25 m 2 of 20 m Total = 12	2 of 25 m 2 of 20 m 6 of 20 m on slope Total = 10
SDAs*	138 of 6 m 44 of 4 m Total = 182	165 of 6 m	113 of 8 m 59 of 4 m Total = 172	165 of 8 m (spacing 1.5 m) 117 of 4 m Toal = 282

*By default all SDAs are of 4 m with 2.5 m out-of-the-plane spacing

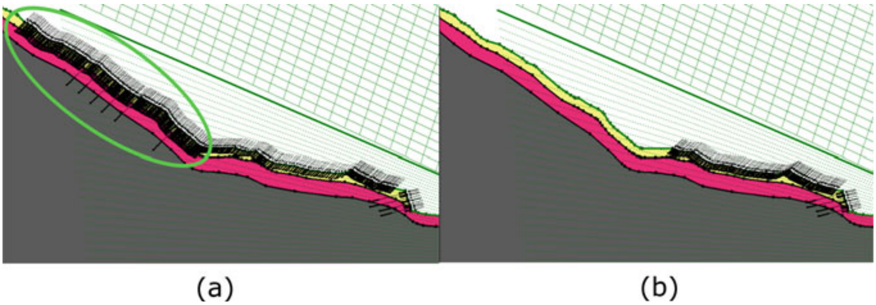

Fig. 3 Section 1 **a** After, **b** before after the required stabilisation

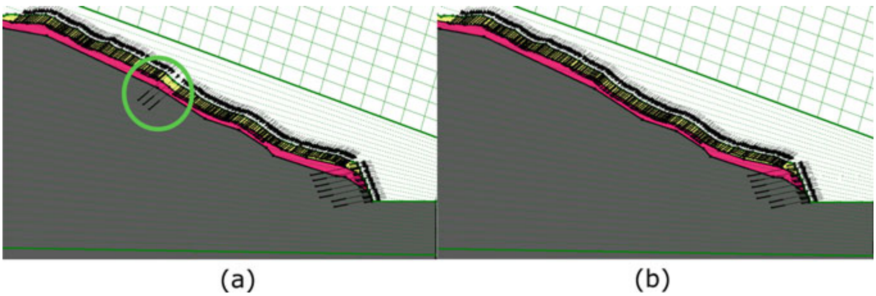

Fig. 4 Section 9 **a** After, **b** before after the required stabilisation

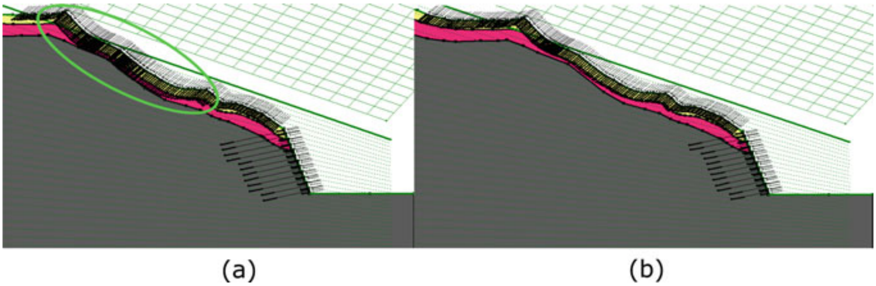

Fig. 5 Section 13 **a** After, **b** before after the required stabilisation

Table 7 Minimum static and pseudo-static FoS comparison

Section	Factor of safety (static)		Factor of safety (pseudo-static)	
	Initial	Final	Initial	Final
9	1.376	1.519	0.943	1.026
11	1.375	1.533	0.941	1.044
13	1.315	1.444	0.930	1.156
1	0.991	1.465	0.731	1.006

Top layer of the soil was the problematic zone for Section 13 as failure surface beginning from there was extending to cause a large failure. Length of SDAs in the steep section was increased to 8 m to keep the section safe as shown in Fig. 5.

The factor of safety of before and after the application of corrections is shown in Table 7 for sections mentioned above.

To summarise the analysis of all 18 sections, Sections 7, 8, 10, 14, 15, and 16 do not require any reinforcement change as they are good to go with initial configuration suggested by the contractor. In Section 4, the initial configuration consisted of cable anchors on overburden and only the spread of the SDAs were changed. Three SDAs were added in Section 2 to achieve desired stability in AAI boundary. In Section 11 and 17, the total number of SDAs was reduced in the initial configuration, but the length of SDAs in the problematic areas was increased. The number of SDAs and length of some SDAs had to be increased to 6 m in Section 3. Number of SDAs was increased in Section 5 to stabilise the slope region. In Section 13, the number of SDAs was kept the same but length of some SDAs was increased to 8 m keeping the rest 4 m. An economical design was determined for Section 18 where the length of SDAs in a major portion was reduced to 3 m increasing the remaining SDAs to 6 m. Although, the overall number of SDAs had to be increased but only the difficult portions were reinforced with 6 m SDAs.

In unstable sections like 1, 9, and 12, cable anchors were additionally introduced on the sloping regions to increase the gripping power along with changes in length and number of SDAs. For the steeper part of Section 1, the out-of-the plane nail spacing is reduced by 1 m to densify the reinforcements, and the length of SDAs were increased up to 8 m from 4 m in steep portions.

6 Conclusions

From the results of this study, some significant conclusions can be drawn about soil slope reinforcement in the lower Himalayan region. These conclusions are given below.

1. Local slope failures can be a big problem in weak soils. In this study, top soil is the weakest; hence, the reinforcements should penetrate the top soil and seated in the lower layer to bind it with the relatively stronger lower layer.
2. Shear reinforcement is a significant concern for steeper slopes. SDAs posses shear strength; hence, they can be used in steeper slopes as compared to cable anchors which are grouted for only 7 m. In critical cases, cable anchors should be used in conjunction with rock bolts and SDAs.
3. From the current study with weak top soil resting on a fractured rock layer, long fully grouted anchors (SDAs longer than the thickness of top soil layer) will be the best solution for slope stability. This conclusion can be generalised for slopes in the lower Himalayan region.

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Impact of Wetting–Drying Cycles on Swelling Behavior and Micro-structural Analysis of Stabilized High Plastic Clay

Khushbu Gandhi and Shruti Shukla

Abstract Clay minerals of expansive soil are susceptible to swelling and shrinkage due to moisture variation. Distress occurs to expansive clay soils due to drying and wetting cycle, which directly affect the lightweight structures. The effect of cyclic wetting–drying phenomena can be reduced by improving the soil. In this study, the high plastic clayey soil was treated with industrial waste granulated blast furnace slag and bagasse ash, and the influence of wetting and drying cycle on swelling characteristics of treated soil has been investigated in laboratory condition. Such investigation is needed to check the durability of stabilizer to modify the expansive soil. High plastic clay was stabilized with different proportion of BA and GGBS to get optimum mix. Both untreated and (0 and 28 days cured) treated expansive clay were analyzed for wetting–drying cycles. Variations in consistency limits of both natural and untreated clay were investigated. The micro-structural studies were also conducted by X-ray florescence and scanning electron microscopy (SEM). The finding indicates that these waste products reduce the gradual deformation of stabilized high plastic clayey soil subjected to drying and wetting cycles. The result of this research revealed that bagasse ash can be used in soil stabilizer as a pozzolanic material in combination with ground-granulated blast furnace slag to improve swell–shrink behavior.

Keywords Bagasse ash · Ground ground-granulated blast furnace slag · High plastic clay · Stabilization · Wetting drying cycles

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1 Introduction

Expansive clayey soil is susceptible to variation in a volume due to moisture variation. It is good for agriculture purposes but problematic for the lightweight structures such as pavements, runways, embankments, and low-rise residential buildings due to high swelling and shrinkage behavior on moisture variation. This behavior of this soil is due to the existence of an expanding lattice structure of mineral montmorillonite. Montmorillonite expands when it comes in contact with water. Soil available in arid and semiarid regions worldwide, rich in these minerals, where climate characterized by alternating rainfall and drought periods, is subjected to the wet and dry cycle, causing instability and distress. Due to swelling of expansive clays, cracks and deformation propagate in roads, pavements, building foundations, irrigation structure, reservoir linings, and pipe lines [1]. It has been reported by Al-Rawas that more than 15 billion dollars cost damage occur on the structure rested on such soil in the United States [2]. A number of researchers, Day [3] and Basma [4] performed full swell–full shrink cyclic wetting–drying tests on untreated clay, which shows an increase in swell behavior and constant after 4–5 cycles. This behavior is explained as destruction of the flocculated clay structure and more swelling and permeable soil formation having a dispersed structure [3, 4]. Kalkan discussed the impact of wetting–drying cycle (full swell–partial shrink) on silica fume stabilized expansive clay. Improved the durability of treated sample against the cycles of wetting–drying. Investigated the effect of cyclic swell–shrink (Partial shrinkage method) on swelling behavior of polymer-stabilized expansive clays [5]. Fateme Yazdandou discussed that SEM analysis shows particles are moving closer together, forming aggregates and reducing the swelling potential [6]. Rao et al., investigated the wet-dry cycles effect (full swell–full shrink) on the consistency limits of the lime-treated clay. Swell potential of natural soils decreases after the first cycle and reached to the equilibrium after fourth cycle. Also swell potentials of soil treated with lime increased due to disturbance in pozzolanic reaction and partial breakdown of inter-particle cementation [7]. Rao A et.al studied the wetting–drying (full swell–full shrink) effect on the swelling behavior of expansive soil treated with fly ash cushion which were treated with cement and lime. Cement-treated fly ash cushions showed decrease in heave when compared with lime [7].

Many researchers studied and well documented the wetting and drying cycles' impact and expansive behavior of natural expansive clay and traditional chemicals such as lime and cement stabilized clay. Studies related to influence of wetting and drying cycle on the swelling behavior of waste material stabilized soils and after 28 days cured period are very limited. Generally, traditional stabilizers relay on capacity of ion exchange and pozzolanic reactions of soil. Pozzolanic reactions happens when aluminous and siliceous materials form cementitious compounds after reaction with calcium hydroxide. Industrial waste, bagasse ash, and blast furnace slag can be used to modify the clay because of siliceous and calcareous source. Most of sugarcane bagasse is used as a source of fuel in the same sector, whereas GGBS is primarily used as partial cement replacement for concrete. There is also excellent potential for using these industrial by products as stabilizing agents. James et al. highlighted that in the 2011–12, Sugarcane production reached to 361.04 million tons in India and became the biggest cane producer in the world [8]. Currently, India is the world's second-biggest producer of sugarcane, produces 341.2 million metric tons of sugarcane. This ash acts as pozzolana which is rich in oxides of alumina and silica. Kiran et al., Chittaranjan and Keerthi and Kharade et al. discussed the bagasse ash as a stabilizer for expansive clay and revealed that it can be used to improve the swelling behavior of expansive soil [9–11]. Wani et al. [12] discussed the mineralogical characterization of bagasse ash, which stated that bagasse ash is composed of silica mineral such as quartz. Other minerals are also reported in varying percent depending on the source of ash [12]. ASTM C618 suggest that for any natural pozzolana; alumina, iron oxide, and silica should be at least 70% and also SO3 content should be less than 4% [13]. It has been reported in almost all the studies related to bagasse ash. The chemical properties of bagasse ash and GGBS vary widely. GGBS is high in calcium oxide, whereas bagasse ash is rich in silica and alumina but low in calcium oxide. When used as a stablelizing agent, the combination of these two products may be more useful instead individual. One will provide enough calcium or silica to react as pozzolana. Several researchers conducted studies on GGBS and/or bagasse ash separately with different stabilizer [14–16]. However, any work has not released to date on the impact of the wetting–drying process on the combine activation of bagasse ash and GGBS as stabilizers for expansive soils. The aim of this research is to explore the long-term behavior on swell–shrink characteristics of stabilized high plastic expansive clay.

2 Materials and Methods

2.1 Material

Clayey Soil

The soil samples used in the research have been taken from a Surat, Gujarat, India. Soil available in local region is low-to-high expansive black clay. The disturbed clayey soil sample was extracted 1 m deep from the ground level. Laboratory tests to classify the soil were performed. Grain size analysis shows the sample content 46% clay, 33% silt, and 21% sand. Sample has been classified as high plastic clay according to the A-line chart. The different geotechnical and chemical characteristics of the soil sample are summarized in Tables 1 and 2, respectively. Clay sample's specific gravity is evaluated to be 2.6. X-ray diffraction (XRD) analysis and scanning electron microscopy (SEM) images of the clay soil sample are discussed in further section.

Baggase Ash

The sugarcane bagasse ash was taken from the boiler of sugar factory, Bardoli, which is Asia's largest sugar factory. Bagasse ash is an industrial waste material,

Table 1 XRF results of CH, BA, and GGBS

Component/metal oxides	Concentration (%)		
	CH	BA	GGBS
SiO_2	34.71	51.361	26.43
Al_2O_3	5.265	2.005	9.36
Fe_2O_3	44.384	21.267	2.51
CaO	4.92	3.32	41.02
K_2O	1.244	4.074	1
M_gO	1.439	1.127	6.5
Na_2O	0.354	0.319	0.3
SO_3	1.533	6.594	–
TiO_2	3.028	1.791	2.1
Cr_2O_3	0.105	1.207	–
MnO	0.623	0.445	–

Table 2 Geotechnical characteristics of clay and optimum mix sample

Soil property	Clay	Optimum mix
Liquid limit (%)	64.05	46.55
Plastic limit (%)	27.12	29.61
Plasticity index (%)	42	19.44
Shrinkage limit (%)	22.42	46.32
Optimum moisture content (%)	22.7	29.23
Maximum dry density (kN/m^2)	1.58	1.431
Swelling pressure (kN/m^2)	54.917	19.74
Free swell index (%)	66	27

generally dumped in farms. In this study, collected ash which was oven dried and passed through 425 μm sieve was used. The measured specific gravity of BA was 2.32. These bagasse ashes (BA) have been chemically studied as given in Table 1, to evaluate the possibility of their use as stabilizing the material (Fig. 1).

Ground-Franulated Blast Furnace Slag

The GGBS was taken from the locally available market of Surat, Gujarat, India. It forms from high-quality clinker, slag, and gypsum after grinding. Its cementitious waste consists of oxides of calcium, silica, and aluminum. GGBS represents the elevated quantity of calcium oxides as a binder material. The specific gravity of GGBS used in this research was measured to be 2.83 (Fig. 2).

Chemical and Mineralogical Properties

The X-ray fluorescence analysis was carried out to obtain the concentration of exchangeable cations which reflects its effect on swelling behavior. The chemical compositions of untreated clay and bagasse ash have been determined using

Fig. 1 Bagasse ash sample

Fig. 2 Powder form of GGBS

wavelength dispersive X-ray fluorescence method as given in Table 1. Chemical constituent of ground-granulated blast furnace slag has been obtained from the cement manufacturer.

Mineralogical analysis was performed using X-ray diffraction (XRD). The X were obtained with the help of X-ray diffractometer advance −8 instrument using Cu Kα radiation. As a result of XRD analysis, Smectite identified in the untreated clay sample which shows existence of montmorillonite mineral. Bagasse ash contains quarts. XRD analysis shows small amount of impurities also due to presence of silt and sand content. Untreated clay shows high liquid and plastic limit due to presence of smectite, which has high water absorption capacity. The analysis has been performed with the help of Joint Committee on Powder Diffraction Standards after comparing with the results of samples.

2.2 Methodology: Preparation of Sample

The collected sample of clay was dried for 24 h at 70 °C before grinding. To get optimum mix for durability test, investigation has been carried out using bagasse ash and blast furnace slag in different percentage by dry weight. All three materials are mixed manually in dry state in different proportion using required water. Mixing was done till uniform mixture obtained. Mix prepared for clay soil with GGBS content varying from 2.5% to 7.5% and bagasse ash varying for 5 to 20% at 2.5% and 5% intervals, respectively, to obtain optimum mix sample [17]. Liquid limit and plastic limit (as per IS 2720: PART 5: 1985), shrinkage limit (as per IS 2720: PART 6: 1972), free swell index (as per IS 2720: Part 40), and swelling pressure test (as per IS 2720: Part 41) have been carried out for untreated and treated samples per Indian standard. Optimum water content of mix has been found using proctor test (as per IS 2720: PART 7: 1980) to make remolded sample for swelling pressure analysis as per Indian standard. The optimum mix was selected for cyclic wetting–drying study to check the durability of stabilized soil.

Consistency Limit Tests

After identification to know the degree of expansive soil; plastic limit and liquid limit test is performed on untreated and different mix content treated soil as per Indian standard code. Special attention was needed as consistency should decrease while progress of experiment. Shrinkage factors were found out using Part VI of the IS 2720. All sample were prepared by adding water equal to or greater than liquid limit of sample and kept for 24 h in humidity chamber to mitigate water evaporation. After filing of shrinkage dish with gentle tapping, sample was weighted and kept for oven dry to find its shrinkage limit.

Compaction Tests

In order to make a remolded sample to evaluate swelling pressure and wetting–drying cycles; optimum moisture content of untreated and treated samples was found out using light compaction test as per Indian Standard. 5 kg oven-dried sample passed through 4.75 mm sieve has been used with different water content on interval of 2–4% which were used depending upon sample. The compaction curves have been plotted to get maximum dry density and optimum water content for every representative samples.

Swelling Pressure Tests

Swelling pressure test performed on remolded samples of different mix using optimum water content and maximum dry density. Constant volume method has been used as per Indian standard code to determine swelling pressure of 60 mm diameter and 20 mm height samples of all mix. Samples were confined from all direction to measure swelling pressure. Free swell index was carried out using 10 gm oven-dried sample passing through 425 μ sieve. Samples were put to swell freely in cylinder filled using distilled water and kerosene for 24 h.

Preparation of Specimens for Durability Study: Wetting–Drying Cycles Tests
This test has been carried out to analyze the impact of stabilizer on the swell behavior of soil subjected to alternate wetting and drying cycles which reflect the durability of stabilized soil on seasonal change. Wetting–drying cycle has been applied to untreated clay sample, 0 days cured optimum mix sample and 28 days cured optimum mix sample. During the test, the samples have been submerged at room temperature in potable water to allow wetting for five hours; wetted specimens were dried at 70 °C in the thermostatically controlled oven for 42 h. In between wetting and drying period, sample is allowed to dry at room temperature for 12 to 18 h. This constitutes one cycle of wetting and drying as per IS 4332: Part IV. At the end of every cycle, swelling pressure has been measured. Swelling pressure test were carried out on swelling pressure apparatus using control volume method. This procedure was repeated till six cycles.

Scanning Electron Microscopy (SEM)
SEM study reported till date shows effect of wetting–drying cycle on treated uncured samples, and no study is reported for 28 days cured sample SEM analysis. To check the effect of wetting drying cycle on 28 days cured optimum mix sample, specimen was given for scanning electron microscopy. Samples for this test were prepared for the following conditions. (a) 28 days cured remolded sample with zero wetting–drying cycle, (b) 28 days cured remolded sample after two wetting–drying cycles, (c) 28 days cured remolded sample after six wetting–drying cycles.

The samples were kept for dry at room temperature. Then, samples were taken for scanning electron microscopy analysis at the Bombay textile research association, Mumbai. The samples were analyzed using instrument named JEOL JSM IT 200. Magnification from 50 to 2000× was used to analyze the sample.

3 Results and Discussion

The experimental work was divided into three phases. In the first phase, the geotechnical and chemical characteristics of the soil, BA and GGBS were identified. In the second phase, the optimum content was determined to modify the soil sample, and durability tests were carried out on untreated and optimum mix sample. In third part, scanning electron microscopy analysis is performed and analyzed. In this paper durability study, wetting–drying analysis and mineralogical study for untreated and optimum sample were presented.

Different tests such as Atterberg limits, swelling pressure test, compaction, CBR, and UCS were performed on different proportion of CH, BA (5–20%) and GGBS (2.5, 5, 7.5%) mix. Based on swelling and strength properties, 82.5% CH, 10% BA, and 7.5% GGBS mix was considered as optimum mix for the soil of study area. Good bonding and pozzolanic reactions occur between BA, GGBS and CH soil for this proportion which increases enough compression strength and decreasing swelling pressure, adding more GGBS may cause over-safe design for lightweight structure.

Effect on Atterberg Limits

Bagasse ash content was found to be directly associated with the reduction of liquid limit and plasticity index as shown in Fig. 3. Plasticity is good swelling potential measure, reduction in plasticity index represent the reduced swelling potential. Same as the shrinkage limit of clay and bagasse ash mixtures increasing with binder content as shown in Fig. 4. Increase in shrinkage may be due to the development of interaction between bagasse ash–GGBS mix surface and soil matrix. It is known that significant modification in the index characteristics of soil occur with the addition of non-swelling material such as Bagasse ash, which reduces the thickness of diffuse layer and so that clay particle flocculated [18]. The consequent reduction in the affinity toward water and the particle surface area resulted in reduction in the plasticity. High plastic clay is converted into low-to-medium plastic for optimum mix samples.

Effect on Compaction Characteristics

Figures 5 and 6 show the impact of the mix on the optimum water content (OMC) and the MDD, respectively. Generally, with increase of mix content, OMC decreases and MDD increases. But in this study, different trend has been observed. With increasing bagasse ash content, OMC increased while MDD decreased. Same trend has been reported by some researchers also for other type of ashes like rise husk ash, fly ash [19–21]. The reduction in dry density explained as varying specific gravities of soil and stabilizer [22]. Light compaction test results indicate increase in MDD with increasing GGBS content and decrease with BA content and OMC increase with BA content and GGBS content.

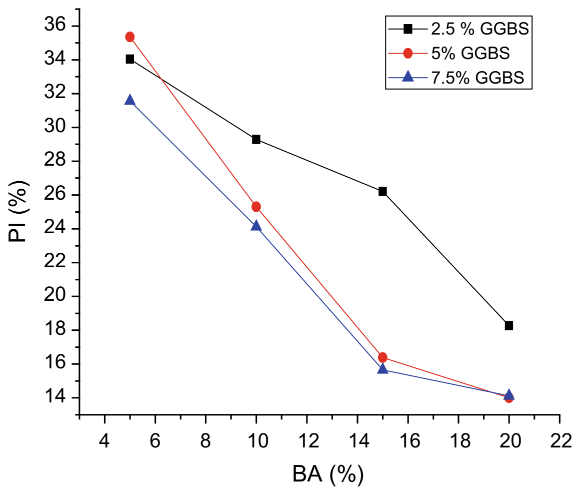


Fig. 3 PI with varying percent of BA for GGBS stabilized clay

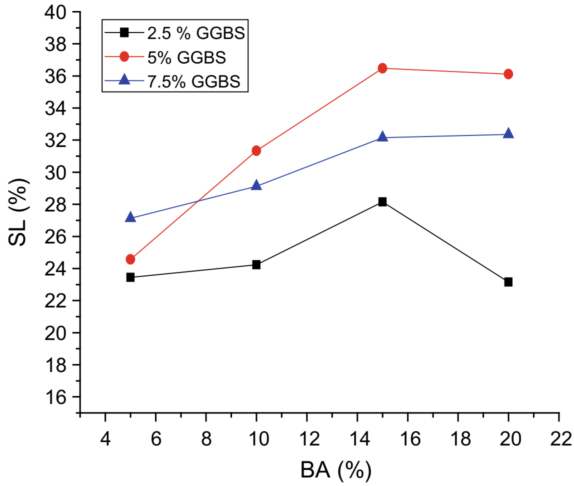


Fig. 4 SL with varying percent of BA for GGBS stabilized clay

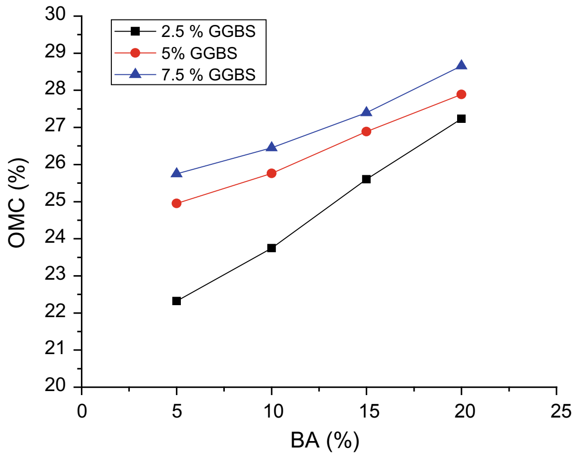


Fig. 5 OMC with varying percent of BA for GGBS stabilized clay

Effect on Swelling Characteristics

Swelling characteristics in this study is defined by swelling pressure and differential free swell index. Reduction in swelling pressure is observed as shown in Fig. 7, with the increase of bagasse ash.

Decreasing rate of pressure is less in case of addition of GGBS when compared with bagasse ash. Free swell index is also decreasing as increase percent of bagasse ash as per Fig. 8. Swelling pressure and free swell index are reduced on addition of BA due to increase of exchange of ions and flocculation. Also, non-expansive material bagasse ash replaces the expansive clay which reduces the swelling pressure and free swell index. 10–20% bagasse ash is recommended for soil available in study area, after that it became constant with the combination of 5–7.5% GGBS.

Fig. 6 MDD with varying percent of BA for GGBS stabilized clay

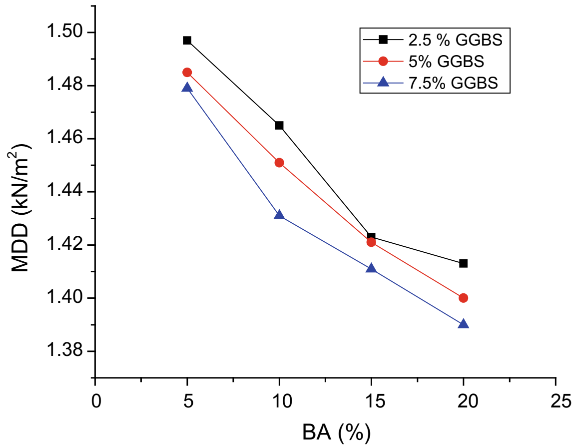
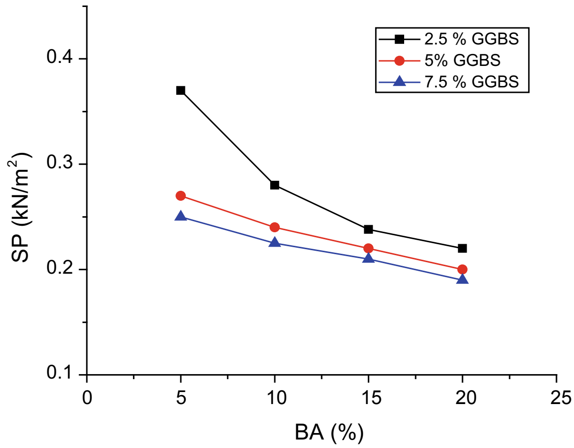


Fig. 7 SP with varying percent of BA for GGBS stabilized clay



Effect of Wetting and Drying Cycles

Wetting–drying cycle has been performed on untreated clay sample and (0 days cured and 28 days cured) optimum mix sample. Figure 9 shows the swelling pressure decreases for untreated and 0 days cured optimum mix sample after every wetting drying cycle. It has been found that with increasing wetting–drying sample swelling pressure is decreasing for both the sample and became constant after fourth cycle for untreated sample. 0 day cured optimum mix sample shows continuous decreasing rate with each wetting–drying cycle. It has been observed that 28 days cured sample converted into non-swelling due to flocculation and pozzolanic bonding between clay and BA particles with GGBS. Wetting–drying results show zero swell pressure when measured by constant volume method using swelling pressure apparatus. Six wetting–drying cycle has been applied on the same sample which results zero swelling pressure for each cycle.

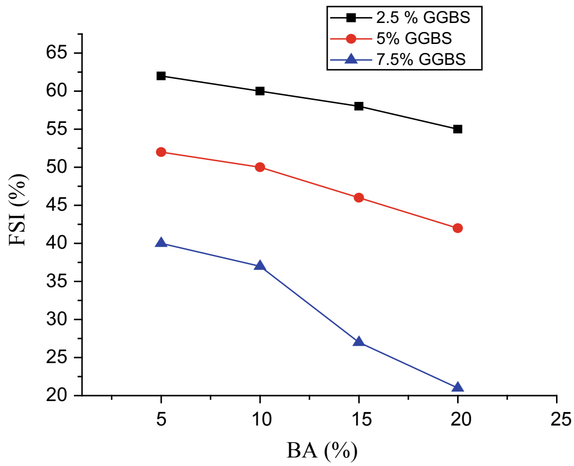


Fig. 8 FSI with varying percent of BA for GGBS stabilized clay

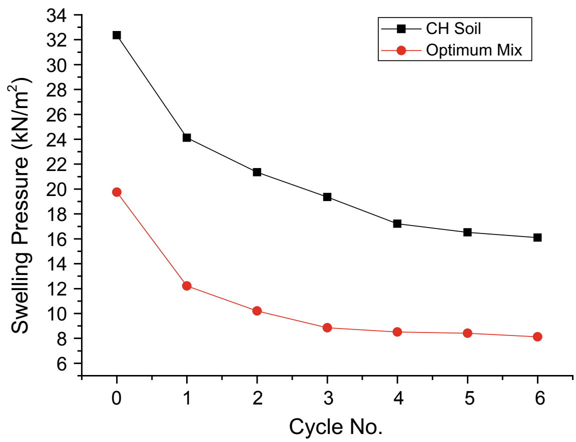


Fig. 9 Swelling pressure with no. of wetting drying cycles

Image Analysis

Scanning electron microscopy images of natural high plastic clay and 28 days cured optimum mix sample after zeroth, second, and sixth wetting–drying cycles have been presented in figure below.

Figure 10 shows that large size pores and unbounded clay particles increase the total void spaces, and also pores are not connected with each other properly. Figure 11 shows the decrease in pore spaces and bonding due to aggregation and flocculation of bagasse ash and GGBS content with the soil after 28 days curing period. These microscopic images and experimental outcomes show that the possible reaction occurs between BA-GGBS mix with clay particles. It has been seen that after second wetting–drying cycle (Fig. 12), minimum voids are observed which represent mix sample not affected by wetting drying cycle. Stabilizer covers the surrounding clay soil particles and fill the voids of the stabilized clayey soil samples. Figure 13

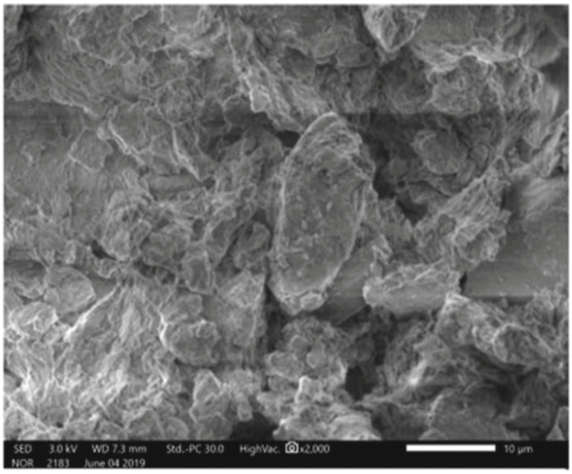


Fig. 10 SEM of CH soil

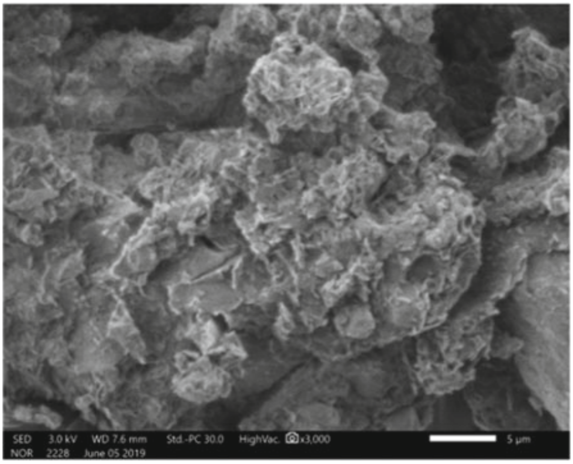


Fig. 11 28 days cured mix sample (0 cycle)

shows the microscopy images after sixth sample, flocculated structure observed, and surface area of particle decreases which reduce the water affinity of sample which may be responsible for the improvement in swelling pressure. A detailed study shows maximum flocculation structure observed on the surfaces of the soil particle.

4 Conclusion

The aim of this study is to evaluate the impact of waste material bagasse ash-GGBS based mix on the swell–shrink characteristics of high plastic expansive clay and to find the optimum mix to stabilize it. The study attempted to analyze the impact of

Fig. 12 28 days cured sample after second cycle

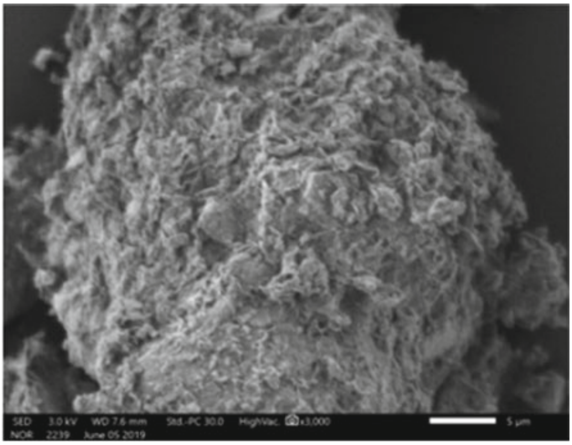
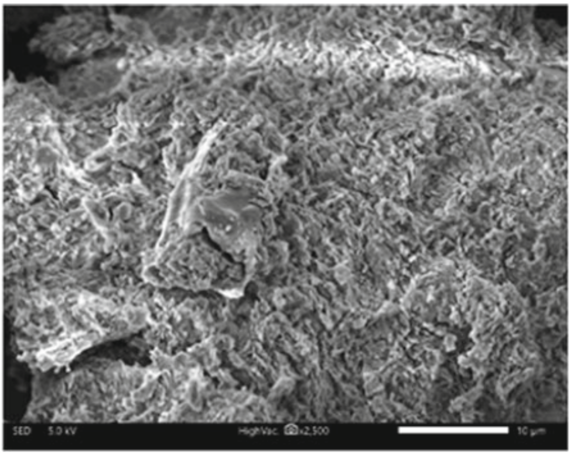


Fig. 13 28 days cured sample after sixth cycle



wet and dry cycles on swelling behavior of natural high plastic clay and optimum mix sample. The following conclusions are drawn from the outcomes of the different experiments performed.

1. Use of bagasse ash and GGBS mix to the soil reduced liquid limit and plasticity index while increasing the shrinkage limit. It is found that the addition of binder causes flocculation of clay particles and increases the number of coarser particles which help in reducing the plasticity.
2. Reduced MDD and increased OMC was observed on addition of mix. It can be explained that due to lower specific gravity of BA it caused reduction in dry density and flocculation and agglomeration of particles occupying larger spaces leading to corresponding decrease in dry density. This is due to predominant effects of reduced clay content and increased frictional resisting, respectively.

3. Swelling pressure and free swell index were reduced on addition of BA due to replacement of expansive clay with non-expansive material. 10–20% BA recommended with the combination of 5–7.5% of GGBS. It has been observed that bagasse ash content directly affects the swell–shrink behavior of sample, and various GGBS improve the strength characteristics of clay. So combined mix of bagasse ash and GGBS improved all the characteristics of high plastic clay.
4. From the experimental results, 82.5% CH, 10% BA, and 7.5% GGBA mix has been chosen as an optimum mix for the soil of study area, and wetting–drying cycle is given to untreated and (0 and 28 days cured) optimum mix sample to check its durability against seasonal variation. Untreated sample shows reduction in swelling pressure for each wetting drying cycle and reached to equilibrium after fourth cycle, whereas 0 day cured optimum mix sample shows reduction pressure for all cycle. Results indicate non-swelling behavior for 28 days cured optimum mix sample. Scanning electron microscopy images of 28 days cured sample after zeroth, second, and sixth wetting drying cycles show agglomeration and flocculation of structure. It has been seen that pore space and particle surface area reduced which reflect the less water affinity of particle.

The findings of the test showed that industrial waste bagasse ash is able to protect low to high plastic expansive clay from the adverse effect of swelling. Furthermore, bagasse ash reduced the swell pressure of expansive clay after replacing clay minerals. Combine effect of BA-GGBA mix on high plastic clay proved to be a good pozzolanic material against seasonal variation and gives good durability. It is very essential to use it as a step toward sustainable development in the current world, where a range of industrial waste is accessible in large amount.

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Strength Characteristics of Subgrade Soil Stabilized with Plastic Bag Strips

A. Vismaya, Monica Simon, P. K. Jayasree, and Leema Peter

Abstract Plastic is perhaps the most dangerous scrap and pollution that it causes and has become a villain to the society. For the betterment of our planet, we must use and recycle the plastic in a fruitful way. During recycling process due to melting of plastic, toxic gases are released. One of the effective ways to manage plastic waste is using it for soil stabilization. This study investigated the possibility of utilizing plastic bag waste for the reinforcement of soils. The effects of variation in thickness and aspect ratio on strength characteristics were studied in this work. The various thickness ranges used were 15, 30, and 45 μm, and waste plastic carry bag strips were added at 0.1, 0.2, and 0.3% concentration. The unconfined compressive strength test was conducted, and the results obtained favorably suggest that up to an optimum value, shear strength increases with increase in plastic content. An improved UCS value was achieved at 0.2% plastic content having an aspect ratio of 2.5. Results of experimental studies on soil reinforced with plastic waste showed that plastic can be effectively used as stabilizing material so as to solve environmental issues.

Keywords Plastic waste · Stabilization · Shear strength · Thickness

1 Introduction

The rapid development of urban areas and the increase in construction activities have resulted in a scarcity of land with favorable soil conditions, necessitating the

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use of locally available weak soils for construction activities through stabilization techniques. Soil stabilization can be explained as the improvement in soil properties by chemical, physical, or biological means in order to enhance the engineering quality of the natural soil. This process is accomplished using a wide variety of additives, including lime, cement, ground-granulated blast furnace slag (GGBS), fly ash (FA), and bottom ash (BA), and they are highly expensive. However, using lime and cement raise environmental concerns and are not preferred nowadays. Due to the increasing cost and the harmful effects produced by the additives, alternative methods for soil stabilization are to be found.

Plastic waste is a major issue in urban and rural areas in India. Littering of plastic waste and its byproducts may lead to major health concerns for all living beings. These days, plastic becomes essential part of our lives; however, the data on plastic pollution is shocking. It is produced on a massive scale worldwide, and its production crosses 150 million tons per year globally. For the environmentally conscious citizens and organizations, disposing off the non-biodegradable used plastic bottles has become a major concern. Approximately, 600 billion bottles are discarded every year all around the world, and only 47% are collected [1].

Therefore, the garbage products should be disposed properly for the better future. The efficient method to handle such wastes is to utilize them for engineering applications. Nowadays, recycled HDPE are used for fence line posts of guard rail posts for highways and lightweight reinforcing inclusions in concrete. The plastic waste can be used as a reinforcing material in weak soils. This improves its strength characteristics, and it is a way of recycling these materials in an efficient, environment-friendly, and cost-effective manner [2]. In this study, effect of plastic waste covers on the strength characteristics of subgrade soil is investigated. Prediction of pavement performance becomes difficult if unconventional materials are used as a part of pavement structure [3]. Therefore, in the present study, the strength characteristics of subgrade soil stabilized with plastic bag strips was studied. The unconfined compressive strength (UCS) tests were carried out by varying the percentage and thickness of the plastic strips added. The results obtained clearly show that plastic cover strips can be used as an effective reinforcing material.

2 Investigations on Soil Stabilized with Plastic Waste

Recycling of plastics is a promising alternative for plastic waste management. Recently, many research studies have been made on the effective reuse of plastic waste in civil engineering constructions. Plastic waste was mixed with cement [4] to produce sturdy and flexible concrete slabs. In India, now, it has become a rule for all road manufacturers to use plastic waste, along with bituminous mixes, for road construction. A new tensile force resisting material called reclaimed high-density polyethylene (HDPE) was introduced and reinforced to locally available sandy soil to enhance the engineering property of subgrade soil [5].

Research studies were conducted to check the alternative of stabilizing soils using waste plastics in the form of bottles and bags. Experiments were conducted on clayey soil to find out the consolidation characteristics of soil stabilized with plastic waste and found that the plastic waste stabilized specimen exhibited a lower initial void ratio [6]. Waste plastic strips of appropriate size, and proportions were added to locally available sand which results in increase in both the CBR and secant modulus. It may be due to the increased friction between reinforcing material and soil [7].

Recently, industrialized and developing countries are greatly fascinated to use industrial waste in road construction, and it is based on technical, economical, and ecological considerations. Absence of prevalent materials and improvement of the environment makes it compulsory to search for replacement, comprising that of industrial wastes. Industrial wastes (e.g., fly ash, slag, and mine tailing) have combined with lime and cement to enhance the geotechnical properties of subgrade soil. Polypropylene was added in the form of fibers to silty soil stabilized with lime and rice husk ash [8]. The addition of fibers resulted in a decrement of the friction angle while the cohesion of the mixture boosted initially and then dropped with addition of fiber content, and the largest value was obtained at 0.4% fiber content.

The use of waste polyethylene material for soil stabilization can be considered as an eco-friendly method for soil stabilization [9]. To understand more about the behavior of HDPE plastic strips, perforated HDPE strips were used to reinforce sandy soils, and it was found that the reinforced soil exhibits the maximum angle of internal friction at 0.1% strip content only with width of 6 mm and perforation diameter of 2 mm [10]. It was also seen that the longer and wider strips resulted in strength deduction.

Production of polyethylene grains as a stabilizing soil material has a lower carbon footprint than cement or other hydraulic binders. As a sustainable solution to shallow slope failures, fiber-reinforced recycled plastic pins (RPP) were exerted into the slope face, which gave more resistance along the slip surface, adding the factor of safety against shallow slope failure [11].

3 Experimental Studies

3.1 Materials Used

Waste plastic covers used in the study were collected from the institution premises. The thickness ranges used were 15, 30, and 45 μm. The properties of plastic [12] are given in Table 1. The soil for stabilization was collected from a road construction site at Mangalapuram, Trivandrum, which was found to be clayey in nature upon visual inspection. The soil was classified as MH [13]. The soil properties are tabulated in Table 2.

Table 1 Properties of plastic waste

Thickness (μm)	Tensile stiffness (kN/m)
15	0.5
30	0.9
45	1.2

Table 2 Properties of soil

Property	Value
Specific gravity	2.56
Gravel (%)	1
Sand (%)	40
Silt (%)	32
Clay (%)	27
Liquid limit (%)	53
Plastic limit (%)	37
Plasticity index (%)	16
Shrinkage limit (%)	22
MDD (g/cc)	1.6
OMC (%)	20.5
UCS (kN/m^2)	49
CBR (%)	3

3.2 Methodology

The plastic covers were cut into strips of size 12 mm × 30 mm (Fig. 1, having an aspect ratio of 2.5) using scissors and measuring ruler. UCS tests were conducted on plain soil and on soil reinforced with plastic strips with varying percentages of 0.1, 0.2, and 0.3. The effects of thickness of plastic strips on the strength characteristics of reinforced soil were also studied.

4 Results and Discussion

4.1 Effect of Thickness of Plastic Strips on UCS of Soil

The stress–strain relationships from UCS tests for soil reinforced with plastic strips of varying thicknesses 15 μm, 30 μm, and 45 μm are presented in Figs. 2, 3 and 4, respectively. The results of unreinforced and reinforced specimens are included for the purpose of comparison. The unreinforced specimens exhibited brittle failure, whereas the reinforced specimens exhibited ductile behavior. Generally, increased

Fig. 1 Plastic strips

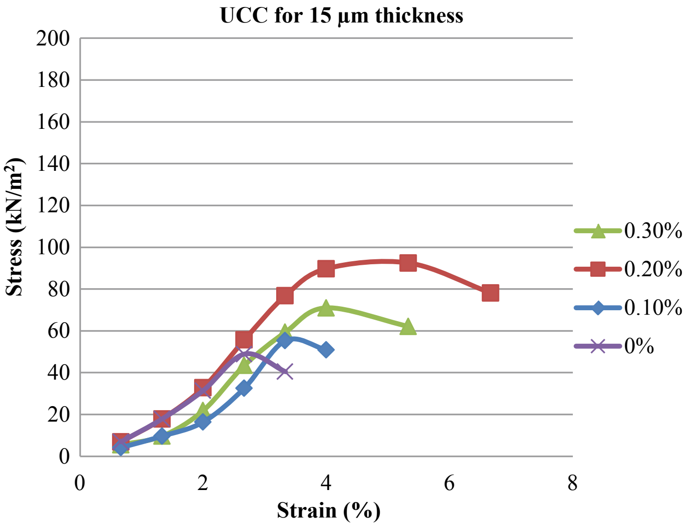

Fig. 2 UCS results for soil reinforced with plastic strips of thickness 15 µm

strain to failure of the fiber-reinforced specimens resulted in improved toughness of the specimens. Toughness is a measure of a specimen's ability to absorb energy during fracture.

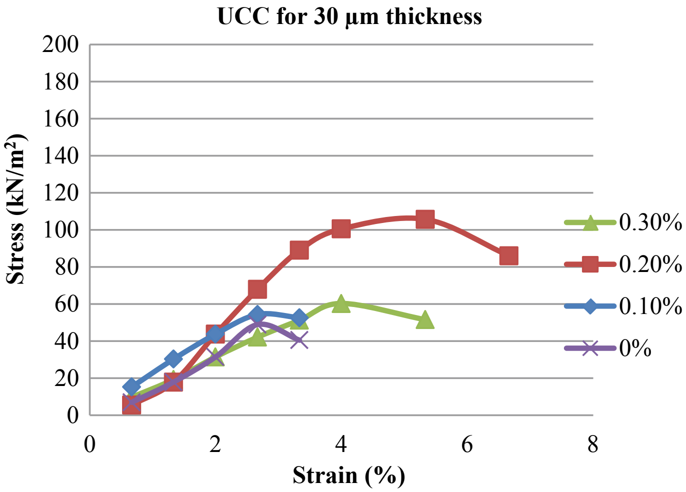

Fig. 3 UCS results for soil reinforced with plastic strips of thickness 30 µm

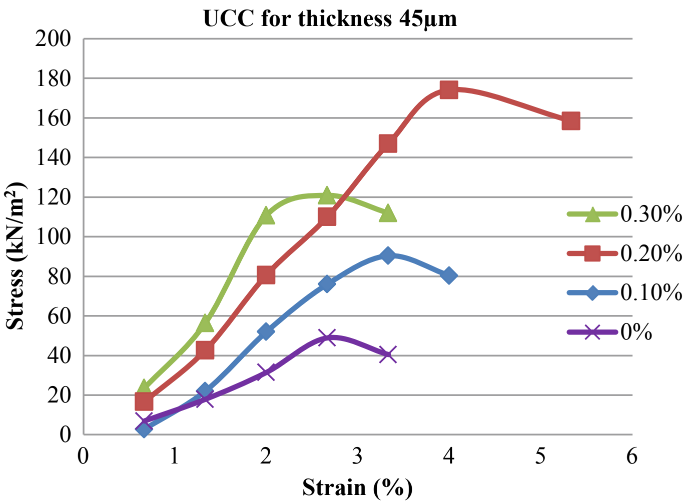

Fig. 4 UCS results for soil reinforced with plastic strips of thickness 45 µm

For 0.2% addition of plastic strips with thickness 15 µm, 30 µm, and 45 µm maximum value of UCS obtained was 92 kN/m^2, 106 kN/m^2, and 174 kN/m^2, respectively (Figs. 2, 3 and 4). The UCS value increases with the thickness of plastic strips. This is because the tensile stiffness of plastic increases with its thickness which ultimately affects the strength of soil.

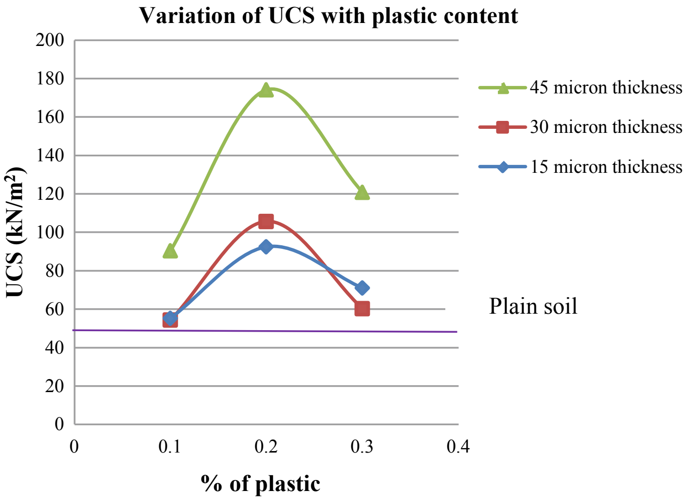

Fig. 5 Variation of UCS with plastic content

4.2 *Effect of Plastic Content on Strength of Soil*

The variation of UCS with the plastic content is shown in Fig. 5. The strength increases up to 0.2% of plastic content for all the thickness ranges and then decreases. This may be due to due to increase in total contact area between plastic strips and soil particles. The increase in plastic content consequently increased the friction between the soil particles which contributes to increasing resistance to the forces applied. Beyond 0.2% addition of plastic, UCS value decreased due to increased interaction between the plastic strips due to more overlapping of plastic, and it results in reduced soil plastic interaction.

The maximum compressive strength was obtained for soil reinforced with plastic strips having a thickness of 45 μm. Hence, there is an improvement in strength with the addition of plastic strips when compared to that of unreinforced soil. Similarly, the variation of strain at failure with the plastic content was shown in Fig. 6. The maximum value of strain was obtained at 0.2% plastic content which is higher than that of the unreinforced soil. Thus, the inclusion of plastic strips reduces the brittleness behavior of soil.

5 Conclusions

- The effect of plastic waste in improving soil properties mainly depends on strip size, plastic content, and the type. The plastic added in the form of strips is more beneficial in improving the strength characteristics.

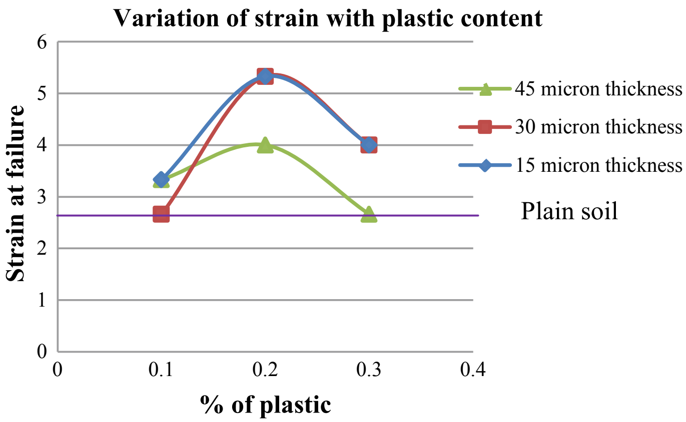

Fig. 6 Variation of strain at failure with plastic content

- The behavior of soil when reinforced with different concentrations of plastic showed almost same trend. There is a significant improvement in strength characteristics at an optimum percentage depending upon the type of soil.
- The maximum value of UCS is obtained for soil reinforced with 0.2% plastic content having an aspect ratio of 2.5. Soil stabilized with plastic strips of thickness 45 μm is having maximum compressive strength.

The tests are done with only one aspect ratio. Further, tests are needed to find the variation in aspect ratio on the strength characteristics. When the types of plastic strips are varied, i.e., PET, HDPE, or combinations of them are used, then the quantum of improvement of different soil parameters would be different. Further, tests are needed to exactly quantify their effects on soil improvement.

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Effect of Kaolinite Clay and Different Sand Gradation Mixture on Compaction Parameters

Nafisa D. Shaikh

Abstract In nature, various types of soils are distributed in such a way that they are found together. It is very much difficult to find clay, sand, silt, gravel in a pure condition. Also, these soils (clay, sand, silt, gravel) have different geotechnical properties. Although various researchers have focused on effect of clay particles on various geotechnical properties of sand–clay mixture. But, sand is available in different size and gradations. So, the effect of sand gradation and various particle size on geotechnical properties of sand–clay mixture is still not clear or very less information available about that. In this study, effort have been made to understand the effect of clay particles (kaolinite) on compaction parameters (OMC, MDD) of various sizes and gradation of sand. Experimental work involves the preparation of six samples of sand (i.e., three poorly or uniformly graded and three well graded) from procured materials. Poorly graded samples were obtained by following sand size: coarse sand (>2 mm), medium sand (>0.425 mm and <2 mm), fine sand (<0.425 mm). For preparation of well-graded samples, efforts had been made by combining two or three types of sands, i.e., coarse, medium, and fine (C + M, M + F, C + M + F). Kaolinite clay was used as cohesive fine fraction. Different amounts of kaolin clay were added in each of the sample, i.e., 0, 5, 10, and 15%. Preliminary tests were performed on all the three well graded, three poorly graded as well as on kaolinite clay in pure condition for finding out the physical properties of the soils. (i.e., Atterberg's limits, grain size distribution, specific gravity). Relative density test was performed for finding out the density of all the sand samples at 0% kaolin content ant 70% relative value was adopted. For all other sand–kaolin mixture (5, 10, 15%) standard proctor tests were carried out. Results indicated that among all the samples (coarse + medium + fine + 15% kaolin) exhibits greater maximum dry density (MDD) and lesser optimum moisture content (OMC) while (fine + 5%kaolinite) has lesser maximum dry density and greater optimum moisture content. So, from the results we can say for (C + M + F + 15%) sample all the particles are well arranged so the gap between the particles are minimum so the MDD of the sample goes to increase.

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Keyword Kaolinite clay · Sand gradation · Optimum moisture content · Maximum dry density

1 Introduction

Compaction of soil is a fundamental part of any construction procedure. It is utilized for supporting the structural entities; foundation of the building, roads and railways, walkways, earthen dams, and retaining structures are some examples. For a given soil type, certain properties may consider it more or less desirable to perform adequately for a particular circumstance. Foundation soil should have satisfactory strength and should be relatively incompressible against the future settlement, volume change should be not occur with varying the water content and other environmental factors and also should be durable and safe against decline also the soil should have appropriate permeability.

Compaction of a soil is an important parameter for finding out the various geotechnical problems, such as building settlement seepage, and also controlling the stability of soil masses. Muni Budhu mentioned in his book that the maximum dry density basically depends on the soil type [1]. Well-graded coarse-grained soils accomplish higher density and lower optimum moisture contents then fine-grained soils. Reason is fine particles have greater specific surface which requires more water for lubrication. Type of the soil, particle size distribution, shape of the soil particles, specific gravity of soil mass, quantity and type of the present clay minerals have a great effect on the maximum dry density and optimum moisture content.

Many factors like particle size and shape, void ratio, degree of saturation, and water absorption capacity have greatly influenced on soil compaction. Das said in nature it is quite difficult to find pure sand or pure clay but generally various soils found to gather [2]. Generally, all engineering methods and designs are for pure soils only, but in nature is always not possible various soils like aggregate, sand, silt, clay found to gather and it is also difficult to classify the properties of that soils. Dafalla concluded that sandy soils found in different gradations and soil gradation also affect the characteristics of soil, hence attempts are made by many researchers to study the sand–clay mixture and apply them in practical application; he also said that in sand–clay if the amount of clay content is small then also the mixture is classified anything other than sand and there is a stage at which the mixture is start as clay and clay domain the engineering property of the mixture [3]. Noor et al. had performed tests on various sand–bentonite mixtures and concluded that for 0–10% clay content maximum dry density increases and further increasing of cohesive fraction reduced the maximum dry density. Optimum moisture content increases with increasing clay content from 0 to 20% and clay contents are the reason for the increasing the moisture content [4]. Akyuli et al. concluded that in sand–clay mixture it is problematic to establish geotechnical characteristics of the mixture because this mixture contains both the properties of sand and clay also. He also established that cohesion, compression index, and plasticity index are increasing with an increase in clay content while friction angle and initial void ratio decreases with increasing clay content [5]. Al-Shayayea had concluded that clay particles have domain influence on soil mass even they are available in very small amount he concluded that dry density of sand–clay mixture increases with increasing moisture content and also the clay content up to the certain limit after that it decrease he also concluded that dry density of sand–clay mixture increases with increasing moisture content and also the clay content up to the certain limit after that it decrease [6]. The soil type, shape, and size of the soil particles, specific gravity of the soil grains and content, and types of clay minerals present in soil have a great influence on the maximum dry unit weight as well as optimum moisture content. Soil compaction is a useful parameter for increasing shear strength bearing capacity of soil, to reduce the settlement, and also to reduce the permeability. Pakbaz and Moquaddam state that nature granular soil contains considerable amount of clay or silt and it affects the geotechnical properties of the soil [7]. Wasti and Alyanak worked on sand–clay mixture and conclude that when clay particles fill all the voids of sandy soil then sandy soil changes its property sand to clay [8]. According to the ASTM when the percentage of material passing by the sieve no. 200 is greater than 50% it is classified as clay [9]. Khan et al. mixed natural clay with 20 and 40% sand and compaction strength parameters determine and concluded that compression strength increases by decreasing water content and decreases by increasing sand content [10]. Dixo et al. concluded that addition of sand content increases the compaction density [11]. Shafiee et al. concluded not only the characteristics of sand and clay but particle size of sand also influences on behavior of sand–clay mixture [12]. Pakbaz and Moquddam worked on effect of sand–clay mixture and concluded that in sand–clay mixture with increasing sand gradation cohesion increases and angle of friction decreases [13]. American Association of State Highway and Transportation Officials suggested the fine contents used for a reinforced soil retaining wall is necessary <15%. However, some other factors also governing, like the accessibility of superior backfill materials and cost of the construction [14]. Elkady et al. mixed 0–60% clay with sandy soil and result shows with an increasing clay content pore size structure changed [15]. Nagaraj concluded that sand gradation has a great influence on sand-clay mixture, and his results show that medium-grained sandy soil and clay mixture have a greater strength as compared to all other sand grades [16]. Srikanth and Mishra made a mixture of 50–90% bentonite clay with fine- and medium-grained soil and concluded that fine sand–bentonite mixture has higher OMC and lower MDD [17].

2 Materials and Methods for Experiments

For the present study work, experimental methodology includes sample procurement followed by preliminary laboratory testing and detailed laboratory testing methodologies. Two types of sample materials were used (i) kaolinite clay and (ii) Ennore sand of different gradation.

2.1 Following Tests Were Carried Out at Geotechnical Laboratory

- Atterberg limits—Liquid limit was determined by Casagrande apparatus, and plastic limit and shrinkage limit were determined as per IS: 1498 for the kaolin clay [18].
- Specific gravity tests were determined using 50-ml density bottle for kaolin clay and pycnometer is used for different graded sands as per IS: 2720—Part VI [19].
- Hydrometer tests were carried out as per IS code for finding out grain size distribution of kaolin clay as per IS: 2720—Part IV [20].
- Sieve analysis for different grade of sands as per IS: 2720—Part IV [20].
- OMC-MDD tests were carried out using light compaction and 24-h soaking duration as per IS: 2720—Part VII [21].

3 Experimental Methodology

The study being an experimental work requires following methodology: collecting samples, analyzing the collected samples, carrying out various experiments on that samples, and arising conclusions from the results (Fig. 1).

3.1 Procurement of Samples

In this research, two different soil materials were used. Sand samples were procured from the locally available sand supplier. Grade of sand is first class and normal silica

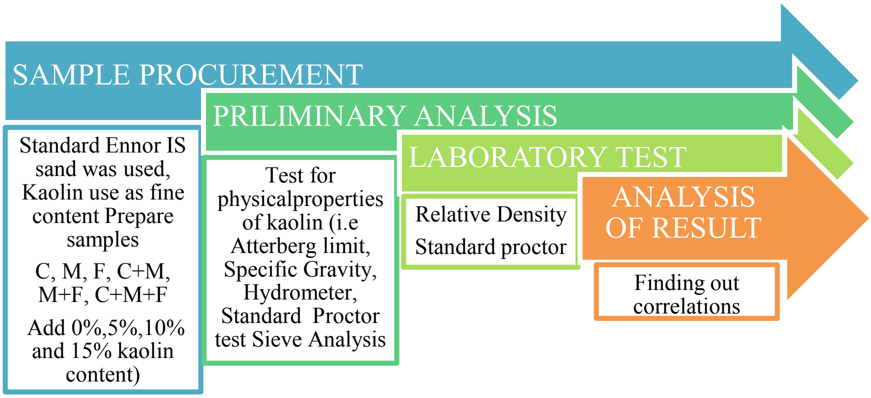


Fig. 1 Methods adopted in the project

Ennore sand. They are fit to be used for various construction works and completely free from the cohesive material, i.e., purely sandy samples. Kaolin clay was used as a cohesive fine fraction which is commercially available.

3.2 Preliminary Analysis

Sieve analysis was performed for finding out gradation of sand and the proportions of course (2 mm), medium (2–0.425 mm), and fine (<0.425 mm) sand in the sample were separate out; total six samples were prepared with different grades of sand particles. Specific gravity tests were performed for find out the specific gravity of different gradation of sand and kaolinite clay. Atterberg limit and hydrometer tests were also performed to know the physical properties of kaolinite clay.

3.3 Laboratory Test

From the procedure six sand samples were arranged with dissimilar gradations. On each set of samples (0, 5, 10, 15%) kaolin would be added. Relative density test would have been carried out for finding out the density of various grade of sand for (0% kaolin clay content) standard proctor test that was performed for all other samples to find out the OMC-MDD (Fig. 2).

4 Results and Discussions

The physical properties of pure kaolin clay and different gradation of sand will be found out before sand–clay mixing. On kaolin clay, grain size distribution, Atterberg limits, specific gravity, and standard proctor compaction test were performed. For various grades of sand samples, sieve analysis was carried out for making the three well-graded and three poorly graded samples.

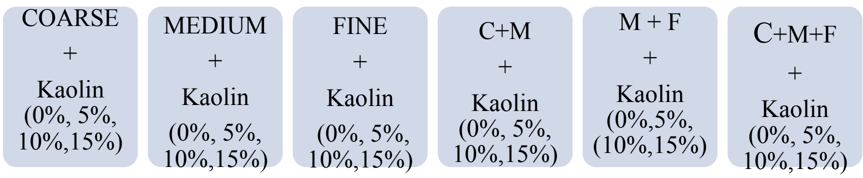


Fig. 2 Test sample distribution

Table 1 Details of laboratory test

S. No.	Experiment	Test no.	IS reference
1	Grain size distribution (hydrometer + sieve analysis)	(01 + 06)	IS: 2720—Part IV
2	Specific gravity	06	IS: 2720—Part III
3	Relative density	06	IS: 2720—Part XIV
4	Standard proctor	19	IS: 2720—Part VII

The results of various relative densities and standard proctor test for different grades of the cohesionless soil obtained were analyzed, and experiential relations have been also recognized between sand gradation and Compaction (Table 1).

4.1 Sieve Analysis

From the procedure six samples were prepared in which three samples were poorly graded (coarse <2 mm, medium 2–0.425 mm, fine <0.425 mm) and three well-graded (coarse + medium, medium + fine, coarse + medium + fine) samples were prepared. From the plot between percentage finer and particle size, the values of D10, D30, D60, Cu, and Cc were determined. D10, D30, and D60 were found from the particle size gradation curve as per IS: 2720—Part-4 [20] (Fig. 3).

$$C_c = D_{30}^2/(D_{10}.D_{60}),\ C_u = D_{60}/D_{10}$$

Table 2 gives the grain size distribution of various gradation of sand and specific gravity of various gradation of Ennore sand. For samples 4, 5, and 6, $C_u > 6$, and $1 < C_c < 3$ implying that these are well-graded samples and other samples are uniformly graded samples or poorly graded samples. The specific gravity for fine sand is minimum and it is 2.58 while specific gravity for coarse sand is maximum and it is 2.72. From the above results, we can say that as the gain. Size goes to irease

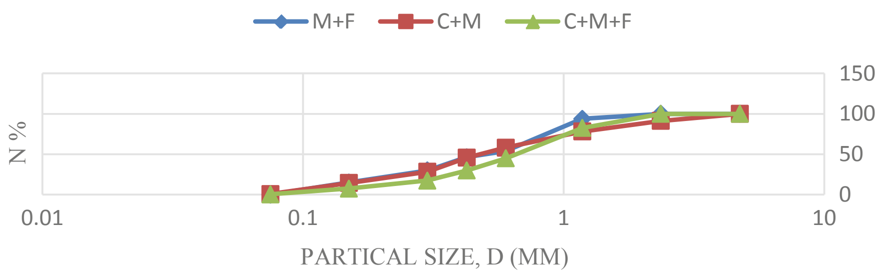


Fig. 3 Partial size distribution curves for well-graded sand

Table 2 Sieve analysis and specific gravity data for sand sample

Sample No.	Coarse sand (%)	Medium sand (%)	Fine sand (%)	D_{60} (mm)	D_{30} (mm)	D_{10} (mm)	(C_u)	(C_c)	Specific gravity
1	100	–	–	–	–	–	–	–	2.72
2	–	100	–	–	–	–	–	–	2.64
3	–	–	100	–	–	–	–	–	2.58
4	50	50	–	0.6	0.45	0.3	6.2	1.02	2.68
5	–	50	50	0.55	0.35	0.13	6.33	1.4	2.614
6	33.33	33.33	33.33	1.00	0.4	0.21	6.5	1.18	2.61

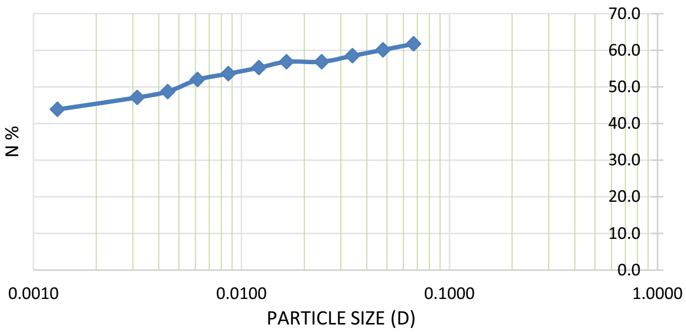
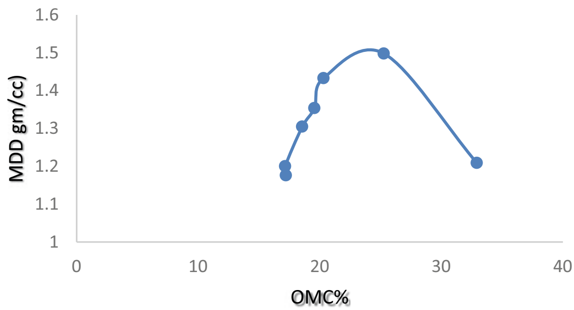


Fig. 4 Hydrometer graph for kaolinite clay

Fig. 5 Compaction curve for kaolinite clay



the Specific density of the soil sample also goes to increase and for the (M + F) and (C + M + F) samples the specific gravity is almost same (Figs. 4 and 5).

As shown in above graph for the kaolinite clay the total clay percent for the kaolin clay heaving the clay content is 45%. The specific gravity of the sample is 2.597 liquid limit of kaolin is 66.4 while (Ip) of kaolin is 33.06 so as per the soil classification the soil is classified as clay of high compressibility (CH) as per IS: 1498—(1970) group [18]. Standard proctor test had been carried out on the pure kaolinite clay sample.

Maximum dry density and optimum moisture content had been finding out for the pure kaolinite sample. For the pure kaolinite clay, MDD is 1.510 gm/cc while OMC is 24.15% (Table 3).

4.2 *Procurement of Samples*

Relative density test was performed for finding out the maximum and minimum density of all of the six samples at 0% kaolin content. This test gives us the maximum and minimum density which is used to find the relative density for the six samples prepared and tested for the relative density test.

Table 3 Physical properties of kaolinite clay

Property	Value
Liquid limit (LL)	66.4%
Plasticity index (Ip)	33.06%
MDD	1.510 gm/cc
OMC	24.15%
Specific gravity	2.597
Percentage of clay	45%
Soil type	CH

Table 4 Relative density data of the prepared samples

Sample No.	Sample type	$\rho_{\min}$ (gm/cc)	$\rho_{\max}$ (gm/cc)	ρ_{d} (gm/cc)
1	Coarse sand	1.55	1.92	1.79
2	Medium sand	1.53	1.89	1.746
3	Fine sand	1.5	1.88	1.726
4	Coarse + medium sand	1.65	2.06	1.94
5	Medium + fine sand	1.7	2.09	1.732
6	Coarse + medium + fine Sand	1.7	2.07	1.865

The maximum and minimum dry densities and void ratios are calculated as follows

$$D_r = \left(\frac{\rho_{\mathrm{d}} - \rho_{\min}}{\rho_{\max} - \rho_{\min}}\right) * \frac{\rho_{\max}}{\rho_{\mathrm{d}}}$$

Relative density tests were performed for finding out the density of pure sand for all the six samples (C, M, F, C + M, M + F, C + M + F) and from all of that 70% relative value was selected for all of the six samples. From all above the six samples, the relative density for (medium + fine) grained soil is maximum while the value for fine-grained sand is minimum (Table 4).

4.3 Compaction

Standard compaction test was carried for finding out the optimum moisture content and maximum dry density of all of the six samples for (5, 10, and 15%) kaolin content. The results of maximum dry density and optimum moisture content are given in Table 5 and relationship for various kaolinite content with the different sand gradations are shown below graphs (Fig. 6).

Relative density for cohesionless coarse-grained soil at 70% relative value is 1.79 gm/cc and when the cohesive fine content is added to the soil dry density of soil

Table 5 Compaction test results

Sample No.	Clay percentage	OMC (%)	MDD (gm/cc)
1	Coarse + 5%kaolinite	11.41	1.845
	Coarse + 10%kaolinite	10.82	1.873
	Coarse + 15%kaolinite	10.34	1.911
2	Medium + 5%kaolinite	12.14	1.78
	Medium + 10%kaolinite	11.85	1.881
	Medium + 15%kaolinite	10.94	1.907
3	Fine + 5%kaolinite	12.29	1.748
	Fine + 10%kaolinite	11.70	1.801
	Fine + 15%kaolinite	11.29	1.865
4	Coarse + medium + 5%kaolinite	10.91	1.946
	Coarse + medium + 10%kaolinite	10.53	1.952
	Coarse + medium + 15%kaolinite	10.46	1.976
5	Medium + fine + 5%kaolinite	12.14	1.745
	Medium + fine + 10%kaolinite	11.85	1.831
	Medium + fine + 15%kaolinite	10.99	1.871
6	Coarse + medium + fine + 5%kaolin	10.66	1.93
	Coarse + medium + fine + 10%kaolin	10.41	1.951
	Coarse + medium + fine + 15%kaolin	10.20	1.997

increase up to 1.85 gm/cc by further increasing the clay content again MDD increase. For medium-grained cohesionless soil, relative density at 70% relative value is 1.746 gm/cc and when cohesive fine content added to the soil dry density of soil goes to increases up to 1.78 gm/cc by further increasing the clay content MDD also increase. For fine-grained soil graph also the relative density at 70% relative value is laser than the MDD for (fine + 5%) but as the clay content increases the MDD also increases. For (coarse + medium)-grained soil, relative density for 0% kaolin content at 70% relative value is 1.732 and further increasing the clay content MDD of the sample again increases. For (medium + fine)-grained soil relative density at 70% for the 0% kaolinite clay is 1.94 gm/cc while MDD for (M + F + 5%) sample is 1.745 gm/cc which is much lesser then the relative density of the same sample. For (C + M + F) grained sample also the relative density of pure clay sample is lesser then the MDD of the (C + M + F + 5 kaolinite%) sample. lesser then the MDD at 5% kaolinite of the same samples. While for the samples 2, 3, and 5 relative density at 70% for the 0% kaolinite content is more than MDD for the 5% kaolinite of the same samples. So, from the results we can say that for the coarse-grained sand-kaolin have greater MDD than the medium and fine grains sand mixture with kaolin for the same so from the above cases we can say that for samples 1, 4, and 6 relative density at 0% kaolinite clay is proportion of bentonite but from all the six samples (C + M + F + kaolin) sample have maximum MDD for all the clay contents (5, 10, 15, 20%) and

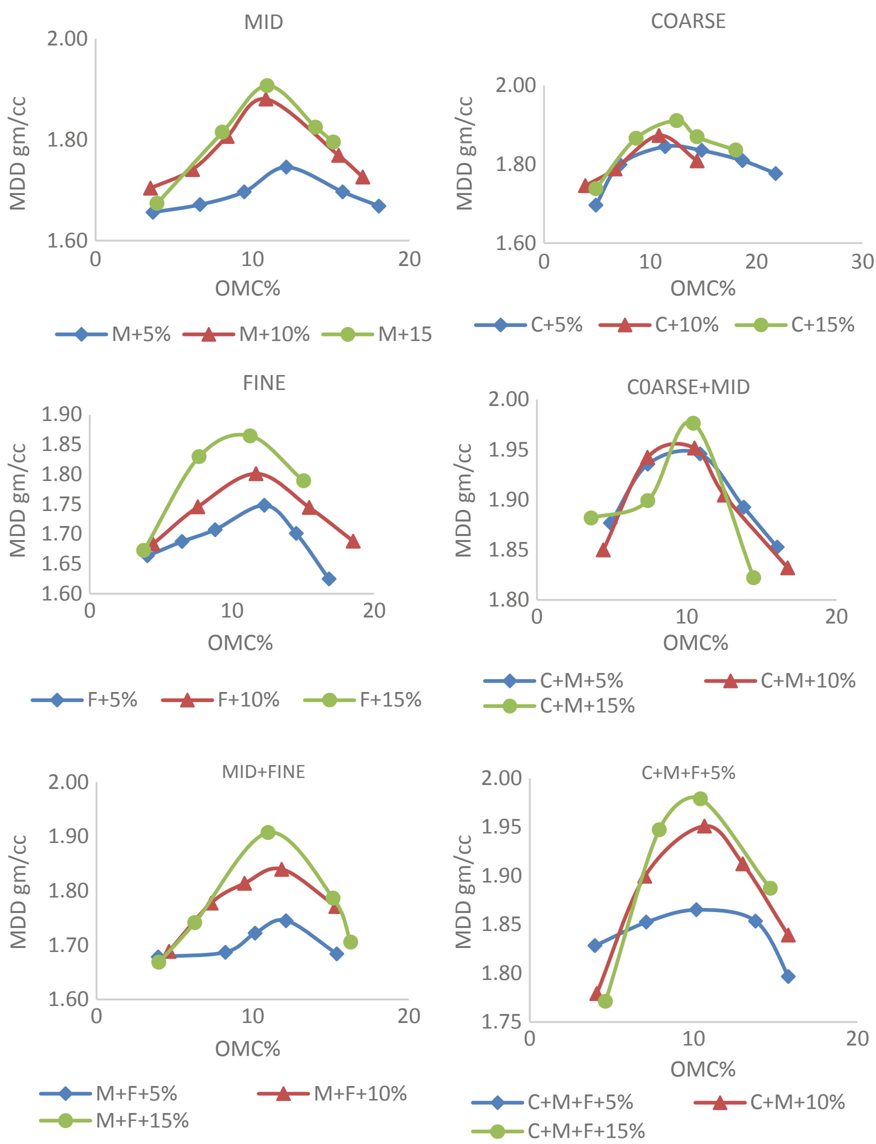


Fig. 6 Compaction curves for various sand–clay mixtures

the reason is fine particles well arranged in the gap between the coarse particles and so for the well-graded samples formed. Hence, MDD for this sample is greater than all other samples, while fine sand has lesser MDD than all other samples (Fig. 7).

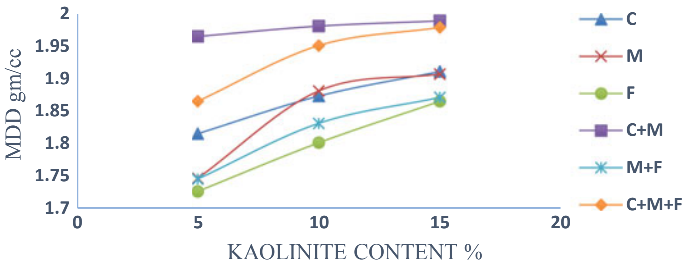


Fig. 7 Influence of kaolin content on maximum dry unit weight

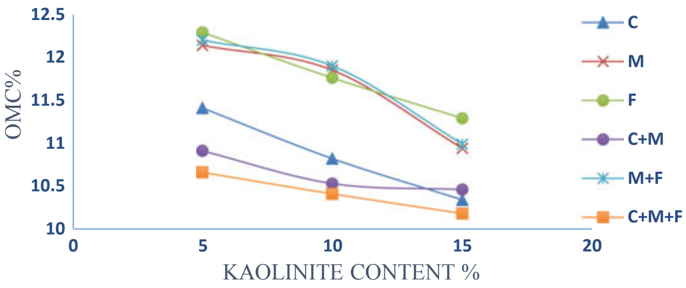


Fig. 8 Influence of kaolin content on optimum moisture content

4.4 Influence of Kaolin Content on Maximum Dry Unit Weight

Above graph shows the effect of various kaolinite contents on MDD of various sand gradation. Maximum dry density increases with increasing the clay content from 5 to 15%. The reason for the increasing the MDD is kaolin particles fill the gap between the sand particles and increase the MDD (Fig. 8).

5 Conclusion

Compaction is a key parameter to understand nature of any soil. Certain conclusions are arrived that will help in understanding the effect of clay content and particle size distribution on moisture content and dry density. The summary of the findings is as follows:

- Cohesive fine particles have falling effect on OMC and MDD, and this effect depends on amount of clay particles.
- Optimum moisture content has been decreased with increasing the plastic fine fraction for all cases.

- For all the cases, maximum dry density increases with increasing the plastic fine fractions.
- Among all the cases (F + 5%kaolin), soil has greater OMC while, (C + M + F + 15%kaolin) soil has lesser OMC.
- Among all the cases (C + M + F + 15%kaolin) grained soil has greater MDD and the reason is in this sample all the soil particles are well arranged while (F + 5%kaolin) soil have lesser OMC.
- From Fig. 7, we can conclude that for the greater kaolinite content, we have greater MDD so the soil has greater shear strength.

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An Experimental Study on Improving the Performance of Silty Soil by Encased Granular Column Using Shredded Tire Chips

Muni Pradeep, Anil K. Choudhary, Awdhesh K. Choudhary, and S. K. Shukla

Abstract Use of stone/granular columns is considered as one of the influential soil-stabilizing methods that can increase the load bearing capacity of soft soil foundations considerably. Recently, it has been reported that the stone aggregate can be replaced by shredded tire chips partially or fully, which will lead to an economical design solution for a granular column. On the other hand, the tire chips are waste materials, and if used for major geotechnical applications, this will not only solve the environmental problems but also result in considerable savings in terms of natural resources thus making the construction method sustainable. This paper presents the results of an experimental study carried out with shredded waste tire chips as a substitute fornatural aggregates and also their effect on granular column behavior. Typical size ranges of tire chips used in the study were 10 mm × 10 mm × 10 mm. The aggregates used were passing through 12.5 mm and retained on 10 mm sieve. Geogrid encasement, namely combi-grid, was used in experimental investigation. A series of small-scale model tests was performed on ordinary granular column (OGC) as well as granular columns made up of shredded tire chips and aggregates and encased with combi-grid (EGC). All the granular materials were compacted at 90% relative density. The strain rate for all the tests were maintained at 1.2 mm/min. The results of the study indicate that the load-carrying capacity significantly increased even the granular column made of 100% tire chips. It has also been observed that ordinary stone column made of aggregates without encasement can be completely replaced by the encased granular column made of 100% tire chips. However, the load-carrying

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capacity of encased stone column made of aggregates found to be higher than that of tire chips.

Keywords Granular column · Shredded tire chips · Combi-grid

1 Introduction

Soils with large volume fractions of silt and clay are the most troublesome and challenging to the geotechnical engineers. Silty soils hold the moisture so that it is very difficult to drain through this. So poor drainage is the most problematic condition with silty soil. Major geotechnical problems because of silty soil are due to low bearing capacity, low strength, poor drainage, low stiffness, high compressibility, and sometimes high liquefaction potential. By considering all these conditions, a suitable and effective ground improvement technique is needed.

So many soil-stabilizing methods are available for soft soils. Out of all stabilizing methods, stabilization of silty soil with granular column using geosynthetic encasement found to be most promising both in terms of reliability and cost feasibility. Granular column consists of crushed coarse aggregates of various sizes and mainly aggregates used as a granular material for granular columns.

Large quantities of natural materials such as aggregates are used in the construction of roads, embankments, and all other civil engineering structures. Nowadays due to lack of these natural materials, there is a need to find suitable alternative material, which will replace these natural materials. Shredded tire chips are the one such alternative material for aggregates which is derived from worn-out tires.

Further, due to the provision of geosynthetic encasement to granular columns, which not only increases the stiffness and load-carrying capacity of granular columns but also reduces the resulting settlements and liquefaction potential of silty soil. In the present study, geosynthetic, namely combi-grid, was used as an encasement material.

2 Motivation and Objective

To evaluate the improvement in ultimate load-carrying capacity of granular column by shredded tire chips with combi-grid encasement, a series of load test has been carried out.

3 Literature Review

Many laboratory studies of granular column and types of alternative materials available for aggregates have been reported in literature by various researchers. The beneficial effects of provision of geosynthetic encasement in the increment of load-carrying capacity of granular column in soft soils have also been reported. Malarvizhi and Ilamparuthi [1] studied the load versus settlement of soft clay bed stabilized with plain stone columns and geogrid encased stone columns using laboratory scale model load tests and found that the ultimate load bearing capacity of geogrid encased granular column treated soft soil beds are three times that of the untreated bed and ordinary granular column-treated soft soil beds are two times that of the untreated bed.

Ayothiraman and Soumya [2] used worn-out waste tire chips as an alternative material to stone aggregates in the construction of granular columns. From the experimental results, it is shown that shredded tire chips can be used as replacement of stone chips up to about 50–60% in the granular columns which is the economical and eco-friendly solution. Shariatmadari et al. [3] used three sizes of shredded waste tires (fine, medium, and large) as a substitution for gravel materials and investigated their effect on stone columns behavior. They reported that, replacing 20% of gravel with medium tire shreds (4.75–9.5 mm) leads to about 30% higher value of bearing capacity and high friction angle. Addition of 40 and 60% tire content causes reduction in bearing capacity. Most recently, Mazumder et al. [4] conducted experimental study on behavior of geonet encased stone column with tire chips as aggregates and found that, stone column encased with geonet showed an increase in load-carrying capacity by about 35% as compared to columns made of 100% stone aggregates without encasement.

4 Test Materials

4.1 Silty Soil

Silty soil used in the present study was collected from Seraikela, Kharsawan district, Jharkhand. The silt was transported to laboratory in Jamshedpur using sealed plastic bags. The properties of silty soil were determined by standard test procedures as stipulated in the relevant IS Code of practice. Figure 1 shows the grain size distribution curves for silty soil used in the study and Table 1 summarizes the basic properties of silty soil, respectively.

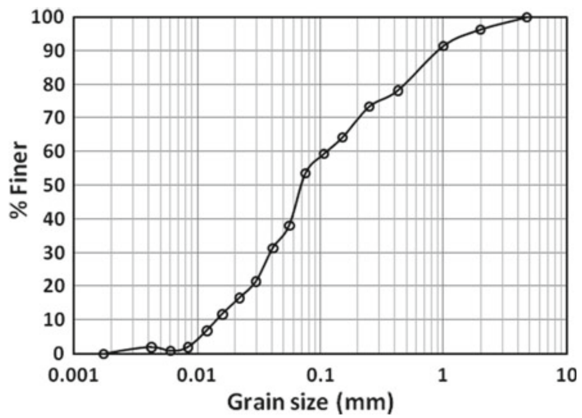


Fig. 1 Grain size distribution of silty soil used in the study

Table 1 Properties of silty soil

Properties	Value	Test code
Specific gravity	2.58	IS 2720-Part 3
Liquid limit	36.2%	IS 2720-Part 5
Plastic limit	28.33%	IS 2720-Part 5
Plasticity index	7.87	IS 2720-Part 6
Soil classification	MI	IS 2720-Part 4
O.M.C	19.55%	IS 2720-Part 7
M.D.D. (g/cc)	1.64	IS 2720-Part 7
Free swell index Fines content	4.17% 53.72%	IS 2720-Part 40 IS 2720-Part 2

4.2 Aggregates

The aggregates required for this study was collected from Jamshedpur. The aggregates passing through 12.5 mm sieve and retained on 10 mm sieve was used in the study. Table 2 summarizes the basic properties of aggregates, respectively.

4.3 Shredded Tire Chips

The derived waste tire chips were obtained by manually cutting waste tires. Derived shredded tire chips of 10 mm × 10 mm × 10 mm made from worn-out tires have been used in this study. The tire chips required for the study was supplied by M.A. Tire shop, Dimna, Jharkhand. The basic properties of tire chips are presented in Table 3.

Table 2 Properties of aggregates

Properties	Value	Test code
Specific gravity	2.73	IS 2386-Part 3
γ_{dmax} (kN/m^3)	16.40	IS 2386-Part 3
γ_{dmin} (kN/m^3)	14.60	IS 2386-Part 3
Relative density (%)	90.00	IS 2386-Part 3
γ_{d} (kN/m^3)	16.20	IS 2386-Part 3
Impact value (%)	7.80	IS 2386-Part 4
Crushing value (%)	13.61	IS 2386-Part 4
Flakiness index Elongation index	21.64 19.08	IS 2386-Part 1 IS 2386-Part 1

Table 3 Properties of shredded tire chips

Properties	Value
Specific gravity	1.14
γ_{dmax} (kN/m^3)	7.40
γ_{dmin} (kN/m^3)	5.98
Relative density (%)	90.00
γ_{d} (kN/m^3)	7.23
e_{max}	0.91
e_{min}	0.54

4.4 Combi-grid

Combi-grid encasement is the next generation of geosynthetic reinforcement products. Combi-grid delivers reinforcement, filtration, separation, and drainage in one composite material. Combi-grid geogrid is mainly used in soft soils where soil reinforcement is in combination with separation and filtration is needed. It is very quick

Table 4 Mechanical properties of shredded tire chips

Properties	Value
Material type	Polypropylene
Standard roll width	4.75 m × 100 m
Tensile strength (kN/m)	>30
Elongation	Nil
Aging	100 years
UV resistance	80%
Seam strength (kN/m) Aperture size	3 32 mm × 32 mm

and easy to install and it has very high radial secant stiffness value. The combi-grid used in this study was supplied by K.K. Suppliers, Kolkata. Table 4 summarizes the mechanical properties of combi-grid used in the study.

5 Test Setup

A cylindrical tank with internal diameter 252 mm and height of 300 mm was fabricated as per (s/d) ratio of 4 and used as a model tank. The inner diameter of the test tank was taken as the diameter of unit cell as per triangular laid granular columns. The diameter of the granular column was taken as 60 mm and all tests were conducted on that granular column. The tanks were made of 3 mm thick galvanized steel sheets. The bottom end was sealed and top end was kept open. Metallic handles were provided for easy handling of tank. The complete test apparatus used in the present study is depicted in Fig. 2.

All the tests were conducted with length to diameter ratio (l/d) as 4. Silty bed was prepared up to height 240 mm and granular column installed at the center and vertical load was applied over a 60 mm diameter steel model footing which is equal to the diameter of the granular column. The footing was placed on the prepared granular column, and the tank was positioned concentrically under the load cell. A sand layer of thickness 3 cm was placed on silty bed around the footing in order to jacket the footing and to prevent it from premature tilt.

Fig. 2 Test apparatus used in the experimental study

6 Preparation of Test Specimen

According to the stone column code IS:15284 (Part 1)-2003**,** this technique is suitable for soft soils like silt and clay which is having undrained shear strength is in the range of 7.0–50 kPa. This technique is not suitable for sensitive soils which are having sensitivity greater than or equal to 4.

A series of unconfined compressive strength tests was conducted on silty soils starting from optimum moisture content 19.55% to up to 31% where undrained compressive strength of soil is 42 kPa at optimum moisture content (19.55%) and 8.5 kPa at 31% water content and maintained same 31% water content throughout the granular column experimental work.

6.1 Preparation of Silty Bed

Soil was crushed using hammer. The required amount of water (31%) was added to oven-dried soil and mixed well to form a smooth paste without any lumps. The soil paste was kept thematically sealed in an airtight polythene bag for 48 h for even distribution of moisture. The inner wall of the tank was cleaned and depth markers were marked at every 3 cm interval. The soil paste was transferred to the test tank in layers of 3 cm each. The amount of soil taken for each layer was equal to the weight of soil required to achieve the needed bulk density. The procedure was continued in layers of 3 cm to a total height of 24 cm. After completion, the top surface was smoothened out using a steel straight edge. Care was taken so as to not disturb the density at the top.

6.2 Mixing of the Tire Chips and Aggregates

The mixing of aggregates and tire chips is done by volume batching. Since the weights or densities of tire chips and aggregates vary by large extent, we adopted volume batching. The mix proportions used in the analysis are 100% AGG (100% aggregates), (50% TC + 50% AGG), and 100% TC (100% tire chips).

The mix proportion of (50% TC + 50% AGG) is as follows.

Weight basis for (50% TC + 50% AGG)

$$\%\text{TC} = \frac{50}{50 + 50} = 50\%$$

Volume basis for (50% TC + 50% AGG)

$$\%\mathrm{TC} = \frac{\frac{\mathrm{Wtc}}{\mathrm{Gtc}}}{\frac{\mathrm{Wtc}}{\mathrm{Gtc}} + \frac{\mathrm{Wagg}}{\mathrm{Gagg}}} = \frac{\frac{50}{1.14}}{\frac{50}{1.14} + \frac{50}{2.73}} = 70.54\%$$

From above calculations it is known that, the tire content of 50% by weight is equivalent to 70.54% by volume.

6.3 Installation of Ordinary Granular Column

All the granular materials (aggregates, tire chips) were cleaned and dried in oven. The dry weight of aggregates and tire chips required to achieve 90% relative density was determined. Dry aggregates material except tire chips were weighed out and soaked in water for 24 h. The soaked material was surface dried with a towel.

Granular column was installed by replacement method. A seamless steel pipe with outer diameter equal to granular column diameter was selected. The driving end of the pipe was sharpened with a bevel edge on the outer surface. The pipe was then inserted at the center of the prepared soil bed until it reached the bottom of the cylinder. The silt inside the cylinder was scooped out using a specially fabricated metallic scoop.

We know the total volume of the cylinder. The total mix proportion is divided into eight equal parts for each 3 cm height based on volume basis. The calculated amount of surface dried granular material was transferred into the pipe in layers of 3 cm thick and compacted with a tamping rod. The depth marking on the inside of steel pipe was used as reference for tamping. After tamping, the pipe was raised in stages in such a way that there is an overlap of 1 cm. The procedure was repeated and granular column was installed in stages till the required height was reached. Because we done on this mixing proportion on a small-scale apparatus, with proper care, we can control the segregation of tire chips and aggregates. After installation, the tank was wrapped with airtight polythene bag for 48 h so that silt regained its lost strength caused by disturbance during installation of granular column.

6.4 Installation of Geosynthetic Encased Granular Column

A steel pipe with diameter equal to that of the granular column was used as template to make the encasement of the right dimensions. The geogrid encasement was prepared by cutting the combi-grid and wrapping tightly around a steel pipe with diameter equal to that of the granular column. The seam end was stitched with face-to-face prayer seam single stitch. The pipe was then removed to obtain the encasement. A strap was attached to one end of the encasement. When the granular material was charged into the granular column, the strap anchored and locked the encasement in position.

6.5 Testing Methodology

The silty bed has been prepared with required consistency found from the series of unconfined compressive strength. The test cylinder was placed in digital CBR machine for carrying out the load vs settlement analysis. The loading was applied at a constant rate of 1.2 mm/min. The load applied on the granular column was measured through a calibrated load cell. The settlement was measured using a calibrated LVDT attached to the cylinder. The test was continued up to failure within the granular column or at the predefined amount of displacement. At the end of each load test, moisture content and dry density need to be determined so as to maintain the standard conditions throughout the investigation.

6.6 Test Variables

Two different series of tests (A and B) were carried out on granular columns. Test series A carried out on ordinary granular column and test series B carried out on combi-grid encased granular column. The undrained shear strength of silty soil was determined by unconfined compressive strength. All the tests were carried out on silty soil at 31% moisture content and at relative density of 90%. All the model tests were conducted on cylinder with spacing to diameter ratio 4 and a total of 7 model tests were conducted in this investigation.

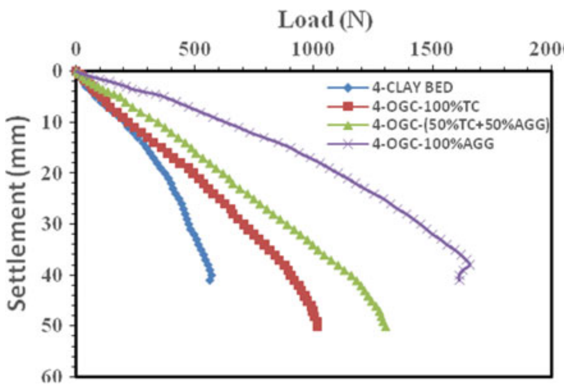


Fig. 3 Load–settlement behavior of ordinary granular column without encasement

7 Results and Discussions

The load versus settlement behavior of granular columns observed from two test series are presented in Figs. 3, 4 and 5. The load–settlement behavior of ordinary granular column with varying percentages of tire chips and aggregates without encasement has been presented in Fig. 3. From the graph, itcould be observed that, the load-carrying capacity is significantly varied with change in mix proportion. So, with the application of granular column, load-carrying capacity increases significantly. From the results, it is also observed that, the ultimate load-carrying capacity of ordinary granular column is decreased by 20 to 40% with increasing percentage of tire chips. It is also observed that the load carrying of column can be significantly increased with provision of combi-grid around the granular column (Fig. 4). This can be clearly seen from Fig. 5. The presentence increase in the performance of granular column with different combination is discussed through efficiency factor in the following section.

Fig. 4 Load–settlement behavior of combi-grid encased granular column

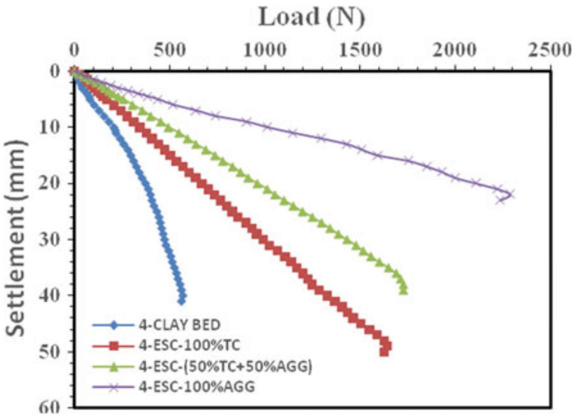
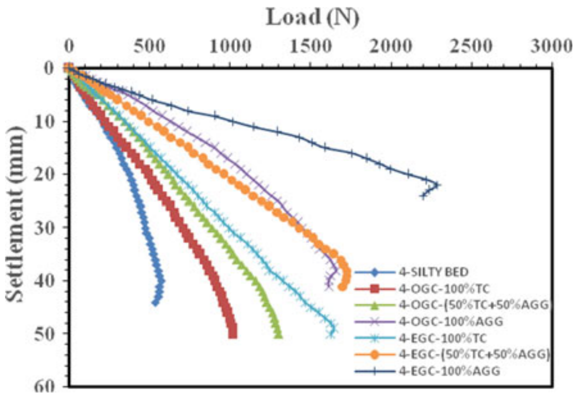


Fig. 5 Load–settlement behavior of both ordinary and combi-grid encased granular column



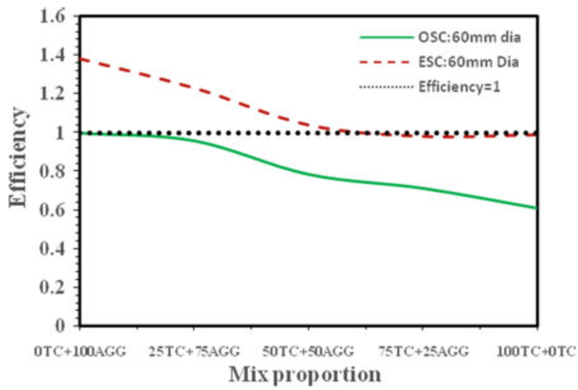


Fig. 6 Variation of efficiency with mix proportion

The variation of efficiency for different mix proportion of aggregates and tire chips and types of granular columns (OGC and EGC) has been presented in Fig. 6. A factor "efficiency" was given by Ayothiraman and Soumya [2], which is used to establish the relationship between optimum percentage of tire chips that can partially replace aggregates in the granular column. Efficiency is defined as a factor which is ratio of load-carrying capacity of either OGC or EGC made of any mix proportion to the load-carrying capacity of OGC made of 100% aggregates. From the graph it could be concluded that, the efficiency of encased granular column is found to be higher than 1.0 for up to 50% tire content and remains close to 1 up to 100% tire chips content.

This demonstrates that the percentage of tire chips that can replace aggregates lies in the range of 50–100% using combi-grid encasement. The efficiency factors were calculated at the peak settlements of individual mix proportion. The peak settlement for 100% tire chips would be 49 mm, 100% aggregates would be 22 mm, and (50% tire chips and 50% aggregates) would be 39 mm. The efficiency reported based on model tests conducted on 60 mm diameter columns made of different mix proportions. So, with the help of combi-grid encasement, the efficiency of granular column could be enhanced. The efficiency of ordinary granular column reduced to 80% up to 50% tire chips content and reduced to 60% up to 100% tire chips content.

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8 Conclusions

From the above model load tests, the following can be concluded.

1. The load-carrying capacity of ordinary granular column can be improved significantly by providing combi-grid encasement.
2. From the results of 10 mm tire chips and 10 mm aggregates with different mix proportions, it is found that the ultimate load capacity of ordinary granular column is decreased by 20–40% with increasing percentage of tire chips.
3. Encased granular column made up of 100% tire chips exhibits load-carrying capacity very close to that of ordinary granular column made up of 100% aggregates.
4. The efficiency of encased granular column is found to be higher than 1.0 for up to 50% tire chips content and remains close to 1.0 for 100% tire chips content. This demonstrates that the percentage of tire chips that can replace aggregates lies in the range of 50–100% using combi-grid encasement.

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Strengthening of Equipment Foundations on Loose Soils of a Power Plant in Eastern Uttar Pradesh

Ravi Sundaram, Sanjay Gupta, Mohit Jhalani, and Jitendra Kumar

Abstract Small equipment foundations of a power plant placed on a 5–6 m thick poorly compacted fill underlain by rock experienced excessive settlements. Strengthening measures adopted included installing micro-piles extending to the top of rock and filling large voids in the fill with a thick grout of cement and sand. This highlights the importance of proper compaction as well as the need to test the bearing strata adequately before constructing foundations on fill.

Keywords Excessive settlement · Gogeotechnical investigations · Foundation strengthening · Micro-piles · Cement grouting

1 Introduction

At a major thermal power plant in eastern Uttar Pradesh, some small equipment foundations were placed on backfilled soils adjoining the turbo-generator (TG) foundation. The TG foundations were seated on the rock at about 8 m depth. The equipments were installed on foundations cast on the backfilled soils.

The filling extended to about 4–6 m depth underlain by rock. These foundations, generally ranging in width from 1 to 2 m bearing just below the floor level, were designed for a low bearing pressure of about 4 T/m^2. The premise was that compacted fill can withstand this low bearing pressure safely.

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Table 1 Measured settlement of equipment foundations on loose fill

Equipment	Measured settlement (mm)
Seal oil	40–180
Stator cooling water skid	60–193
WO & LO coolers of MDBFP-A	40–110
CPU service vessels	16–60
Vacuum pumps	30–60
EHC foundation	35–60
ST forced cooling air foundation	35–60

However, the compaction of the backfilled soil was inadequate. This resulted in excessive settlement of several foundations even before the equipments were made operational. The failure was triggered by the seepage of rainwater into the loose soils. The measured settlement of various foundations ranged from 16 mm to over 180 mm.

Foundation settlements were higher toward the TG side where the backfilling depth is high as compared to the opposite side. Due to foundation settlement, few cracks in the nearby area paving slab was also observed.

Some equipment that had experienced excessive settlement are listed in Table 1.

All these foundations were less than 2 m wide and were designed for a net bearing pressure of 4 T/m^2. Selected photographs of the settled foundations are presented in Figs. 1 and 2.

Since foundations which are placed over inadequately compacted backfilled soil have settled even before the full load was applied on them, the possibility of these foundations to settle further cannot be ruled out as the extent of compaction in the backfilled soil was not uniform.

The possibility of settlement of other foundations which are not settled yet was suspected. Preventive action was required to be taken around all the foundations which are placed over insufficiently compacted backfilled soil in the main power house area especially all around the TG foundation where the depth of backfilling is more.

2 Soil Conditions

To investigate the soil conditions, boreholes were drilled to the top of the rock. Also, dynamic cone penetration tests were performed and terminated upon meeting refusal on the underlying rock. Figure 3 presents typical borehole data.

The borehole data indicated that below the 0.5 m thick floor (150 mm thick RCC, 50 mm thick PCC, 200 mm thick stone soling), the natural soils consist of loose sandy silt/clayey sit of low to medium plasticity. The liquid limit ranges from 22 to

Fig. 1 Cracks in the seal oil foundation

46% while the plasticity index is in the range 8–26%. The shrinkage limit is in the range of 13.9–14.8%. The fill extends to 4–6 m depth below which rock is met.

The field SPT-N values [1] generally range from 1 to 4 indicating the soils are very loose/very soft to soft in consistency. Refusal ($N > 100$) was met at the soil–rock interface at about 4 m depth.

Dynamic cone penetration tests (DCPT) were performed in accordance with IS: 4968 (Part 2)-1976 RA 2007 [2]. DCPT blow-counts (N_{DCPT}) generally range from 0 to 4 to about 3.5 m depth. Refusal was met at the soil–rock interface. Figure 4 presents results of plots of N_{DCPT} versus depth.

In situ density tests conducted below the floor slab suggested that field density of 75–90% of the maximum dry density values (standard proctor [3]) as against the specified minimum of 95%. It is likely that the deeper soils may be looser with percentage compaction of 50–75%.

Fig. 2 Tilted foundation due to settlement

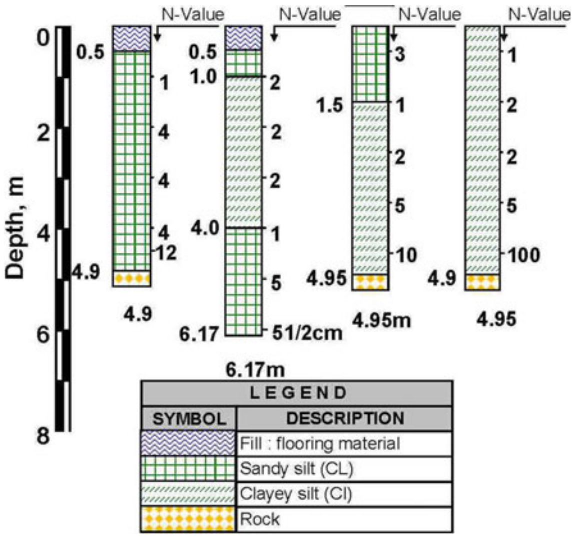
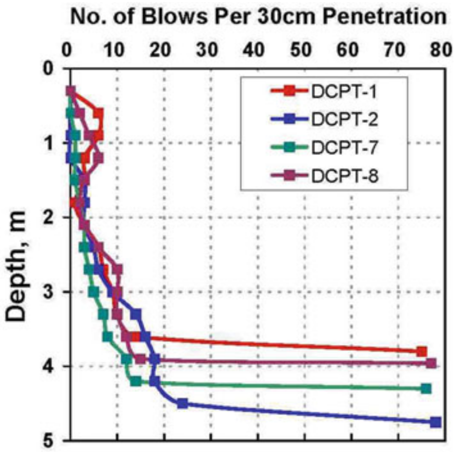


Fig. 3 Typical borehole data

Fig. 4 Typical DCPT plots



3 Engineering Solution

Due to site constraints, it was not feasible to dismantle the equipments, excavate the loose soils, and place back in layers with proper compaction. To arrest the settlement and ensure safe foundations, various methods were explored considering site conditions.

The engineering solution was aimed at restricting further settlement of the loose soils and to transfer the loads safely to the underlying rock.

3.1 Trial Grouting

Initially, trial cement grouting was done all around the foundation to the top of the rock so to form a grout curtain. This shall provide lateral confinement to the soils and restrict further settlement. The grout was prepared by mixing 1 bag of cement with 100 liters of water and pumping into the grout hole until refusal to further grout intake was met under a steady pressure of 1.5–2 kg/cm^2 (Gupta et al. [4]). The intake of cement grout was 60–950 liters of grout (0.5–9 bags of cement) per grout hole. Typical photograph of the grouting in progress is illustrated in Fig. 5.

At places, due to presence of large voids, adequate pressure did not build up even after pumping large quantities of grout. At such locations, soil investigation after grouting did not show much improvement in soil conditions suggesting that the voids were interconnecting and the grout was flowing in an uncontrolled manner.

Considering the uncertainties in grouting in very loose soils, likelihood of high cement consumption and uncontrolled flow direction of grout, grouting adjacent to the foundations was not considered as a preferred sure-shot option.

Fig. 5 Trial grouting in progress

3.2 Micro-piles

3.2.1 Installation Scheme

Micro-piles of 150 mm diameter with casing extending to the top of the underlying rock were installed below and adjacent to the equipment and structurally connected with existing foundation. Prior to piling, the concrete floor slab was cored using a diamond cutter. Boring was done to top of rock and the reinforcement was lowered into the hole. PVC pipe of 150 mm diameter was inserted into the hole to the top of rock. Concreting was done using coarse aggregate of 10–12 mm size to allow flow of concrete into the pile bore. The reinforcement at top was bent and welded to the existing reinforcement of the foundations.

3.2.2 Design Concept

The skin friction of the loose soils was ignored for the purpose of analysis. Since the pies were planned to be seated on refusal stratum/rock, the end bearing component was considered in the design. The refusal stratum was treated as dense sand for purpose of analysis with an angle of internal friction of at least 35°. The vertical compression capacity of the pile was computed as per IS: 2911 Part 1 Section 2—2010 [5].

Conservatively, each pile was designed for a safe vertical load-carrying capacity of 1.5 tonnes in end bearing for a factor of safety of 2.5. A total of 511 piles of 2–4 m length (depending on the depth to rock) was installed in the area adjoining the TG foundation.

3.2.3 Installation Sequence and Guidelines

The micro-piles were installed as per the section drawing illustrated in Fig. 6. The micro-piles were resting on rocky stratum below.

Initially, a hole was drilled in the paving slab for micro-piles. Through these holes cement sand slurry (1:4) in pumpable consistency was pumped below the slab to fill up the void space between slab and filling underneath. Drilling into the soil for micro-piling was started after 24 h of slurry pumping to allow slurry to settle and consolidate. The hole drilled in paving slab was roughed before start of concreting for adequate bonding.

The maximum spacing guidelines, followed for micro-pile execution, is shown in Fig. 7. The first row of micro-piles is installed as close as possible from the edge of foundation or end of floor slab. The spacing between the micro-piles is 1.75 m center

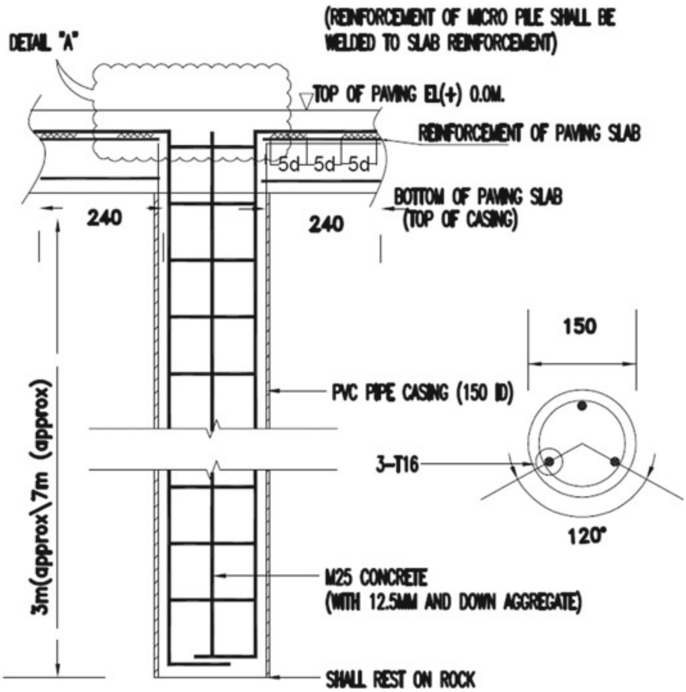


Fig. 6 Typical section of micro-pile

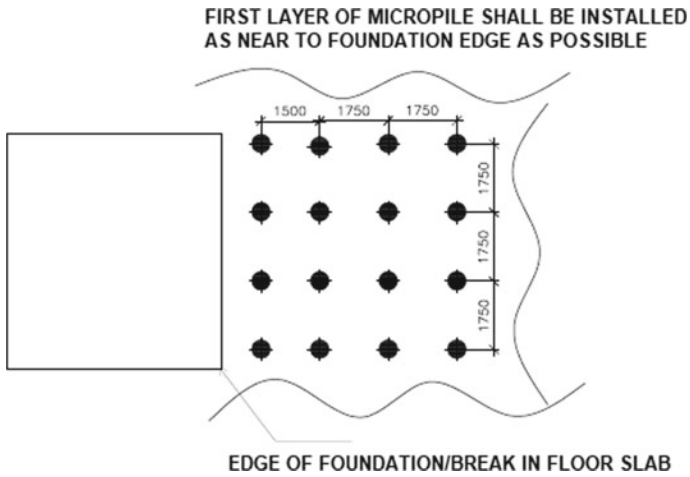


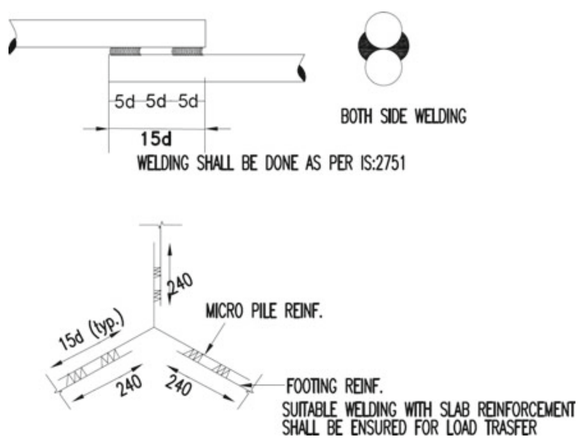
Fig. 7 Spacing guideline for micro-pile

to center. The second row of micro-piles is placed at 1.5 m distance away from the first row. All the further rows of micro-piles are installed at 1.75 m spacing.

3.2.4 Interconnection with Existing Ground Floor Slab

The micro-piles will be effective only if the piles are suitably connected with the existing floor slab and the load transfer mechanism is taking place. The connection detail of slab with micro-pile reinforcement is shown in Fig. 8.

Fig. 8 Interconnection details of micro-pile and floor slab



4 Grouting

In areas where large voids were observed beneath the floor slab (formed by settlement caused by seepage of rainwater through the loose soils), a thick grout of cement plus sand was pumped to fill the voids. Figure 9 shows the concrete flooring between equipment cored for grouting purpose.

This was done as a preventive action around all the foundations and paving area which are placed over insufficiently compacted back filled soil in the main power house area especially all around the TG foundation where the depth of backfilling is more. Improvement of soil using grouting was envisaged to avoid further possibility of settlement.

The purpose of injecting grout was to increase shear strength and to decrease compressibility and permeability. Displacement/compaction grouting consists of using cement slurry or cement–sand mixture which when forced into the soil under pressure displaces and compacts the surrounding material.

A 20-mm diameter conduit pipe was placed in the hole. A steel plate was attached to the conduit pipe at 0.5 m depth. A concrete plug was placed in the grout hole at 0.5 m depth below ground level to seal the hole and to ensure that the soil below the foundation level is strengthened. A typical section of the grout hole is illustrated in Fig. 10.

Fig. 9 Holes cored in between equipment for grouting

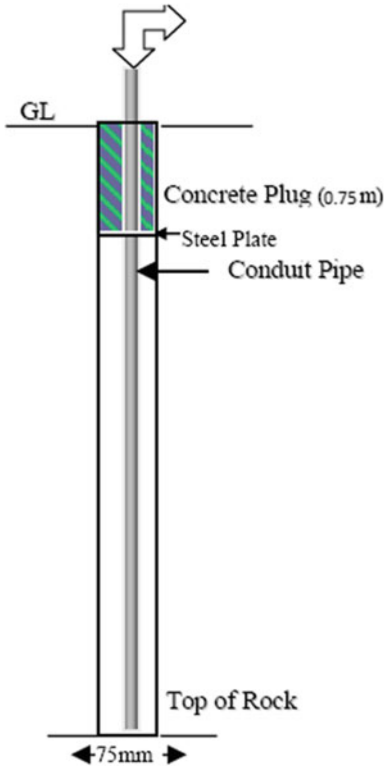


Fig. 10 Schematic of grout hole

5 The Lessons Learnt

This case study demonstrates the importance of proper geotechnical investigation and ensuring thorough compaction of fill on which foundations will be supported, even if it is lightly loaded.

Even if foundations are lightly loaded, placing them on loose uncontrolled fill is fraught with serious risk. On major projects, timelines often dictate such decisions but should be assiduously avoided. Prior to construction, the extent of compaction of the fill should be independently verified to ensure adequate foundation behavior.

Micro-piles extending to the top of the underlying rock were effectively used to transfer the loads safely and bypassing the loose fill. Grouting in the open areas between the equipment was done as a precautionary measure to fill large voids.

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Experimental Study of Stabilization of Expansive Soil Mixed with Sawdust and Marble Dust

Sukanya Sharma, Kalpana Verma, and J. K. Sharma

Abstract This paper aims at studying the effects of marble dust powder and sawdust content as mixtures in clayey/expansive soil and its engineering properties. Expansive soils have high potential for shrinking or swelling. Due to this phenomenon, surface crack occurs resulting in openings during dry season. The expansive soils have variable strength based on its moisture content and have large volume change leading it to unfit for the construction purpose. Based on Indian standard guidelines, CBR, UCS, and standard proctor tests were conducted on the soil sample mixed with 2–10% sawdust and 2–15% marble dust powder to determine the maximum dry density and optimum moisture content at varied percentages of waste admixtures in the soil. The admixtures had an overall positive effect on the geotechnical properties of soil and they can be used as a measure to improve soil strength and contribute toward decreasing the environmental impact of waste materials on our surroundings and it also resolves the problem of waste disposal.

Keywords Black cotton soil · Specific gravity · Atterberg's limit · Sawdust · Marble dust powder · Unconfined compressive strength · Plasticity index

1 Introduction

Black cotton soils in India form a major soil category and cover approximately 20% of the absolute area and are most regularly accessible soil at all places. It is mostly found in central and western parts in India. Black cotton soils for the study were

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derived from Fatehpur, Baran Dist., Rajasthan. The properties of high compressibility and plasticity, high shrinkage, and swelling properties are classified as from medium to high. It is further seen that this soil possesses low strength and undergoes excessive volume changes, making their use in constructions very difficult. The properties of the black cotton soil may be altered in many ways, mechanical thermal, chemical, and other means. It especially involved as a construction material and for foundation purposes; hence, it becomes very important to investigate the physical and engineering properties associated with the black cotton soil especially. In the present study, various tests like grain size analysis, specific gravity, Atterberg's limits, standard proctor compaction, unconfined compressive strength test, and California bearing ratio test were conducted on the soil specimens. The solution of this soil is stabilization with appropriate stabilizing agent. Here, we used waste material sawdust and marble dust and tested for strength, MDD, OMC, plastic limit, liquid limit, CBR, and UCS.

For the study of behavior of expensive soil, the sawdust is mixed in varied percentage of expensive soil. The mixture of expensive soil with different percentage of sawdust may improve the engineering properties of expensive soil. Further different percentage of marble dust powder has been used to strengthen the mix of expensive soil. The main objective of stabilization is to increase the strength or stability of soil and to reduce the construction cost by making best use of the waste materials.

2 Literature Review

Bhavsar et al. [1] presented the effect of marble powder on engineering properties of black cotton soil which has been studied by conducting series of tests. The Atterberg's limits were observed to be decreasing with the increase in the percentage of marble powder. The compaction test was performed which showed that the maximum dry density was increased and the optimum moisture content was decreased with the increase in the percentage of marble powder. The linear shrinkage is decreased with the increase in the percentage of marble powder. It was concluded that the marble powder is preferable for stabilization as it gives positive result.

Singh et al. [2] studied the effect of marble dust on index properties of black cotton soil which is investigated. Various laboratory experiments have been performed on black cotton soil samples mixed with 0–40% of marble dust by weight of dry soil. The test results of samples containing marble dust showed a significant change in consistency limits. The liquid limit decreased from 57.67 to 33.9%. The plasticity index was found to be decreased from 28.35 to 16.67% and shrinkage limit increased from 8.06 to 18.39% when added with marble dust from 10 to 40% of the dry weight of black cotton soil. Also, the differential free swell index decreased from 66.6 to 20%, showing appreciable decrease in swelling behavior. The differential free swelling has reduced from 66.6 to 20%. The degree of expansiveness reduced from very high to low which is indicated by the results of plasticity index, shrinkage limit, and DFS.

Venkatesh et al. [3] evaluated the effect of waste sawdust ash on compaction and permeability properties of black cotton soil that has been studied. The dry density of the soil was increased from 1.40 gm/cc to 1.46 g/cc when 2% of waste sawdust ash was added. The density starts decreasing with the further addition of waste sawdust ash. The coefficient of permeability was found to be reduced from 0.18 to 0.08 with the increase in the percentage of waste sawdust ash. Jasim et al. [4] discussed the effect of sawdust usage on the shear strength behavior of clayey silt soil that is studied. A series of test has been conducted and it has been concluded that the addition of sawdust up to 5% decreased the liquid limit and plasticity index by 14.96% and 17.65%, respectively, and decreased the plastic limit by 13.16%. It has also decreased the maximum unit weight and optimum water content by 8.22% and 10.74%, respectively. The unconfined compressive strength value and unconsolidated undrained shear strength were increased by 41.436% and 39.535%, respectively, when sawdust content was between 0 and 3%. Further addition up to 5% decreased the values. The shear strength of the clayey silt soil was found optimum at the addition of 3% sawdust.

Gautam et al. [5] investigated the effect on the properties of the expensive soil that have been studied using marble dust and coir fiber. With the increasing percentage of marble dust in expansive soil, the medium plasticity clay was found to be turned into low plasticity clay. The DFS of expensive soil is found to be decreased up to 93.44% when 40% marble dust was added. The optimum moisture content was found to be decreased and maximum dry density was increased. The CBR value was observed to be greatly affected as it was increased about 103.63%; when 20% marble dust was added with expensive soil, further addition decreases the CBR value; when 1.5% coir fiber was added to the mixture, the CBR value was increased further with 21.4%. The unconfined compression strength value is increased with the increase in the percentage of marble dust till 20%. The value of swelling pressure was decreased to 0.11 from 1.38 kg/cm^2 when 40% marble dust was added. From the study, it is revealed that the combination of 20% marble dust and 1.5% coir fiber gave the best results.

3 Materials Used

3.1 Black Cotton Soil

The black cotton soil for the present study was procured from Fatehpur, Baran district, Rajasthan, India. The specimens were extracted from the ground with the help of Auger. Further, to avoid any change in moisture content arising due to increase or decrease in the atmospheric temperature, the specimens as derived were immediately placed in polythene covers (Fig. 1; Table 1).

Fig. 1 Black cotton soil

Table 1 Various laboratory tests are mentioned in table as per IS code that were conducted on the soil specimen

Test performed	IS code used
Grain size analysis	IS:2720(Part 4)-1985
Atterberg's limit test	IS:2720(Part 5)-1985
Specific gravity	IS:2720(Part 3)-1980
Std proctor compaction test	IS:2720(Part 7)-1975
UCS test	IS:2720(Part 10)-1991
California bearing ratio test	IS:2720 (Part 16)-1987

3.2 Marble Dust Powder

Waste marble dust is produced from marble cutting and polishing of natural stones. The definition of marble is the metamorphic rock which is hardened and under hydrothermal conditions. The production of marbles dust produced from grinding and cutting of it has non-plastic and very fine particle size and almost well grated. For the stabilization of the soil, the traditional techniques face problems like high cost and/or environment issues. The improvement of soil by marble dust is the alternative solution. The soil stabilized by marble dust can be utilized in the construction of canal lining, pavement structures, and foundations. By using the marble dust, this work focuses to reduce the expansion of expansive soils, and with the increase in the percentages of the soil sample, it notices the change in index properties of soil samples (Fig. 2).

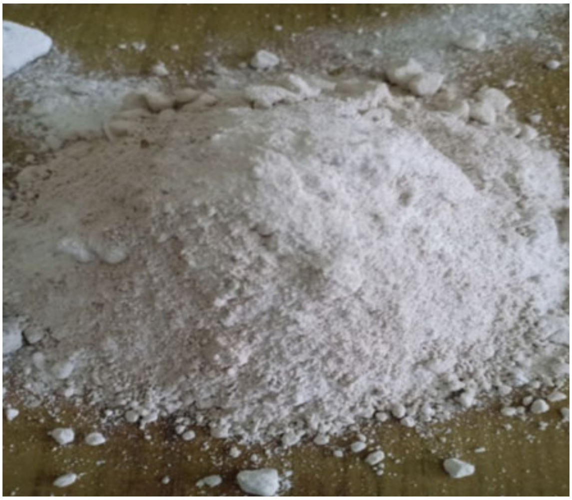
Fig. 2 Marble dust

3.3 Sawdust

It is defined as the by-product of cutting, drilling, grinding, sanding, or pulverizing wood or any other material with the help of saw or by other means of tools. It is also known as wood dust. It is composed of fine particles of wood. It can also be defined as the by-product of certain animals, birds, and insects, i.e., which live in wood, such as the woodpecker and carpenter ant. It can be hazardous to manufacturing industries, especially in terms of its flammability. Sawdust is the main component of particleboard. In this test, the sawdust which is used is of teak (Sagwan wood) of Grade I which is retained from the 4.75 mm sieve (Fig. 3).

4 Experimental Details

The soil sample is collected from Baran district. These soil samples were classified according to Indian standard classification. Atterberg's limit test, sieve analysis, standard proctor test, unconfined compression test, and California bearing ratio tests were conducted. The consistency limit test includes liquid limit and plastic limit test of soil by using cone penetrometer apparatus. Five different percentages of sawdust 0, 2, 4, 6, 8, and 10% and three different percentages of marble dust 5, 10, and 15% were used (Table 2).

Fig. 3 Sawdust

Table 2 Engineering characteristics of black cotton soil

S. No.	Parameters	Test values
1	Soil classification	CH
2	Specific gravity	2.42
3	Liquid limit	52.17%
4	Plastic limit	32.8%
5	Plasticity index	19.37%
6	Optimum moisture content	21.96%
7	Maximum dry density	1.83 g/cc
8	Unconfined compression strength	11.6 kg/cm^2

5 Results and Discussion

5.1 Grain Size Analysis

For the determination of the grain size, the Indian standard guidelines are used and as per IS 2720 (Part 4) 1985 it can be determined. The percentage mass retained was determined for each and the percentage of sample passing each was determined from the data obtained. This was used to plot the graph of particle size distribution on semi-log curve of the sample (Fig. 4).

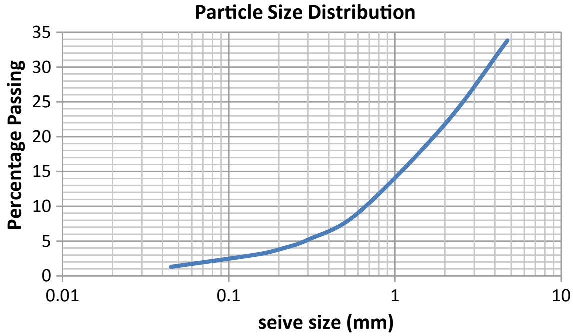

Fig. 4 Particle size distribution curve for black cotton soil

Table 3 Atterberg's limit test values of black cotton soil

Atterberg's limit	Values (%)
Liquid limit	52.17
Plastic limit	32.8
Plasticity index	19.37

5.2 *Atterberg's Limit*

Atterberg's limit tests were conducted for the determination of liquid limit, plastic limit, and plasticity index which were shown in Table 3. This test is conducted according to the IS: 2720(Part 5)-1985.

5.3 *Specific gravity*

For the determination of the specific gravity, the Indian standard guidelines are used and as per IS: 2720 (Part 3) 1980 and the specific gravity test is performed. Table 2 shows the value of specific gravity for the black cotton soil mixed with various percentages of sawdust. The specific gravity values of the samples are decreasing with the inclusion of sawdust. At 4% addition of the sawdust, the specific gravity reaches to maximum and the value is 2.59. Specific gravity decreases with increasing percentage of the sawdust (Table 4).

Table 4 Specific gravity of black cotton soil and sawdust

Properties	Virgin soil	2%	4%	6%	8%
Specific gravity	2.42	2.35	2.59	1.78	1.82

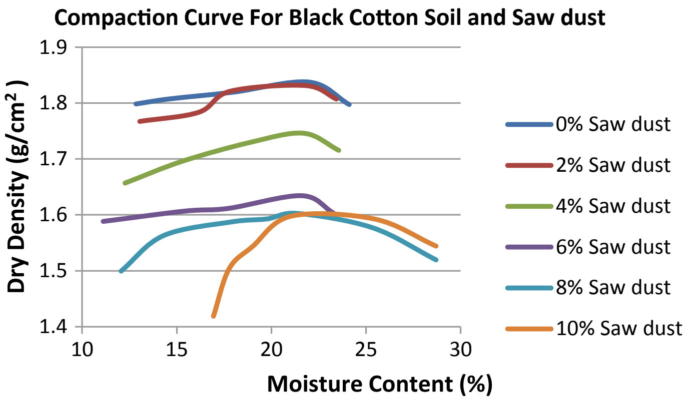

Fig. 5 Compaction test curve of black cotton soil with sawdust

5.4 Standard Proctor Compaction Test

The test has been conducted and the curve is plotted taking moisture content at *x*-axis and dry density at *y*-axis. The following result is interpreted by the figure shown. OMC was decreased by 0.77 at 2% sawdust, further 1.27 and 1.73% at 4 and 6% sawdust. At the addition on 8% sawdust, the OMC is decreased by 2.82 and 4.18 at 10% sawdust. MDD was decreased by 0.33% for 2% sawdust. It is further decreased by 4.98 and 11.06% at 4 and 6% sawdust. At 8% sawdust, it is decreased by 12.74 and 13.04% for 10% sawdust. OMC was decreased by 10.74 at 10% marble dust. Further it is decreased at 20% marble dust by 20.26%.

MDD of marble dust powder at 10% is increased by 2.18% and increased by 9.83% at 20% marble dust (Fig. 5).

5.5 Unconfined Compression Test

The unconfined compression test is performed as per the IS: 2720(Part 10)-1991. Stress–strain curve is plotted which is shown in Fig. 7 taking stress at *y*-axis and strain value is taken on *x*-axis. From the figure, the UCS value is increased by 2.12 at 10% marble dust and 4.78 at 20% of marble dust (Fig. 6).

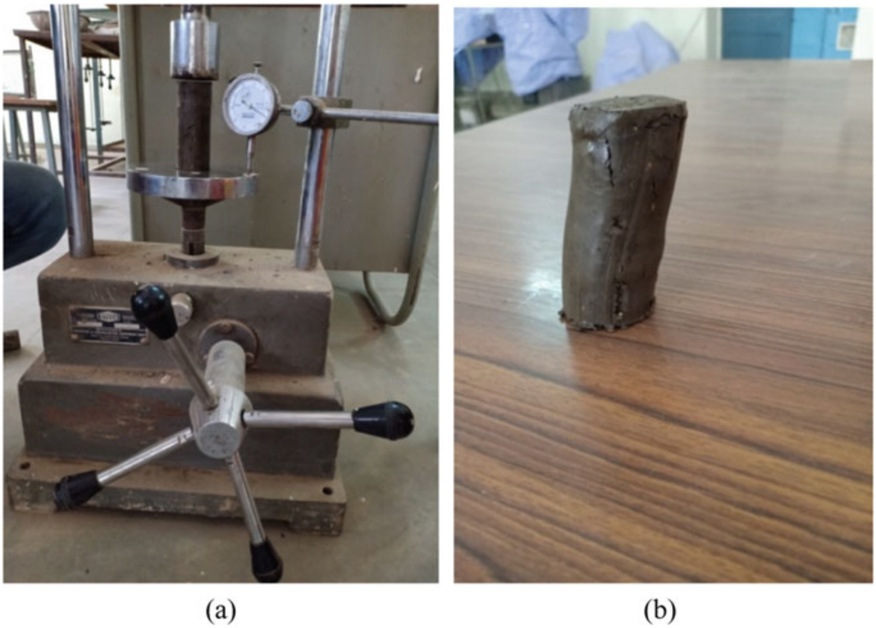

Fig. 6 **a** Unconfined compression testing machine. **b** UCS sample

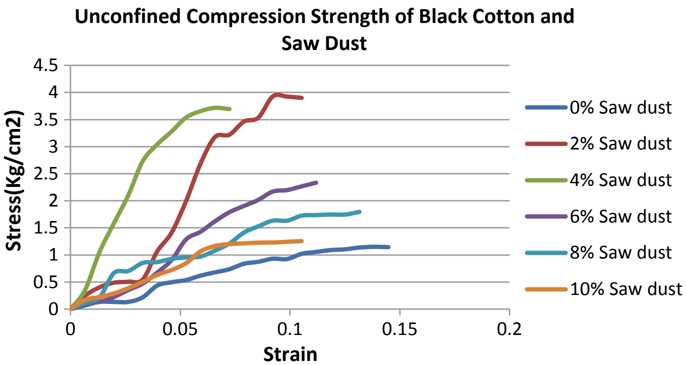

Fig. 7 Unconfined compression strength of black cotton soil with sawdust

5.6 California Bearing Ratio Test

1. California bearing ratio is performed as per the IS: 2720 Part 16. This test is conducted with different percentages of sawdust with the black cotton soil (Fig. 8).

Fig. 8 California bearing ratio

The load versus penetration curve is plotted taking load on *y*-axis in KN and penetration on *x*-axis in mm which is shown in Fig. 9. Further from the curve we can interpret that the CBR value of black cotton soil improves considerably to 10.16% on 6% sawdust content. The figure shown above consists of virgin soil, i.e., 0, 2, 6, and 10% sawdust (Fig. 10).

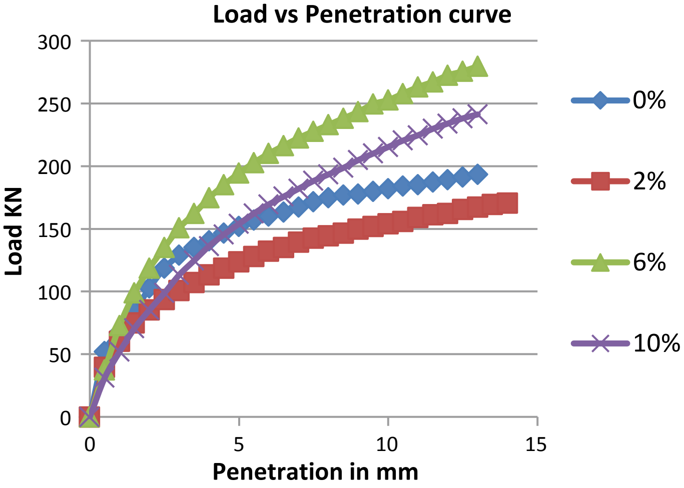

Fig. 9 California bearing ratio test of black cotton soil with sawdust

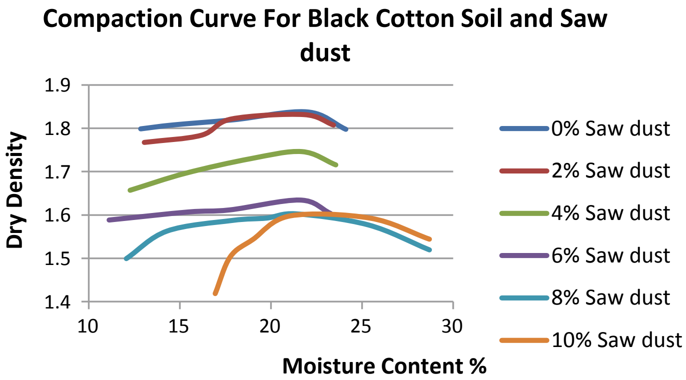

Fig. 10 Compaction test curve for mixed sample

The test has been conducted and the curve is plotted taking moisture content at *x*-axis and dry density at *y*-axis. The following result is interpreted by the figure shown. OMC was decreased by 0.77 at 2% sawdust, further 1.27 and 1.73 at 4% and 6% sawdust. At the addition on 8% sawdust the OMC is decreased by 2.82 and 4.18 at 10% sawdust. MDD was decreased by 0.33% for 2% sawdust. It is further decreased by 4.98 and 11.06% at 4 and 6% sawdust. At 8% sawdust it is decreased by 12.74 and 13.04% for 10% sawdust. OMC was decreased by 10.74 at 10% marble dust. Further it is decreased at 20% marble dust by 20.26%.

MDD of marble dust powder at 10% is increased by 2.18% and increased by 9.83 at 20% marble dust (Fig. 11).

The unconfined compression test is performed as per the IS: 2720(Part 10)-1991. Stress–strain curve is plotted which is shown in Fig. 7 taking stress at *y*-axis and strain value is taken on *x*-axis. From the figure, the UCS value is increased by 2.12 at 10% marble dust and 4.78 at 20% of marble dust (Fig. 12).

The load versus penetration curve is plotted taking load on *y*-axis in KN and penetration on *x*-axis in mm which is shown in Fig. 9. Further from the curve we can

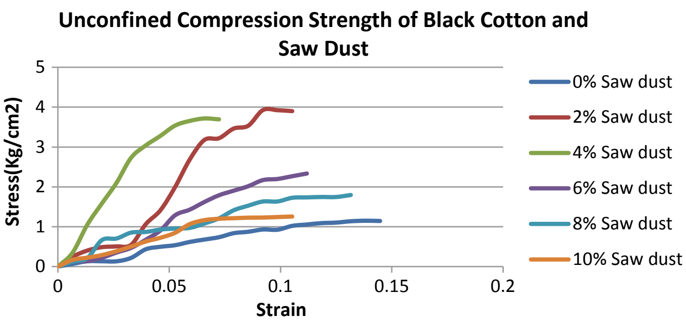

Fig. 11 Unconfined compression test values of a mixed sample

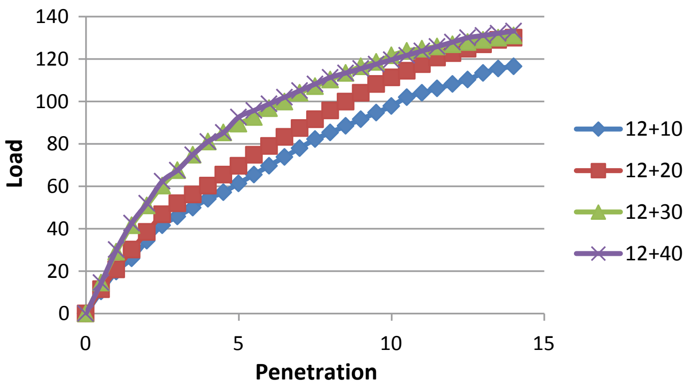

Fig. 12 California bearing ratio curve for mixed sample

interpret that the CBR value of black cotton soil improves considerably to 10.16% on 6% sawdust content. Apart from that the sawdust content is fixed taken as 12% and the CBR value is determined by changing the marble dust values, i.e., 10, 20, 30, and 40%.

6 Conclusion

Based on extensive laboratory tests conducted on black cotton mixed with sawdust from 0 to 10% by weight of dry clay,

The following conclusions can be drawn:

1. The specific gravity values of the samples are decreasing with the addition of sawdust.
2. 4% addition of the sawdust the specific gravity reaches to maximum and its value is 2.59.
3. OMC was decreased by 0.77 at 2% sawdust. Further, it is decreased by 1.27 and 1.73% at 4 and 6% sawdust. On the addition of 8% sawdust the OMC is decreased by 2.82 and 4.18 at 10% sawdust.
4. MDD was decreased by 0.33% for 2% sawdust. It is further decreased by 4.98 and 11.06% at 4 and 6% sawdust. When the 8% sawdust is added it is decreased by 12.74 and 13.04% for 10% sawdust.
5. OMC was decreased by 10.74 at 10% on the addition of marble dust. Further it is decreased at 20% marble dust by 20.26%.
6. MDD of marble dust powder at 10% is increased by 2.18% and it is increased by 9.83 at 20% marble dust.
7. The UCS value is increased by 2.12% on the addition of 10% marble dust and 4.78 at 20% of marble dust.

8. The CBR value of black cotton soil improves considerably to 10.16% on 6% sawdust content.
9. Wood dust accumulations create a number of safety and health hazards. Sawdust becomes a health problem when the wood particles become airborne and they are inhaled.

From the above laboratory investigation, it can be concluded that the sawdust has a potential to modify the characteristics of expansive clay like black cotton soil and to make it suitable in many geotechnical applications and strength will be increase due to marble dust.

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Investigation of the Microstructure of Brahmaputra Sand Treated with *Bacillus megaterium*-Mediated Single-Dosed Bio-Cementation

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Abstract The bio-mediated soil improvement has promising capabilities to provide sustainable aid to the geotechnical challenges. The geotechnical behavior of the soil can be modified utilizing bio-mediated processes. Most of the studies on the bio-mediated soil improvement focuses on a particular bacteria *Sporosarcina pasteurii.* This study utilizes urease positive bacteria *Bacillus megaterium* (NCIM 5472) as an alternative to *S. pasteurii* and Brahmaputra riverbank sand for the investigation of bio-cementation in the soil. In this study, the primary characterization of *B. megaterium* has been reported. The qualitative urease activity of the bacteria has been assessed with Urea Agar Base (Christensen), and quantitative analysis of urease activity assay has been evaluated by the phenol-hypochlorite method. Quantitative calcite precipitation has been evaluated at equimolar cementation solution. After the characterization, single dosing of bacterial broth solution mixed along with cementation solution (one pore volume) is introduced to Brahmaputra riverbank sand, and the microstructure of the sand has been investigated with the help of field emission scanning electron microscope (FESEM) images. The influence of bio-cementation was observed significantly on the microstructure of Brahmaputra riverbank sand in the form of bridging of the calcite precipitated. This study is a preliminary study to investigate the applicability and potential of the bacteria *B. megaterium* for bio-mediated soil improvement. The study concludes that the bacteria *B. megaterium* is moderately urease active, and it has the potential for bio-cementation.

Keywords Bio-mediated soil improvement · Urea hydrolysis · Bio-calcification · *Bacillus megaterium* · Microbial-induced calcite precipitation (MICP)

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1 Introduction

There are various biological activities occurring in the soil media, which may alter the soil pore network [1] and therefore, utilization of the biological processes in the soil media holds promising aid for the geotechnical engineering applications such as in ground improvement, liquefaction mitigation, erosion control, and hydraulic barrier materials [2, 3].

The most prevalently used bio-mediated soil improvement is catalysis of urea hydrolysis process by utilizing ureolytic bacteria such as *S. pasteurii* [3–6]. The urea hydrolysis reaction can be explained as follows

$$CO(NH_2)_2 + H_2O \rightarrow^{\text{Bio-mediator}} H_2CO_3 + 2NH_3 \tag{1}$$

The ammonia further reacts in water and yields in ammonium and hydroxide ions leading to the rise of the pH of the solution, as shown in Eq. 2.

$$NH_3 + H_2O \rightarrow NH_4^+ + OH^- \tag{2}$$

The bicarbonate breaks further into carbonate ion, as shown in Eq. 3 and in the presence of calcium ion, this leads to precipitation of the calcium carbonate crystals, as shown in Eq. 4.

$$H_2CO_3 \rightarrow 2H^+ + 2\,CO_3^{2-} \tag{3}$$

$$Ca^{2+} + CO_3^{2-} \rightarrow CaCO_3 \downarrow \tag{4}$$

The rise in the alkalinity is caused by the yield of ammonium and hydroxide ions as shown in Eq. 2, and the accelerated carbonation in the presence of urease enzyme leads to an accelerated rate of the calcite precipitation [7].

After MICP treatment, the soil pore gets filled with calcite crystals, which leads to a decrease in pore volume, eventually results in a decrease of saturated hydraulic conductivity of soil and an increase in shear strength properties. The bridging among the particles due to bio-cementation also influences the engineering properties of MICP-treated soil, apart from precipitated calcite crystals itself. The order of change in hydraulic conductivity and shear strength of the soil depends on various other parameters like the uniformity and morphology of precipitated $CaCO_3$ crystals [8].

This study is an investigation of *Bacillus megaterium* (NCIM 5472 equivalent collection number ATCC 14581)-mediated MICP and its influence on the microstructure of the Brahmaputra river bank sand. The MICP-related primary characterization of the bacteria *B. megaterium* is accomplished, including quantification of its urease activity and calcite precipitation capacity.

This study accentuates on the bacteria *B. megaterium* as it can grow well in a range of temperature of 3–45 °C, in toxic situations and can survive on a variety of carbon sources [9] and the influence of oxygen on the urease activity of *B. megaterium* is insignificant [10]. As most of the MICP studies emphasize on conventionally popular ureolytic bacteria *S. pasteurii*, there are very few studies available on *B. megaterium*-mediated MICP [11, 12].

2 Materials and Methodology

Brahmaputra river bank deposited sand was collected from the river basin nearby Indian Institute of Technology Guwahati, Assam, India. The soil is poorly graded fine sand as per USCS classification. The fine content was observed to be less than 4%.

The coefficient of uniformity, coefficient of curvature, D_{60}, D_{10}, and D_{30} are shown in Table 1.

The ureolytic bacteria *B. megaterium* (NCIM 5472 equivalent collection number ATCC 14581) is grown in a nutrient broth (NB) media in conical flasks at pH 8 and 30 °C temperature in a shaking incubator at 200 rpm. The growth characteristics of *B. megaterium* were evaluated by measuring its optical density at 600 nm in the spectrophotometer over a time duration of 48 h. Urease activity of the bacteria is one of the indicators of the potency for bio-cementation. The qualitative urease activity of the sample was evaluated using Urea Agar Base, Christenson (UAB), and the quantitative urease activity of the strain was evaluated by phenol-hypochlorite method [13–15]. The qualitative urease test the bacteria was evaluated by the capacity of the bacteria to turn yellowish UAB plates to pink. The calcite producing capacity of the bacteria was also evaluated in the flask tests after mixing the culture broth with the equimolar cementation solutions.

Then one pore volume of the bacterial broth (optical density 1) and equimolar cementation solution (urea and $CaCl_2$) was mixed in equal proportions with the Brahmaputra river bank sand in a cylindrical column of diameter 40 mm and length 80 mm. The samples were extruded for the evaluation of their geotechnical and microstructural properties.

Table 1 Grain size characteristics of Brahmaputra river bank sand

Characteristics	Values (unit)
C_u	1.875
C_c	0.833
D_{60}	0.15 mm
D_{30}	0.1 mm
D_{10}	0.08 mm

3 Results and Discussion

The geotechnical properties of Brahmaputra river bank sand are given in Table 2 as follows

The FESEM and XRD of the Brahmaputra sand (BS) revealed that the BS is smooth surfaced, angular, and quartz dominant, as shown in Fig. 1.

The qualitative urease activity of the bacteria *B. megaterium* was observed using Urea Agar Base (UAB), Christensen (HiMedia lab.). The change in color from yellowish to pink is due to the alkalinity caused by urea hydrolysis and is noted within 24 h, as shown in Fig. 2.

The Urea Agar Base took 24–36 h for turning the plate's color from yellow to pink, indicating that the bacteria is a moderate urease producing bacteria.

Table 2 Geotechnical properties of Brahmaputra sand

Geotechnical properties	Values (unit)
Specific gravity	2.7
pH	7.2
Minimum density	13.1 (kN/m$^{3)}$
Maximum density	15.4 (kN/m^{3})

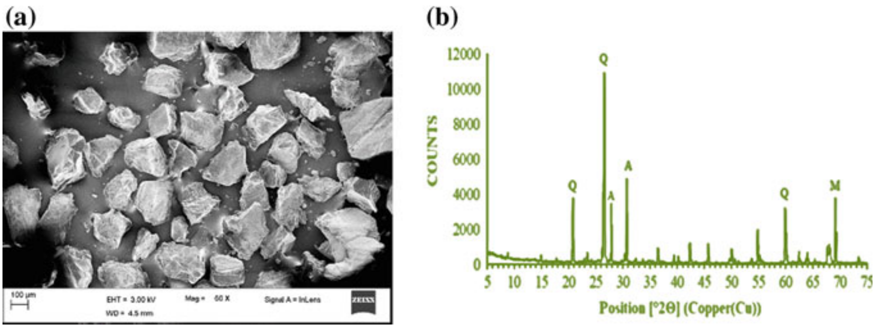

Fig. 1 **a** Field emission scanning electron microscope (FESEM) and **b** X-ray diffraction (XRD) graph of Brahmaputra sand (Q denotes quartz)

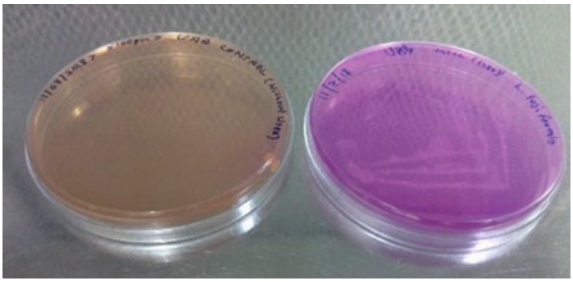

Fig. 2 Qualitative urease test for *Bacillus megaterium* on urea agar base

The quantitative urease activity of the bacteria was 247 U/min. One unit of bacterial urease is termed as the quantity of enzyme required to hydrolyze 1 micromole urea/min/ml [15]. The calcite producing capacity of bacteria in the flask at temperature 30° Celsius and pH 8.0 was observed to be 504 mg/100 ml of cementation solution. The XRD and FESEM of the precipitates revealed it to be calcite (Figs. 3 and 4).

The soil columns were kept for 1 week for bio-cementation. The sample was oven-dried and then extruded before testing it for geotechnical and microstructural characterization. The extruded soil (sand) sample was able to stand due to the bonding induced by the calcite produced. However, the sample did not show sufficient strength for unconfined compressive strength, which may be due to two reasons, 1. less calcite

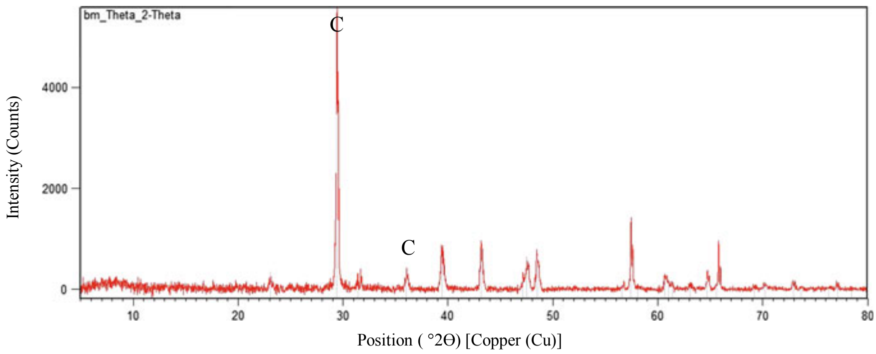

Fig. 3 X-ray diffraction (XRD) graph of Brahmaputra sand of precipitates in flask test (C denotes calcite)

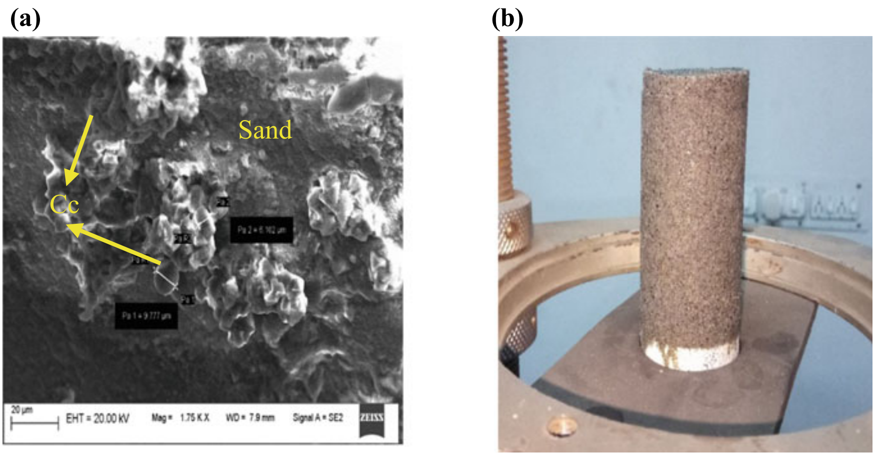

Fig. 4 **a** Field emission scanning electron microscope (FESEM) of the lumps collected from treated soil column, **b** Treated soil column

content due to a single number of dosing and 2. heterogeneous calcite precipitation. The lumps of broken soil sample were taken for microstructural analysis (FESEM and XRD). FESEM revealed that there are precipitates in between the soil pores.

4 Conclusion

In this study, the geotechnical properties of the Brahmaputra river bank sand and the potential of the bacteria *B. megaterium* as a mediator for urease-based bio-mediated soil improvement techniques were evaluated. The major conclusion from this study may be encapsulated as following

- *Bacillus megaterium* (NCIM 5472) is observed to have significant potential as a bio-mediator for soil improvement with a moderate urease activity of 247 U/ml. *Bacillus megaterium* also yields a significant amount of calcite precipitation potential (509 mg/100 ml of equimolar cementation solution) for bio-cementation purposes.
- The FESEM and XRD of treated sand reveal the presence of bonding in forms of precipitated calcite, which indicates that the *B. megaterium* can be used effectively for bio-mediated soil improvement.

The bio-mediated ground improvement certainly holds possibilities as eco-friendly ground improvement technique; however, to utilize the process for engineering applications, in-depth exploration, and the limitations such as ammonia generation must be addressed. This study is a primary study on the potential of *B. megaterium*-mediated soil improvement and its potential for engineering the hydraulic conductivity behavior of the soil. Further exploration is required to study the shear strength characteristics and the factors influencing the process. However, this study will provide significant insights for the characterization of *B. megaterium* as an alternative and potent urease-based bio-mediator for its utilization in the ground improvement techniques.

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Numerical Simulation of Liquefaction Mitigation by Using Grout Under Existing Building

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Abstract Ground improvement using jet grout columns is a well-known technique to mitigate liquefaction hazard in sand stratum under existing building. However, the performance of conventional jet grout reinforcement technique has not achieved the sufficient level yet in terms of reducing shear strains and excess pore water pressure generated within the liquefiable soil layer. Therefore, a new countermeasure method, using small diameter jet grout column with additional horizontal slab, is introduced to control the shear deformation and excess pore pressure more effectively. To determine the efficiency of the new countermeasure method, numerical studies on unimproved and improved ground were separately performed in this study. The effectiveness of jet grout column with horizontal slab was evaluated by comparing the changes in excess pore water pressure, acceleration as well as distribution of shear stress and shear strain in the liquefiable soil before and after improvement. The results showed that the new liquefaction mitigation method offers positive effect on control of excess pore water pressure and shear deformation.

Keywords Liquefaction prevention · Ground improvement · Jet grouting · Numerical simulation

1 Introduction

After two devastating earthquakes occurring in 1964: Alaska and Niigata earthquakes, liquefaction has become a well-known disaster induced by earthquake due to its destructive effects to infrastructures and human lives. From that time, the negative effects of liquefaction are frequently encountered around the world. As liquefaction is one of the major problems that causes settlement, lateral spreading

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as well as lateral displacement in liquefiable soil during earthquakes, researches and efforts are emphasized on the necessity of soil remediation against liquefaction.

The mechanisms to improve liquefiable soil resistance against liquefaction are basically done by densifying the surrounding soil, reducing the generation of excess pore water pressure, decreasing the shear stress and shear strain. Several ground improvement techniques based on aforementioned mechanisms, such as gravel drain method, sand compaction pile method, deep mixing method, and jet grouting method, have been developed for liquefaction mitigation. However, most of the commonly available ground improvement methods require the proposed area to be free from structures. And, methods that can be used under existing buildings are less readily available. Although permeation grouting and chemical grouting methods have been used at existing housing projects, they are not suitable to use in finer-grained soils due to the difficulty of the low hydraulic conductivity, as well as the high cost of this technology. In such case, shear reinforcement method, jet grouting is considered to be effective by reducing shear stress and shear strain in improved ground under existing structure [1].

In Japan, the grid-type deep mixing method was developed for liquefaction mitigation since 1990s, where the grid of stabilized column walls function has been used to restrict generation of excess pore pressure by confining the soil particle movement during earthquake, as shown in Fig. 1. The effect of this improvement method was first evaluated in the Hyogoken-Nanbu earthquake in 1995. Subsequently, many numerical analyses, physical model tests, and field tests have been conducted to investigate the behavior of the grid type, interaction between the improved ground and the surrounding ground, and performance of ground improvement (e.g., Kitazume [4]). According to good results from numerical analyses and filed tests, the grid form liquefaction mitigation technique has been frequently used in construction sites. For instance, this method was adopted to prevent future earthquake induced liquefaction damages in Urayasu City. Past experiences show that liquefaction remediation using cement deep mixing grid form reinforcement method can reduce liquefaction risk. Nevertheless, it is still unable to eliminate the risk completely because of its

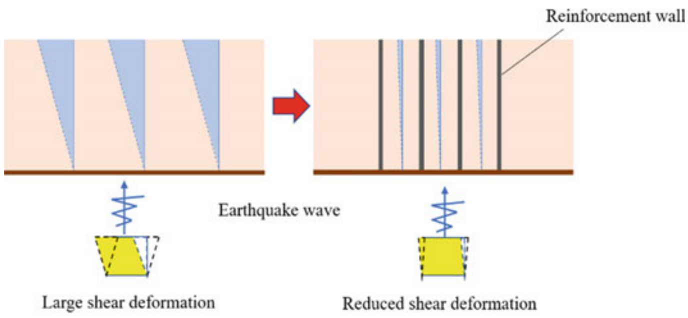


Fig. 1 Liquefaction mitigation mechanism of grid form wall

conventional design method. Moreover, jet grouting machines are very tremendous and difficult to deploy in city area.

To overcome these problems, a new countermeasure method, using small diameter jet grout columns enhanced with the additional horizontal slab, is introduced. This method is instructive compared to the conventional approach because it uses closely spaced jet grout wall ($L/H = 0.2$) to defend against liquefaction. In addition, the entire liquefaction-prone layer is improved by confining with contiguous jet grout columns in both vertical and horizontal directions to restrain the shear deformation of structure during an earthquake. Based on the previous researches together with observations and experiences in the fields, it is observed that the increase in the improvement area ratio is particularly effective in increasing the potential of the improved ground for liquefaction mitigation [6]. Furthermore, the outcomes of the results performed by research group of the Port and Airport Research Institute indicated that the pore water pressure generation and seismic response of shear stress and shear strain distribution in a sand layer are highly influenced by grid spacing [8]. This paper presents the findings of the effectiveness of closely spaced jet grout wall with horizontal slab in reduction of liquefaction risk based on PLAXIS 2D numerical analyses, by comparing the changes in excess pore water pressure, acceleration as well as distribution of shear stress and shear strain in the liquefiable soil before and after improvement.

2 Numerical Modeling

In this study, numerical simulations, using PLAXIS two-dimensional software, were performed to measure the effect of jet grouting in liquefaction mitigation. In order to gauge the behavior and performance of high modulus jet grout columns in liquefiable soil, numerical cases with and without soil improvement were separately evaluated.

2.1 Geometry Model and Boundary Conditions

As for soil profile used in numerical model, an idealized three layers of soil column was utilized. The water table was assumed to be coincident with the ground level. The effectiveness of ground improvement was measured at the 10-m-thick sand layer with $D_r = 50\%$, which is overlying 25 m of clay. The underlain layer was assumed to be a bedrock where the earthquake data of the Loma Prieta earthquake (1989) was imposed with the maximum peak ground acceleration of 0.3 g. The aforementioned earthquake data was recorded at the outcrop of a rock formation and characterized by a magnitude Mw of 6.9. The history of acceleration time was depicted in Fig. 2.

In these numerical analyses, 15 nodes triangular plain-strain elements were used to create a mesh distribution. Fine mesh option was used in the numerical analyses to meet the minimum required finite element length as suggested by Kulemeyer and

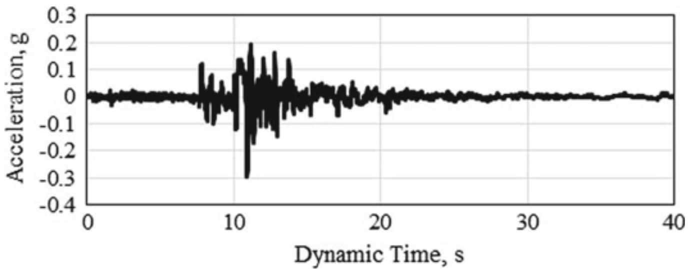


Fig. 2 Seismic input motion applied in the analyses

Lysmer [5]. The horizontal dimension of the soil profile was chosen to be large enough to minimize the boundary condition effect. Default fixities were applied for the static stages. In the dynamic phase, the vertical boundaries were modeled with tied-degree of freedom while compliance based was selected at the base as suggested in the site response and liquefaction evaluation by Brinkgreve [2]. To define the Rayleigh damping coefficients, damping ratios and related frequencies were considered based on the proposed method by Hudson et al. [3].

2.2 Parameters and Constitutive Models

In this study, an effective stress model of UBC3D-PLM was used to measure the development of excess pore water pressure and capture the onset of liquefaction in loose sand under dynamic loading. The liquefaction model, UBC3D-PLM, is a 3D generalized extension of the UBCSAND model and it was developed by Tsegaye [10] and Petalas and Galavi [7]. In the model, the primary and secondary yield surfaces are utilized to account for the effect of soil densification and predict the smooth transition into the liquefaction state under undrained loading. Additionally, a soil densification rule, f_{dens}, is implemented to better predict the evolution of pore pressures during cyclic loading.

Even though UBC3D-PLM is an advanced model, it is relatively simple to apply due to its reasonable number of parameters that can be extracted from laboratory or in situ tests. The input liquefaction parameters are derived based on the corrected clean sand equivalent SPT blow-count number $(N_1)_{60}$. However, the selection and calibration of parameters play a significant role to obtain reliable results. Hence, the calibrations for the parameters used in UBC3D-PLM model were conducted prior to the analysis. As depicted in Fig. 3, the liquefaction parameters were calibrated by fitting with cyclic stress ratio curve reproduced by means of cyclic direct simple shear test implemented in soil test facility of PLAXIS 2D and the experimental data of cyclic loading test on Toyoura sand, DR = 50% as published by Toki et al. [9]. The results show that the PLAXIS UBC3D-PLM model can give a good agreement

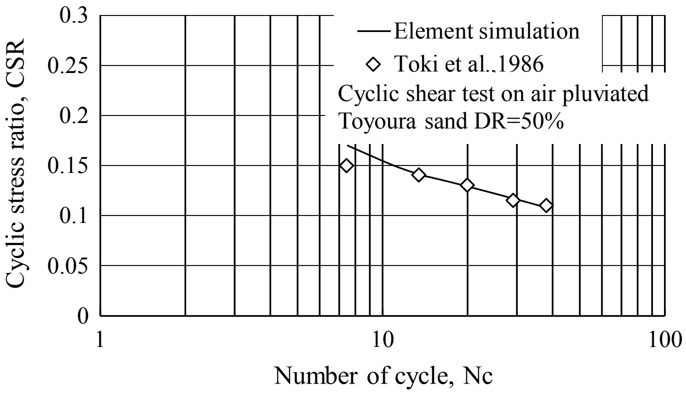


Fig. 3 CSR ratio obtained from numerical and experimental results

with the experimental results and prove the liquefied state of the soil. The properties of soil parameters and numerical models for each soil layer are shown in Table 1.

However, the UBC3D-PLM model is not advisable to use in static analysis since the parameters used in model are designated to evaluate liquefaction in loose soils and suitable only for dynamic calculation (PLAXIS 2D Material Models Manual, 2018). Therefore, hardening soil model was used in initial static phase to generate the stress state correctly for liquefiable soil prior to the dynamic phase. Furthermore, hardening soil model was also applied in clay layer to simulate the behavior of the stress-dependent stiffness and cyclic subjected to earthquake loading. The underlain bed rock was modeled as linear elastic model. The properties of soil parameters used in hardening soil model and linear elastic model were tabulated in Tables 2 and 3.

Table 1 Parameters of liquefiable soil used in the UBC3D-PLM model

Parameter	Symbol	Value	Method/formula
Peak friction angle	ϕ'_{p}	35°	CD test
Constant volume friction angle	ϕ'_{cv}	33°	CD test
Elastic shear modulus number	$K_{\mathrm{G}}^{\mathrm{e}}$	967.67	$21.7 \times 20 \times (N_1)_{60}^{0.333}$
Elastic bulk modulus number	$K_{\mathrm{B}}^{\mathrm{e}}$	677.37	$0.7 \times k_G^e$
Plastic shear modulus number	$K_{\mathrm{G}}^{\mathrm{p}}$	458.40	$k_G^e \times (N_1)_{60}^2 \times 0.003 + 100$
Elastic shear modulus index	m_{e}	0.5	Default
Elastic bulk modulus index	n_{e}	0.5	Default
Plastic shear modulus index	n_{p}	0.4	Default
Failure ratio	R_{f}	0.77	$1.1 \times (N_1)_{60}^{-0.15}$
Densification factor	f_{dens}	0.45	Curve fitting
Post liquefaction factor	f_{post}	0.02	Curve fitting
Corrected standard penetration test	$(N_1)_{60}$	11.1	$\mathrm{DR}^2/15^2$

Table 2 Parameters used in hardening soil model

Parameter	Symbol	(Unit)	Clay	Sand
Unit weight	γ_{sat}	(kN/m^3)	18	20
Effective cohesion	c'_{ref}	(kN/m^2)	13	0
Effective friction angle	ϕ'	(°)	22	35
Dilatancy angle	Ψ	(°)	–	1
Secant modulus	E_{50}^{ref}	(kN/m^2)	5436	20,380
Tangent stiffness for primary oedometer loading	E_{oed}^{ref}	(kN/m^2)	5436	20,380
Unloading/reloading stiffness	E_{ur}^{ref}	(kN/m^2)	16,310	61,130
Power of stress-level decency	m		0.8	0.5

Table 3 Parameters of bedrock used in linear elastic model

Parameter	Symbol	(Unit)	Values
Unit weight	γ_{sat}	(kN/m^3)	22
Young's modulus	E	(MN/m^2)	6000
Poisson's ratio	υ		0.2

On the other hand, the 0.6 m diameter of jet grout columns (10 m long) with closely grid spacing ($L/H = 0.2$) was installed for improved ground case. Moreover, 1 m thickness of horizontal slabs was added at every 1 m interval as shown in Fig. 4. The loose sand layer was improved in both vertical and horizontal directions to restrain the shear deformation during the earthquake. Mohr–Coulomb model was applied in jet grout columns modeling. Elasticity modulus, Poisson's ratio, and undrained shear strength of the columns were selected as 1000 MPa, 0.2, and 1 MPa, respectively.

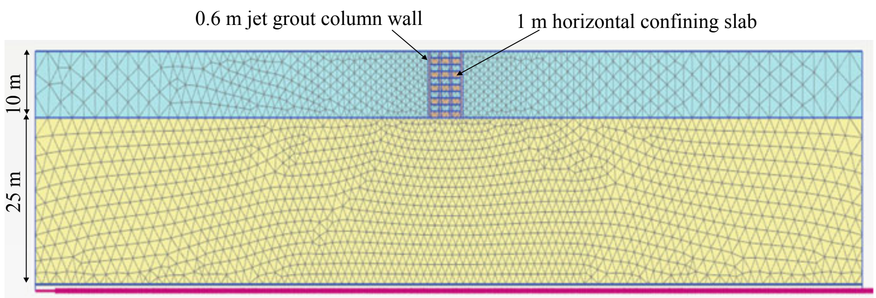


Fig. 4 Finite element model of the improved case

3 Result and Discussion

The aim of the study is to evaluate the effect of liquefaction mitigation by using closely spaced jet grout columns with the horizontal slab in liquefiable ground. Thus, numerical cases of soil improvement with and without were separately measured. The analysis results were evaluated based on the effect of jet grout columns on the distribution excess pore water pressure ratio, acceleration, shear stress, and shear strain in the liquefiable soil layer.

3.1 Excess Pore Water Pressure Ratio

Excess pore water pressure ratio is an indicator of liquefaction occurrence, which can be calculated by means of the ratio of the excesss pore water pressure and initial effective vertical stress at the depth. In the UBC3D–PLM model, the excess pore water pressure ratio was measured by vertical effective stress at the end of the dynamic calculation and initial effective vertical stress prior to the dynamic stage [2]. The corresponding layer can be determined as a complete liquefied layer when the excess pore water pressure ratio is 1. The comparison results of excess pore water pressure distribution with dynamic time at different depths for unimproved and improved cases were stated in Fig. 5. Based on the comparison results, it can be observed that, the proposed liquefaction countermeasure method is effective in excess pore water pressure control, because the excess pore water pressure ratio in the improved ground case did not increased into 1.0 until the end of the seismic loading. On the other hand, liquefaction occurred in the original unimproved case, where excess pore water pressure ratio is increased into 1.0 after the dynamic time 13 s.

3.2 Acceleration

Acceleration time histories of the unimproved case and improved case at different depths were described in Fig. 6. Some noticeable differences in the behavior were identified when the acceleration time histories of the unimproved and improved cases were compared. The peak accelerations recorded at the surface of improved case and unimproved case are 0.24 and 0.18 g, respectively. Besides, soil acceleration attenuation behaviors occurred in all acceleration time histories measured at different depths of the unimproved case. That is, the soil acceleration appeared to decrease upon the onset of liquefaction (around dynamic time at 13 s) due to the reduction of soil strength and stiffness and increase in hysteretic damping. However, there was no trace of acceleration decreasing due to liquefaction encountered in improved case.

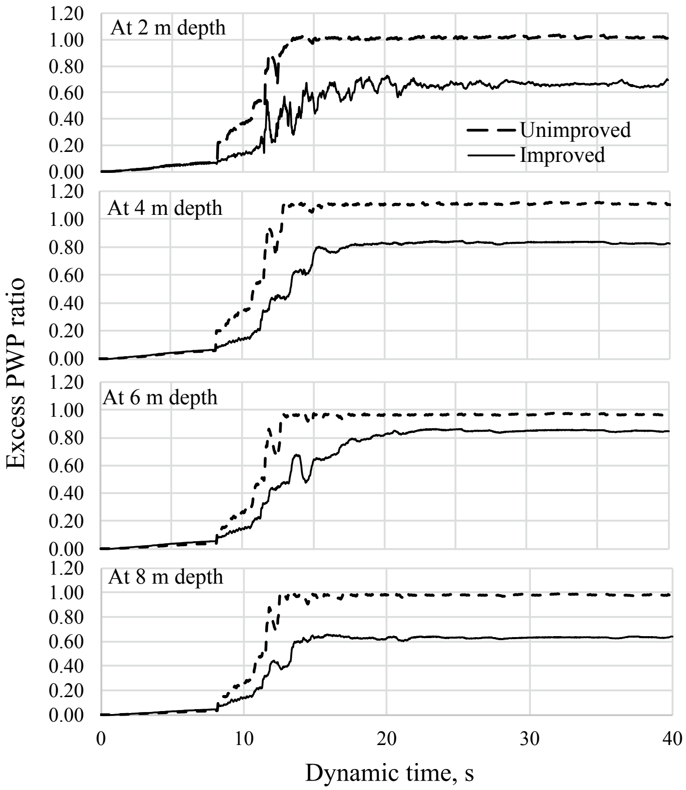


Fig. 5 Comparison result of PWP ratio measured from unimproved and improved cases

In other words, the newly proposed liquefaction countermeasure method can control the soil stiffness reduction against seismic loading.

3.3 Shear Stress and Shear Strain Distribution

In order to investigate the improvement effect of the proposed liquefaction countermeasure method, the response of shear stress and shear strain of the liquefiable soil inside the grid form was evaluated. Figure 7 shows the results of shear stress distributed in unimproved and improved cases. Like in acceleration time histories, a similar characteristic of liquefaction identification was observed in unimproved case. Significant stress degradation has occurred upon the onset of liquefaction at dynamic time 13 s and then vanishes until the earthquake ends. Nevertheless, the aforementioned stress degradation due to liquefaction was not observed in the improved case.

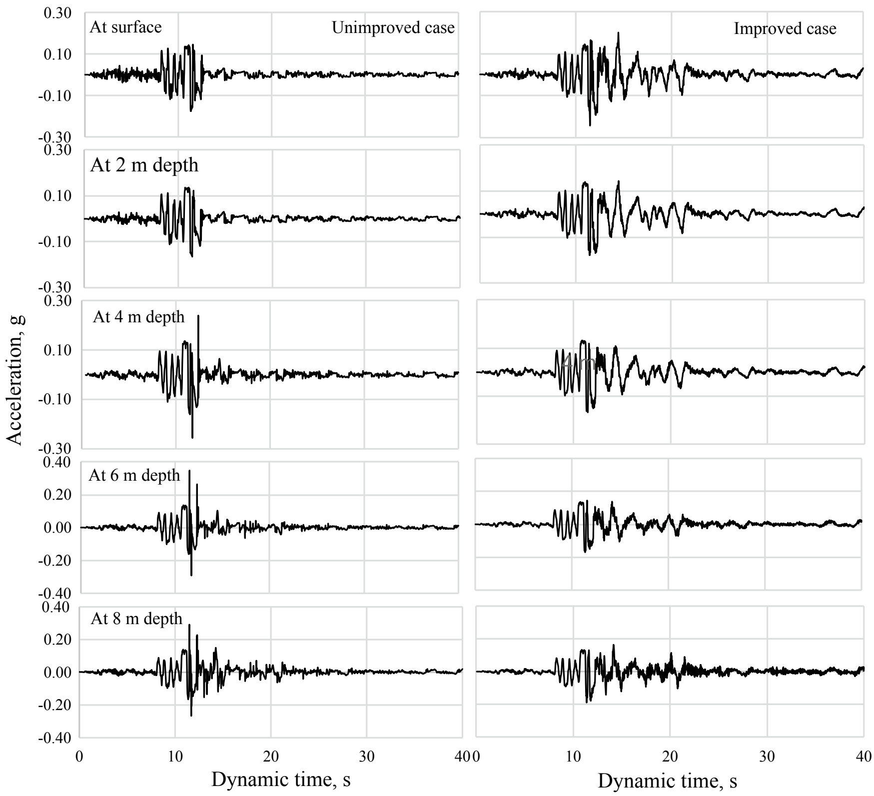


Fig. 6 Acceleration time histories measured from unimproved and improved cases

Figure 8 depicts the comparison results of the shear strain variations with depth after the earthquake measured from unimproved and improved cases. As seen from figure, the shear strains tend to increase linearly with depth from the ground surface in both unimproved and improved cases. However, the range of shear strain distribution from the ground surface to the depth is wide as approximately 0.01–3.5% in the unimproved case. In contrast, the slightly distribution of shear strain along the entire depth was observed in improved case. It can be determined that the loose soil layer was effectively controlled from shear deformation during the earthquake as the vertical and horizontal reinforcement. Moreover, the effect of liquefaction mitigation due to soil improvement method, the comparison results of the relationships of shear stress and shear strain and shear stress and vertical effective stress relationships at different depths were described in Fig. 9. Significant reduction of shear stress was not observed compared to the shear strain. Figure 9b shows the vertical effective stress at measured depth is gradually reducing to zero as per increasing of excess pore water with dynamic time in unimproved case. However, the behavior of vertical effective stress reductions was not found since excess pore water was effectively controlled

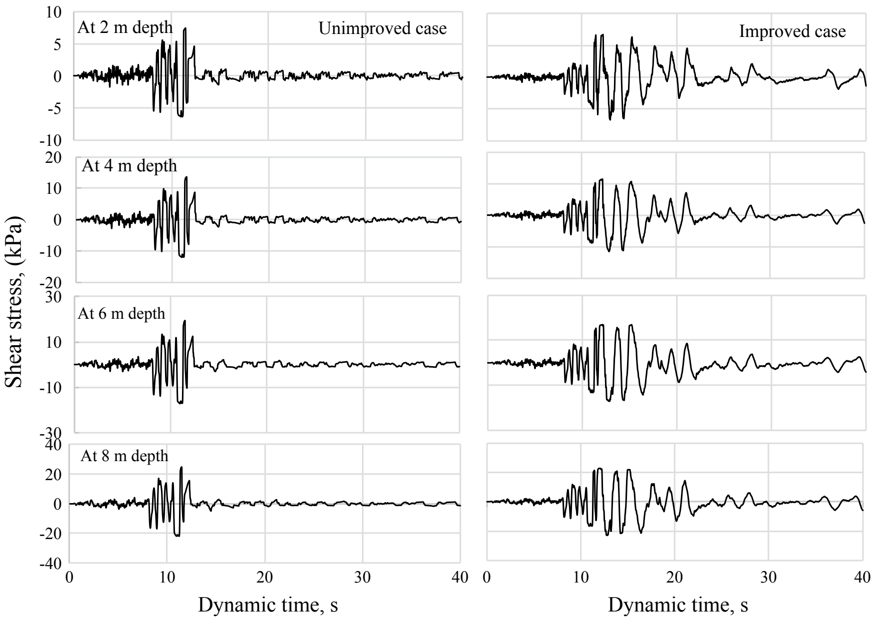
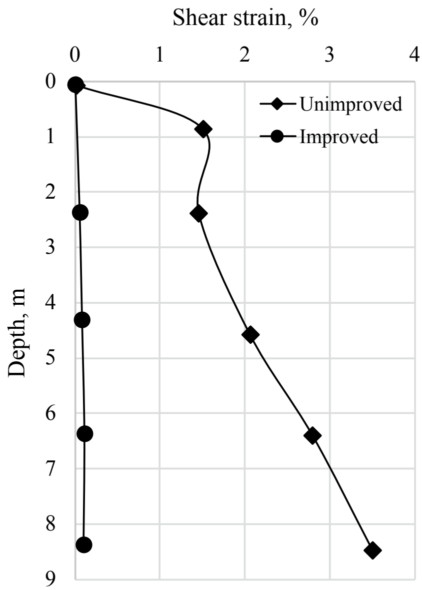


Fig. 7 Distribution of shear stress in unimproved and improved cases

Fig. 8 Distribution of shear strain in unimproved and improved cases with depth



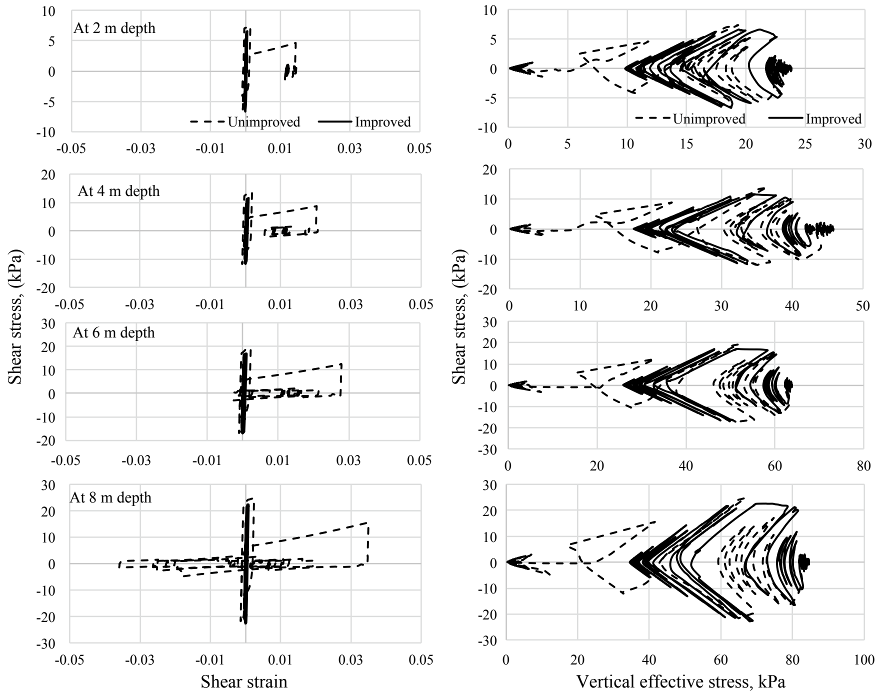


Fig. 9 Variation of shear stress and strain in unimproved and improved soil **a** shear stress and shear strain relationship, **b** shear stress and vertical effective stress relationship

by the improved method. Generally, it can be said that excess pore water pressure and shear strain in the liquefiable soil was effectively restricted by jet grout columns with horizontal slab soil improvement method.

4 Conclusion

Numerical simulations for unimproved and improved ground conditions were conducted to assess the effectiveness of the liquefaction mitigation. As an initial assessment for liquefaction prevention, the study on the effectiveness of the close spacing of jet grout contiguous columns with horizontal confining method was performed. The following findings were deduced based on the comparison results of unimproved and improved cases.

1. Liquefaction did not take place in the improved case since the excess pore water pressure ratio did not reach to 1.0. On the other hand, liquefaction was found to occur in original unimproved case, where excess pore water pressure ratio is increased to 1.0 after the dynamic time 13 s.

2. Regarding the soil acceleration, acceleration attenuation due to soil strength and stiffness reduction did not occur in the improved case, while significant decrease in acceleration was observed upon the onset of liquefaction in unimproved case.
3. Based on the shear stress distribution, the characteristics of shear stress degradation were not observed in improved ground case, whereas the occurrence of significant stress reduction took place in the unimproved case.
4. Shear strain variations with depth were observed in both unimproved and improved cases. However, the range of shear strain distribution from the ground surface to the depth of interest is obviously larger in unimproved case compared to improved case. Thus, the close spacing of jet grout contiguous columns with horizontal slab can effectively control the shear deformation of improved ground layer during the earthquake.
5. If vertical effective stress is concerned, the effect of liquefaction mitigation can be clearly seen in improved ground case. Because, the vertical effective stress at measured depth was gradually reducing to zero as the excess pore water pressure increasing with dynamic time in unimproved case. Nonetheless, this behavior did not exhibit in improved case since the excess pore water pressure was effectively controlled by the jet grout column with horizontal slab liquefaction mitigation method.

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A Study on the Effect of Phosphogypsum on the Properties of Subgrade Soil Mixed with Fly Ash

Tanmoy Maity and Sanjay Paul

Abstract The present study is an attempt to enlighten the direction of utilization of industrial wastage materials such as phosphogypsum and fly ash, by mixing them with subgrade soil to improve the load-bearing capacity of silty-sandy soil. With the increasing urbanization, industrial growth and installation of various plants generate a huge volume of wastes. The soil subgrade plays a pivoted role in the load-carrying capacity in both flexible and rigid pavement. For the laboratory investigation, the silty-sandy soil samples have been collected from Rabindranath Tagore Hostel inside the N.I.T. Agartala campus, Tripura, India. Each soil sample has been collected from the depth of 3 m below the natural ground surface. The fly ash sample has been collected from Kolaghat Thermal Power Plant, West Bengal, India. Different tests were performed as per the requirements of Indian Standard Codes to know the compaction properties, unconfined compressive strength, and C.B.R. values of the original soils and the soils treated with fly ash and phosphogypsum. It is observed from the test results that, for soil blended with fly ash and phosphogypsum, the values of C.B.R. and unconfined compressive strengths are significantly higher compared to that of untreated natural soil and soil mixed with phosphogypsum. From the tests, it can also be concluded that soil content with 30% fly ash and 6% phosphogypsum can be used as a subgrade material. Also, the thickness of the subgrade layer can be reduced as the subgrade is good. So the construction cost of the pavement may be reduced.

Keywords Subgrade Soil · Fly ash · California bearing ratio (c.b.r.) · Phosphogypsum (PG)

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1 Introduction

Soil stabilization or improvement, in a broad sense, includes the various techniques employed for modifying the properties of soil to improve its engineering performances. It is being used for many of construction and engineering works where the main objective is to increase the strength or stability of soil and to reduce the construction cost by making best use of the locally available materials. In the present-day world, Engineers usually face a series of potential soil problems due to the bad soil layers. In such cases, the improvement of the strength behavior of the soil stabilized with various materials may be an excellent solution. For the improvement of the strength behavior of soil, several materials may be used as an additive, such as cement, lime, and also some industrial waste products for example, fly ash, and phosphogypsum. The use of those industrial waste products as a material to stabilizing the soil may help economically and environmentally to a great extent.

Several studies have been carried out by number of researchers [1, 5–8] on the improvement of engineering properties of soil by adding fly ash, phosphogypsum, etc.

2 Objective of the Present Study

The strength parameter of soil is one of the important geotechnical propertied used in the various field. The present study is based on the strength behavior of soil mixed with fly ash to understand the effect of fly ash on different parameters of the soil. Subsequently, it has also been studied the effect of phosphogypsum on the optimum proportion of soil-fly ash mixture. The compaction characteristics, strength parameters, and the deformation of the soil under different loading are studied by mixing soil-fly ash-phosphogypsum samples.

3 Materials and Methodology

3.1 Materials

The soil samples have been collected from newly constructed Rabindranath Tagore Hostel inside the N.I.T. Agartala campus, Tripura, India. The soil samples are extracted 3 m below the natural ground surface. After collection of the soils, these are oven-dried and pulverized. Fly ash was collected from Kolaghat Thermal Power Plant, West Bengal, India. Raw Phosphogypsum is collected from the stockpile of a fertilizer and chemical production unit, Odisha, India. The different physical and engineering properties of the soil, fly ash, and phosphogypsum samples have been tabulated in Tables 1, 2, and 3, respectively.

Table 1 Physical and engineering properties of the silty-sandy soil sample

Properties	Experimental results
Specific gravity	2.60
Sand (4.75–0.075 mm) (%)	53.28
Silt (0.075–0.002 mm) (%)	35.32
Clay (<0.002 mm) (%)	11.40
Soil group symbol	*SW–SM*
Maximum dry density (gm/cc) [Heavy compaction]	2.00
Optimum moisture content (%)[Heavy compaction]	11.80

Table 2 Physical and engineering properties of the fly ash sample

Properties	Experimental results
Specific gravity	1.98
Sand (4.75–0.075 mm) (%)	16.25
Silt (0.075–0.002 mm) (%)	83.65
Clay (<0.002 mm) (%)	6.10
Soil group symbol	*SM*
Plasticity index (%)	Non-plastic
Maximum dry density (gm/cc) [Light compaction]	2.00
Optimum moisture content (%) [Light compaction]	11.80

Table 3 Chemical compositions in phosphogypsum sample

Properties	P_2O_5	Na_2O	K_2O	Al_2O_3	Fe_2O_3	F	SO_3	Organic compound	pH value
Compositions (%)	0.820	0.051	0.009	0.170	0.139	0.012	45.08	2.36	3.66

Phosphogypsum is a gray colored, damp, fine-grained powder, silty material. The average specific gravity of phosphogypsum is 2.45.

3.2 Methodology

The strength behavior of the soils has been studied by mixing the fly ash and phosphogypsum in different proportions with the soil. The fly ash mix proportions in the present study are 10, 20, 30, and 40% by dry weight of the soil, and phosphogypsum samples have been mixed at 3, 6, 9, and 12% by dry weight with 30% of fly ash mixed soil samples. The mixed proportions have been summarized in Table 4.

Table 4 Different proportions of soil, fly ash, and phosphogypsum samples

Soil mixtures	Description
Soil (*S*)	Soil sample only
Soil (*S*)-fly ash (FA) mix	Soil mixed with 10% fly ash by dry weight
	Soil mixed with 20% fly ash by dry weight
	Soil mixed with 30% fly ash by dry weight
	Soil mixed with 40% fly ash by dry weight
Soil (*S*)-fly ash (FA)-phosphogypsum (PG) mix	Soil mixed with 30% fly ash and 3% PG by dry weight
	Soil mixed with 30% fly ash and 6% PG by dry weight
	Soil mixed with 30% fly ash and 9% PG by dry weight
	Soil mixed with 30% fly ash and 12% PG by dry weight

4 Results and Discussions

For evaluating the strength behavior of soil different tests have been conducted as per [2–4].

4.1 *Effect on Compaction Tests of Soil, Soil-Fly Ash Mixture, and Soil-Fly Ash-Phosphogypsum Mixture*

The heavy compaction tests have been conducted to determine the maximum dry density (MDD) and the optimum moisture content (OMC) of the soil, soil-fly ash, and soil-fly ash-phosphogypsum samples. The values of MDD and OMC obtained by adding soil and fly ash are summarized in Table 5, from which it is observed that with the increase in fly ash content in soil, the maximum dry density of the soil-fly ash mixture decreases and optimum moisture content increases. It is due to the presence of hollow particle in fly ash, which increases the optimum moisture content. The fly ash has major amount of silt size particles, whereas the soil has large

Table 5 Variation of maximum dry density (MDD) and optimum moisture content (OMC) with soil and soil-fly ash (FA) mixture by the modified compaction test

FA (%)	0	10	20	30	40
MDD (gm/cc)	2.00	1.93	1.89	1.83	1.71
OMC (%)	11.8	12.6	13.4	14.0	14.7

Table 6 Variation of maximum dry density (MDD) and optimum moisture content (OMC) with soil, soil-fly ash (FA), and soil-fly ash-phosphogypsum (PG) mixture by the modified compaction test

FA (%)	30	30	30	30	30
PG (%)	0	3	6	9	12
MDD (gm/cc)	2.00	2.10	2.21	2.08	1.96
OMC (%)	11.80	11.40	11.17	12.61	13.31

amount of sand particles. As a result, in soil-fly ash mixed samples the amount of sand content decreases with increase in fly ash content and thereby the maximum dry density decreases.

The heavy compaction tests have also been conducted considering samples with soil +30% fly ash and 3%, 6%, 9%, and 12% of phosphogypsum content, respectively. The values of MDD and OMC are summarized in Table 6. It has been observed that, for fly ash content 30% and phosphogypsum content of 6%, the MDD value is maximum and OMC value is minimum (Figs. 1 and 2).

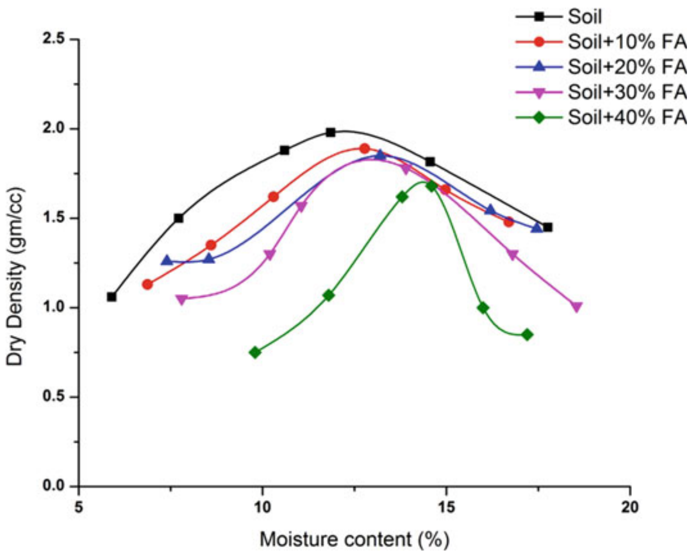

Fig. 1 Variation of dry density with moisture content of soil and soil-fly ash (FA) mixture in the modified compaction test

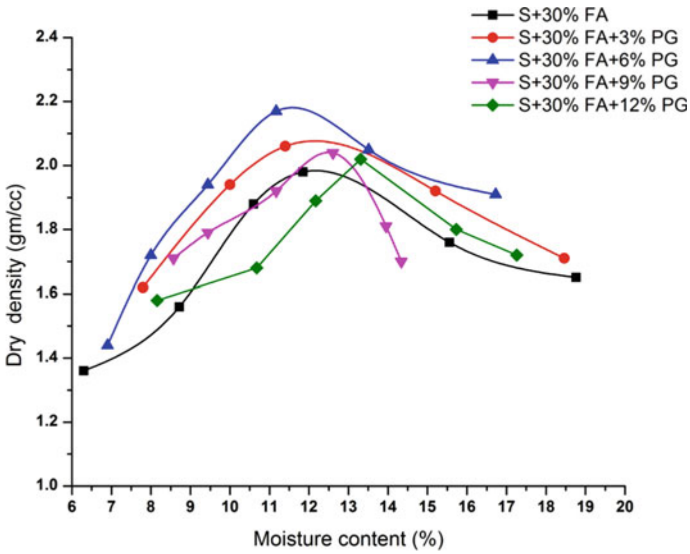

Fig. 2 Variation of dry density with moisture content of soil, soil-fly ash (FA) and soil-fly ash-phosphogypsum (PG) mixture in the modified compaction test

4.2 Effect on California Bearing Ratio of Soil, Soil-Fly Ash Mixture, and Soil-Fly Ash-Phosphogypsum Mixture

The California bearing ratio (CBR) test on N.I.T. Agartala campus soil, soil mixed with fly ash, and soil-fly ash-phosphogypsum mixtures has been conducted in laboratory. The results of CBR value of soil-fly ash and soil-fly ash-phosphogypsum mixture with different proportion of fly ash and phosphogypsum are shown in Figs. 3 and 4 and also in Tables 7 and 8.

It has been observed from Table 7 that the soil mixed with 30% of fly ash content has maximum CBR value in both cases (both soaked and un-soaked). It is due to the large sand content in soil as compared to silt and clay, and in case of fly ash, major amount of particles are of silt size. Thus, the voids between sand particles are filled with these silt size particles of fly ash. During the cat-ion exchange process in the soil-fly ash mix, the sodium ions in the soil are replaced by the calcium ions of fly ash thus reduces the settlement and hence increases the CBR value. Both soaked and un-soaked CBR value follow a similar trend, but soaked CBR value is lower as compared to un-socked CBR value.

In case of soil-fly ash-phosphogypsum mixture (Table 8), it is observed that the CBR value increases significantly with increase in PG content up to 6%, after that the CBR value decreases with the increase in PG content. It is observed that the maximum un-soaked and soaked CBR values are 18.78% and 13.83%, respectively, for the soil mixed with 30% fly ash and 6% PG content. Whereas the un-soaked and soaked CBR percentages for normal soil sample are 6.3 and 4.65% (Table 7). The

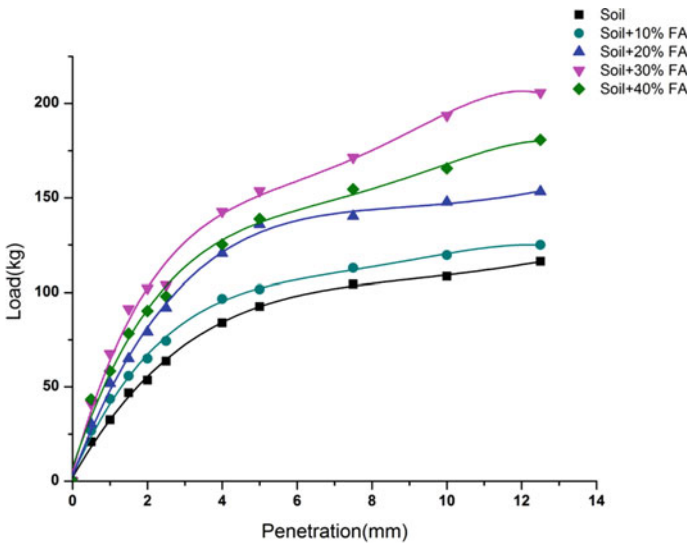

Fig. 3 Load versus penetration graph for soil-fly ash (FA) mixture in soaked condition to find the CBR values

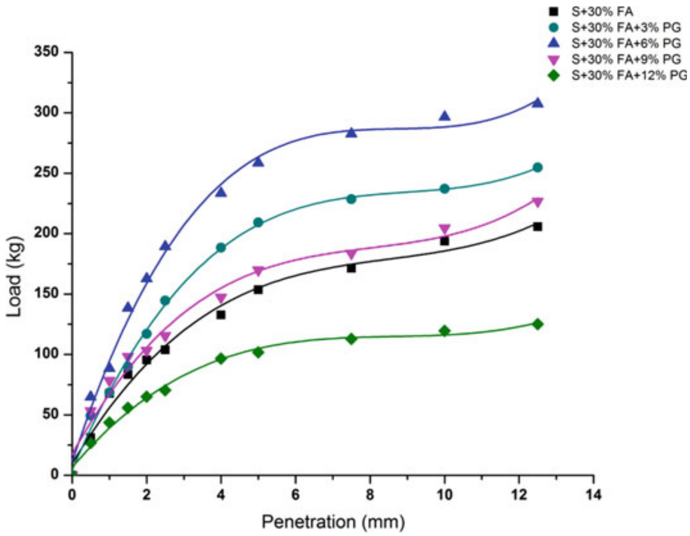

Fig. 4 Load versus penetration graph for soil-fly ash (FA)-phosphogypsum (PG) mixture in soaked condition to find the CBR values

Table 7 Variation of California bearing ratio (CBR) with soil and soil-fly ash (FA) mixture under soaked and un-soaked conditions

FA (%)	0	10	20	30	40
Un-soaked CBR (%)	6.30	7.07	7.97	9.51	9.13
Soaked CBR (%)	4.65	5.43	6.70	7.58	7.09

Table 8 Variation of California bearing ratio (CBR) with soil, soil-fly ash (FA), and soil-fly ash (FA)-phosphogypsum (PG) mixture under soaked and un-soaked conditions

FA (%)	30	30	30	30	30
PG (%)	0	3	6	9	12
Un-soaked CBR (%)	9.51	13.37	18.78	11.70	8.23
Soaked CBR (%)	7.58	10.56	13.83	8.43	5.14

gain in strength of soil-fly ash-phosphogypsum mix is primarily a result of pozzolanic reaction between fly ash and PG, due to the formation of various types of cementing compound.

4.3 *Effect on Unconfined Compressive Strength of Soil, Soil-Fly Ash Mixture, and Soil-Fly Ash-Phosphogypsum Mixture*

The unconfined compressive strength test on soil, soil mixed with fly ash, and soil-fly ash-phosphogypsum mixture has been performed in the laboratory. The typical stress versus strain curves for soil, soil-fly ash, and soil-fly ash-phosphogypsum mixtures have been shown in Figs. 5 and 6.

The results from the UCS test on the soil and soil mixed with fly ash are summarized in Table 9. From the table, it is observed that unconfined compressive strength of the soil increases with the increase in fly ash content up to 30% after that the UCS value decreases with the further increment of fly ash content. The reason behind UCS value increment is, during cat-ion exchange in the soil-fly ash mix, sodium ions in the soil are replaced by the calcium ions and formation of bonding.

From the UCS test on the soil mixed with 30% fly ash and different percentages of phosphogypsum are presented in Table 10. From the table, it is observed that unconfined compressive strength of the stabilized soil samples increases with the increase in phosphogypsum content up to 6% after that the UCS value decreases with the further increment of phosphogypsum content. The reason behind UCS value increment is due to the potential strength increment of stabilized samples.

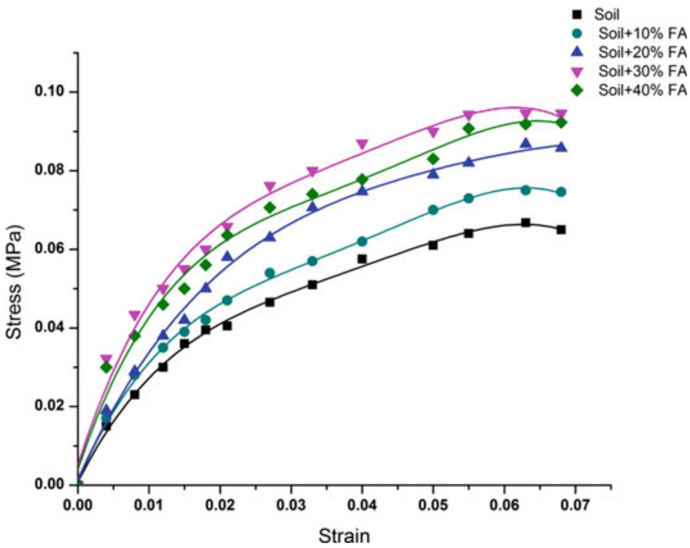

Fig. 5 Unconfined compressive strength curves of soil and soil-fly ash mixture

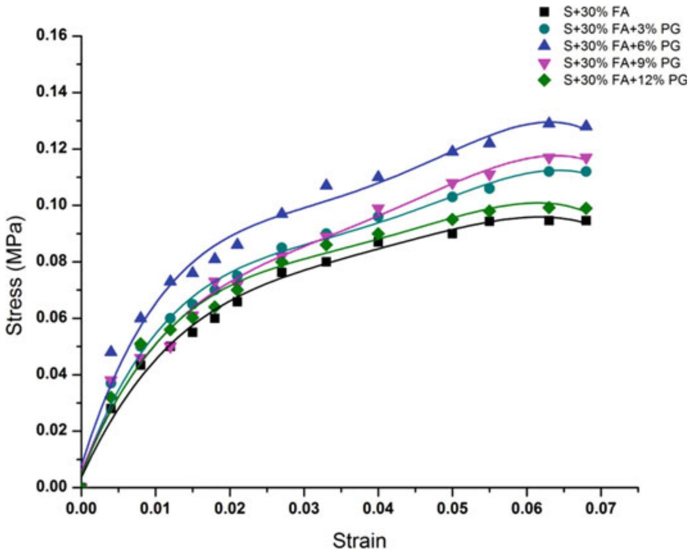

Fig. 6 Unconfined compressive strength curve of soil-fly ash mixtures and soil + 30% FA + different percentages of phosphogypsum mix samples

Table 9 Variation of unconfined compressive strength (UCS) with soil and soil-fly ash (FA) mixture

FA (%)	0	10	20	30	40
UCS (MPa)	0.067	0.075	0.088	0.095	0.093

Table 10 Variation of unconfined compressive strength (UCS) with soil and soil-fly ash (FA)-phosphogypsum (PG) mixture

FA (%)	30	30	30	30	30
PG (%)	0	3	6	9	12
UCS (MPa)	0.095	0.112	0.129	0.117	0.099

5 Conclusions

Based on the above results, the following conclusions may be made:

- With the increase in fly ash content in soil, the maximum dry density of the soil-fly ash mixture decreases and optimum moisture content increases due to the presence of hollow particle in fly ash.
- For an optimum proportion of fly ash and phosphogypsum content in soil, the maximum dry density value is maximum and optimum moisture content value is minimum.
- The CBR value of soil-fly ash mixture is maximum at an optimum value of fly ash content under both un-soaked and soaked conditions of CBR determination. Also, the soaked CBR values are lower as compared to the un-soaked CBR values.
- The CBR value of soil-fly ash-phosphogypsum mixture is maximum at an optimum value of fly ash and phosphogypsum content under both un-soaked and soaked conditions of CBR determination. Also, the soaked CBR values are lower as compared to the un-soaked CBR values.
- The unconfined compressive strength of soil-fly ash mixture is maximum at an optimum value of fly ash content.
- The unconfined compressive strength of soil-fly ash-phosphogypsum mixture is maximum at an optimum value of fly ash and phosphogypsum content.
- The lattice planes are not closely placed and a sharp peak is observed, which shows that the voids are present and denser packing of lattice planes is possible. Buy for Soil + 30%FA + 6%PG sample, the lattice planes are closely placed, which shows that the voids are not more present in the mixed sample.
- Soil content with 30% fly ash and 6% phosphogypsum can be used as a subgrade material. Also, the thickness of the subgrade layer can be reduced as the subgrade is good. So the construction cost of the pavement may be reduced.

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Effect of Coir Fiber on Compressibility Behavior of Clayey Soil

K. S. Sajini and K. Niranjana

Abstract The significant high compressibility and low shear strength of clayey soil impose challenges to the civil engineers. Use of coconut coir fiber for improving soil property was advantageous because they are cheap, locally available and eco-friendly and used as a reinforcement material. Coconut fiber is a natural fiber extracted from the husk of a coconut. This paper focuses on the experimental investigation on the effect of coir fiber on the compressibility characteristics and permeability behavior of the clayey soil. Coir fiber was added in different percentages (viz 0.3%, 0.4%, 0.5%, 0.6%, 0.7% and 0.8%) to the soil sample with 10 mm length, and the effect of coir fiber in compressibility and permeability characteristics of the soil was studied. Results show that compressibility decreased and the coefficient of consolidation increased on the addition of the coir fiber. Coir fiber of different percentage is used for the study, and optimum moisture content and maximum dry density were used for preparing the soil sample. An optimum percentage of coir fiber for the enhanced properties of soil was found out, and also, the coefficient of consolidation, compression index is found out from consolidation test with different loading rates. The compression index (c_c) decreases with the inclusion of coir fibers in the soil up to certain fiber content and increases thereafter.

Keywords Consolidation · Clayey soil · Coir fiber

1 Introduction

Consolidation characteristics as well as settlement characteristics are the most problematic topics in the field of geotechnical engineering. This property is directly related to the total settlement of any structure. Excess and non-uniform settlement

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of foundation soil may cause a high amount of imbalance moment in the entire joint of the existing structure, which may cause the failure of the structure. Therefore, it is required to understand the process properly which will eventually help to mitigate the problems related to the settlement. Consolidation settlement is the vertical displacement of the surface corresponding to the volume change at any stage of the consolidation process. Consolidation settlement will result if a structure is built over a layer of saturated clay or if a water table is lowered permanently in a stratum overlying a clay layer. On the other hand, if excavation is made in a saturated clay, heaving will result in the bottom of excavation due to the swelling of clay. In any field of engineering science, researchers have concentrated their studies on the development of new materials through the elaboration of composites. In the case of geotechnical engineering, the idea of inserting fibrous material in a soil mass to improve its consolidation behavior has been known since ancient times. That is some modifications can be done to the soils by providing reinforcements using locally available natural fibers (coir fiber). Reinforcing with such material in the soil will provide an easy drainage path so that more and more water will dissipate through the pores and improves the compressibility and permeability characteristics of the soil and reduces the time required to reach the primary consolidation [8].

2 Experimental Program

One-dimensional consolidation tests were conducted to evaluate the effect of coir fiber on the consolidation and permeability characteristics of the soil. Materials used for the study are illustrated below.

2.1 Materials Used

Soil. The clayey soil was collected from Erumapetty Thrissur District, Kerala. The geotechnical properties of clay are given in Table 1. The clayey soil is classified as high plastic clay (CH) according to IS Plasticity chart.

Coir fiber. The use of natural fibers was widely adopted in the ancient period due to the eco-friendly behavior of natural fiber. The use of coir fiber for the soil improvement is highly attractive in countries like India where such materials are locally and economically obtainable, in view of preservation of the natural environment and cost-effectiveness. The coir fiber is one of the hardest natural fibers available because of its high content of lignin and has low density. The coir fiber is collected from Irinjalakuda coir manufacturing factory, Thrissur District, Kerala. The physical properties of coir fiber from the manufactures manual are shown in Table 2 (Fig. 1).

Table 1 Geotechnical properties of soil

Properties	Clay
Specific gravity	2.76
Percentage of gravel (%)	5
Percentage of sand (%)	12
Percentage of fines (%)	83
Liquid limit (%)	52
Plastic limit (%)	25
Shrinkage limit (%)	16
Plasticity index (%)	27
Soil classification system	CH
Optimum moisture content (%)	18
Maximum dry density (kN/m^3)	16.1

Table 2 Physical properties of coir fiber (manufactures manual Irinjalakuda, Thrissur, Kerala)

Properties	Values
Density (g/cc)	0.6–1.4
Elastic modulus (N/m^2)	3.79×10^9
Tensile strength (N/m^2)	165×10^6

(a) Clayey soil

(b) Coir fiber

Fig. 1 Materials used for the study

3 Sample Preparation

The collected natural clayey soil in the form of a wet condition placed in an oven for 24 h and then crushed into dry powder form suitable for the work. The light compaction tests were conducted to determine the optimum moisture content and maximum dry density of the soil sample. The measured quantities of soil sample and the corresponding quantity of water content are mixed together and placed in a desiccator to ensure that uniform distribution of moisture content within the soil mass.

The fibers were cut into an average length of 10 mm for preparing the sample. These fibers were added to the soil sample at different percentages varying from 0.4%, 0.5%, 0.6%, 0.7% and 0.8% at an increment of 0.1% of coir fibers. The soil samples were prepared by initial dry mixing of oven-dried soil and the corresponding quantity of fiber content (according to the percentage by weight of oven-dried soil). The optimum water content obtained from the compaction test was added gradually and mixed until the water spread all over the soil. The mix was used for the preparation of consolidation test specimens. The test was also conducted on unreinforced soil specimens. The variation in compressibility and permeability characteristics of unreinforced soil and soil reinforced with fiber content was compared.

4 Testing Program

The compaction test results were used for preparing the soil samples. The consolidometer of 20 mm thickness and 60 mm in diameter was used for conducting the test. The porous stones were saturated by immersing them into the water. After assembling the consolidometer, bottom porous stone, bottom filter paper, specimen and the top porous stone are placed one by one. The loading block is positioned centrally on the top porous stone. The assembly was mounted on the loading frame and centered such that the axial load was applied. The dial gauge was set in position, and the mold assembly was connected to the water reservoir. The water was allowed to flow into the specimen until it was fully saturated. A seating pressure of 25 kPa was applied, an initial load was set to give a pressure of 25 kPa, and the initial reading of the dial gauge was taken and after note the dial gauge readings at the time interval of 0, 0.25, 2.25,4, 6.25, 9, 12.25, 16, 20.25, 25, 36, 49, 64, 81, 100, 120, 180 and 240 min, and allow this load to 24 h. The load increment of 50, 100, 200 and 400 kPa was applied, and the same procedure was repeated. After completing the loading stage, the specimen preferably within the ring was weighed and thereafter placed in an oven for 24 h for drying. The oven-dried mass of the sample was taken (Fig. 2).

The compression dial gauge readings in thousands of mm are plotted against the square root of time in the pressure range of 200–400 kPa from which the time required for 90% consolidation (t_{90}) is found out which is used to find the coefficient of consolidation (c_v). Void ratio (e) versus log p curves are plotted from which the compression index (c_c) is found out as slope of straight line portion of e log p curve. From the values of void ratio, the coefficient of volume change (m_v) and coefficient of compressibility (a_v) are found out.

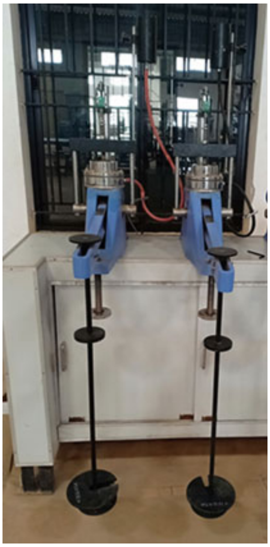

Fig. 2 Experimental setup of consolidation test

5 Results and Discussions

The variation of coefficient of consolidation with different loading rates is illustrated in Fig. 3. The void ratio–pressure relationship curves for both unreinforced and reinforced soils are given in Fig. 4. The values of compression index (c_c) were calculated from e log P curves, and the variation of compression index with varying percentage of fiber is shown in Fig. 5.

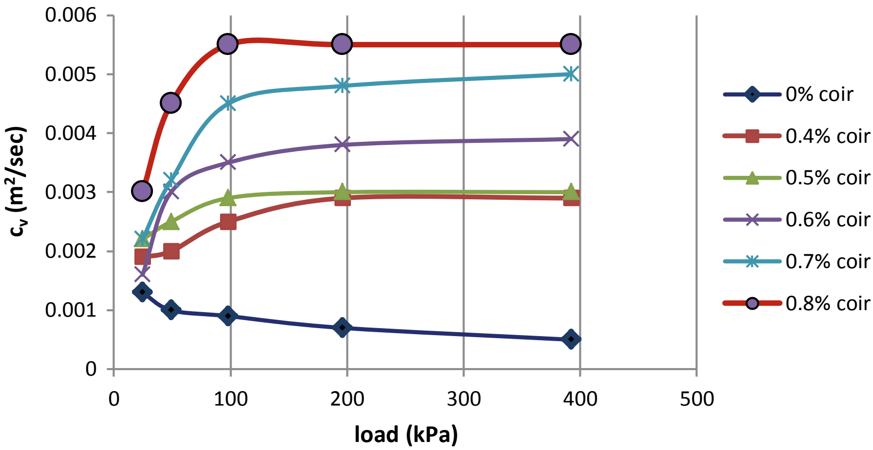

Fig. 3 Variation of the coefficient of consolidation with different percentage of coir fiber

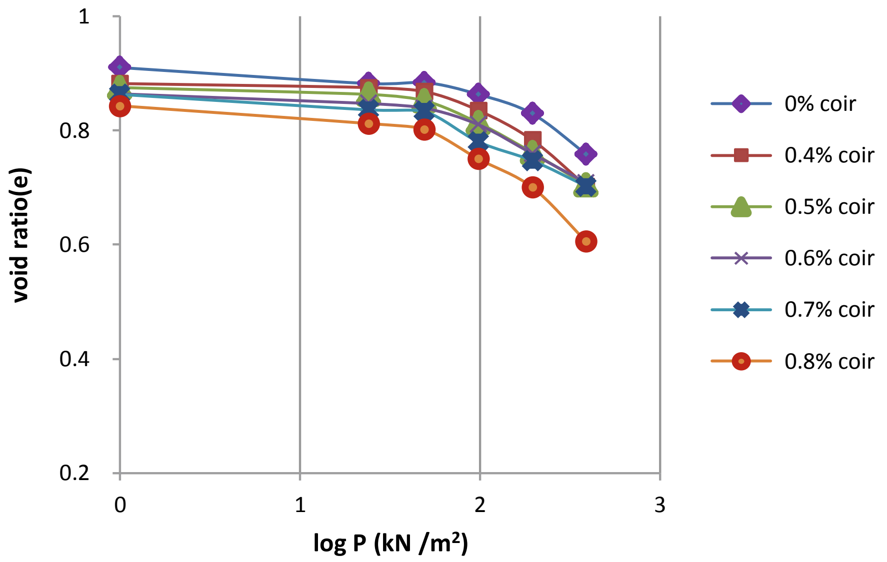

Fig. 4 Void ratio versus pressure curve

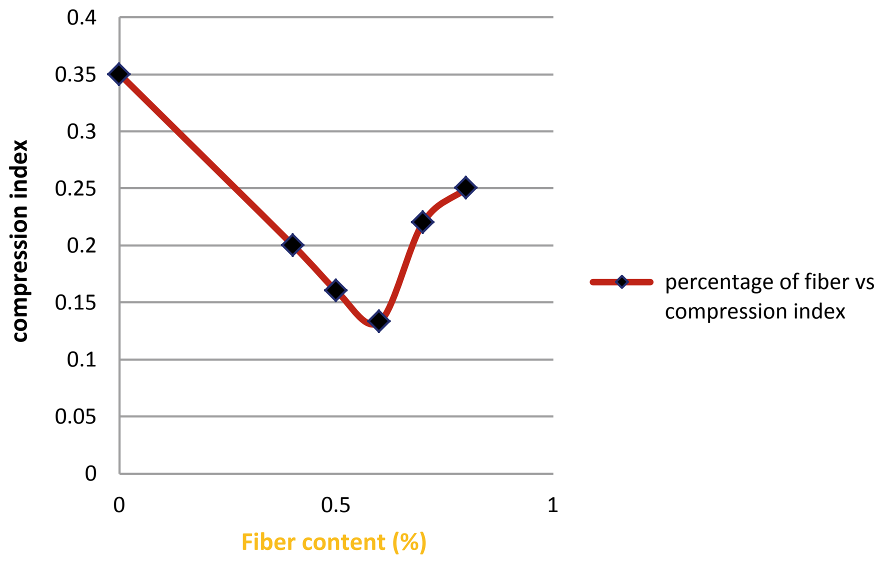

Fig. 5 Compression index versus percentage of fiber

From Fig. 1, the values of the coefficient of consolidation (c_v) increased on adding a different percentage of coir fibers, and thereafter with the addition of coir fiber, coefficient of consolidation shows a decrease. This is mainly due to the fact that the fibers provide a drainage path to drain out the water from the soil [1, 8]. So that more water will dissipate through the pores and reduces the time required to achieve primary consolidation. The void ratio versus pressure curve is given in Fig. 4. The void ratio decreases with the percentage inclusion of coir fibers since the void spaces are occupied by fibers [5].

The compression index versus percentage of fiber graph is given in Fig. 5. The value of compression index (c_c) decreases on adding coir fibers up to 0.6%, and thereafter with the addition of coir fiber, compression value shows an increase. The decrease in the compression index value is due to the fact that the tensile strength of coir fiber induces cohesion in clay particles. The confinement effect, friction angle and shear strength increase on adding fiber content and hence decrease in compressibility [12].

From Table 3, the values of coefficient of compressibility (a_v) and coefficient of volume change (m_v) were decreased as the addition of coir fibers. This is mainly due to the presence of fiber increases resistance to compression. The coefficient of permeability increased on adding fiber content and provides drainage path.

6 Conclusions

A series of one-dimensional consolidation test was conducted to evaluate the effects of coir fiber with varying percentages on the consolidation characteristics and permeability behavior of clayey soil.

- The compression index (c_c) decreased with the inclusion of coir fibers in the soil up to 0.6% fiber content and increased thereafter. Thus, minimum c_c value is observed at 0.6% for soil reinforced with the coir fibers.
- The value of the coefficient of volume change (m_v) was increased with the fiber content up to 0.6% fiber content and thereafter increases.
- The coefficient of consolidation (c_v) increases with the inclusion of coir fibers. Thus, the time required to achieve primary consolidation decreases for fiber reinforced soil for a given degree of consolidation and a given drainage path. Thus, c_v increased by 74% on adding 0.6% fiber, and thereafter on increase in fiber content, c_v increases.
- Compression index (c_c) decreased on adding fiber content. Thus, c_c decreased to 63% on adding 0.6% fiber content.

Based on the results obtained, it is recommended that coir fiber can be used as a reinforcing material for improving the consolidation characteristics and permeability behavior of the soil.

Table 3 Consolidation parameters of soil reinforced with coir fibers

Percentage of coir fiber	Load (kN/m^2)	Coefficient of compressibility(a_v) in (m^2/kN) $a_v = \frac{de}{d\sigma}$	Coefficient of volume change (m_v) in (m^2/kN) $m_v = \frac{a_v}{1+eo}$	Coefficient of permeability(k) (m/s) k = c_v m_v γ_w
0%	24.52	3.93×10^{-2}	2.05×10^{-2}	3.93×10^{-6}
	49.05	$4.32. \times 10^{-2}$	2.26×10^{-2}	$4.32. \times 10^{-7}$
	98.10	4.58×10^{-2}	2.39×10^{-2}	4.28×10^{-7}
	196.2	2.41×10^{-2}	1.26×10^{-2}	2.41×10^{-7}
	392.4	3.01×10^{-2}	1.57×10^{-2}	2.01×10^{-7}
0.4%	24.52	1.46×10^{-3}	7.76×10^{-3}	1.46×10^{-5}
	49.05	1.14×10^{-3}	6.02×10^{-3}	1.14×10^{-5}
	98.10	6.72×10^{-3}	3.57×10^{-3}	6.72×10^{-5}
	196.2	6.30×10^{-3}	3.35×10^{-3}	6.30×10^{-5}
	392.4	6.27×10^{-3}	3.33×10^{-3}	6.27×10^{-5}
0.5%	24.52	6.23×10^{-4}	3.33×10^{-4}	6.08×10^{-4}
	49.05	6.21×10^{-4}	3.32×10^{-4}	6.12×10^{-4}
	98.10	6.10×10^{-4}	3.26×10^{-4}	6.18×10^{-4}
	196.2	6.08×10^{-4}	3.25×10^{-4}	6.24×10^{-4}
	392.4	6.05×10^{-4}	3.23×10^{-4}	6.31×10^{-4}
0.6%	24.52	7.95×10^{-5}	2.45×10^{-5}	9.25×10^{-3}
	49.05	7.91×10^{-5}	2.32×10^{-5}	9.35×10^{-3}
	98.10	7.88×10^{-5}	1.18×10^{-5}	9.42×10^{-3}
	196.2	7.82×10^{-5}	9.98×10^{-5}	9.49×10^{-3}
	392.4	7.66×10^{-5}	9.90×10^{-5}	9.58×10^{-3}
0.7%	24.52	7.59×10^{-3}	4.13×10^{-3}	9.69×10^{-3}
	49.05	7.45×10^{-3}	4.04×10^{-3}	9.72×10^{-3}
	98.10	7.42×10^{-3}	4.03×10^{-3}	9.78×10^{-2}
	196.2	7.33×10^{-3}	3.98×10^{-3}	9.85×10^{-2}
	392.4	7.25×10^{-3}	3.94×10^{-3}	9.89×10^{-2}
0.8%	24.52	7.18×10^{-4}	3.86×10^{-4}	9.92×10^{-2}
	49.05	7.05×10^{-4}	3.79×10^{-4}	9.94×10^{-2}
	98.10	7.85×10^{-5}	4.22×10^{-5}	9.95×10^{-2}
	196.2	7.52×10^{-5}	4.04×10^{-5}	9.97×10^{-2}
	392.4	7.20×10^{-5}	3.87×10^{-5}	9.99×10^{-2}

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Ground Improvement of a Liquefiable Soil by Granular Piles

Koushik Pandit, Pradeep Kumar, and Gaurav Sharma

Abstract Liquefaction is a physical process by which soil sediments below the groundwater table temporarily loose strength and stiffness, and initiate to behave as a viscous liquid rather than a solid. In addition to earthquake and rapid application of large loads, a soil may liquefy due to construction activities like blasting and during ground improvement by vibro-flotation and dynamic compaction. Liquefaction phenomenon may cause unrecoverable damage to a building and other civil structures. Hence, it is very important to know about the liquefaction potential of a construction site so that suitable protection measures can be adopted before construction. Granular piles are one of the popular treatment methods to make the soil less prone to liquefaction. In this study, liquefaction potential of a project site at Darbhanga in Bihar has been evaluated (by method developed by Youd et al. 2001) from the borehole data where groundwater table is at a shallow depth. Once the liquefiable depths in substrata are identified, granular piles along with the shallow foundation have been selected as the best viable foundation solution based on the soil characteristics determined. The design of granular piles consists of certain parameters, such as their diameter, total length, number of piles and their arrangement at site. Use of granular piles will not only improve soil strength significantly but also will provide drainage option for the risen groundwater at high pore water pressure which may be generated during an earthquake event. To present the benefit of granular piles over conventional RCC piles, a comparative design and cost assessments of granular piles with RCC piles were also performed. It has been observed that a significant reduction of construction cost and settlement control may be achieved by granular piles over RCC piles. This kind of study will help in selecting an appropriate liquefaction measure and its design, leading to safer construction of the civil structures.

Keyword Liquefaction · Bearing pressure · Ground improvement · Granular piles

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1 Introduction

1.1 The Liquefaction Phenomenon

Soil liquefaction, which is usually known as sudden loss of shear strength in soil due to ground shaking followed by a rapid increase in pore water pressure, generally occurs in loose to very loose saturated granular soils. The ground shaking, predominantly due to cyclic earthquake motions, quickly causes dislodgement of the grain to grain contact of the individual soil grains. This cyclic disturbance causes a substantial loss in shear strength of soil which could result in instability or bearing capacity failures.

Liquefaction may cause any one or a combination of more than one of the following hazards at a vulnerable site: (i) lateral spreading, (ii) flow failures, (iii) loss of bearing strength and increase in settlement of the super-structure, (iv) increased lateral pressure on retaining walls and (v) ground oscillation which may alter ground motions in terms of amplitude, frequency content and duration. All or any of these may lead to ground failure and a subsequent failure of the super-structure. Liquefaction causes decrease or loss in vertical pile load capacity from both skin and end bearing resistances, depending upon the zone of liquefaction along the pile depth. It also reduces the lateral load carrying capacity of piles. Some of the key recent studies on liquefaction may be found in the literature [1–4]. Amini and Qi [1] conducted comprehensive experimental studies by stress-controlled undrained cyclic tri-axial tests to compare the behavior of stratified and homogeneous silty sands during seismic liquefaction conditions for various silt contents and confining pressures in the range of typical field conditions. The homogeneous and heterogeneous soils are important factors for dynamic liquefaction mechanism which is explained perfectly by Chakrabortty and Popescu [2]. Owen and Moretti [3] suggested that liquefaction develops most readily in loosely packed coarse silt to fine sand that is saturated with groundwater and at shallow depths. However, several new liquefaction phenomena have been observed in connection to twenty-first-century earthquakes, for example, liquefaction in areas of moderate seismic intensity, liquefaction of gravelly soils, liquefaction of deep-level sandy soils, re-liquefaction in aftershocks and liquid-like behavior of unsaturated sandy soils [4].

Hence, it is very important to know about the liquefaction potential of a construction site so that suitable protection measures can be adopted before construction. Granular piles are one of the popular treatment methods to make the soil less prone to liquefaction. In this study, liquefaction potential of a project site at Darbhanga, Bihar, has been evaluated (by method developed by Youd et al. [5]) from the borehole data where groundwater table is at a shallow depth. Once the liquefiable depths in substrata are identified, granular piles along with the shallow foundation have been selected based on the soil characteristics determined. To present the benefit of granular piles over conventional RCC piles, a comparative design and cost assessment of granular piles with RCC piles were also performed. It has been observed that a significant reduction of construction cost and settlement control may be achieved by granular piles over RCC piles.

1.2 Evaluation of Liquefaction Potential

Evaluation of soil liquefaction potential and its consequent hazard require an engineering skill with good judgement from past experiences besides testing and analysis. Significant developments have been accomplished in the past few decades in evolving tools to help evaluating the soil liquefaction potential; however, still some characteristics of the liquefaction problem continue to remain ambiguous. An extensive variation of approaches from researchers and experts has been in use to perform the soil liquefaction analysis. The "current standard-of-practice" for evaluating soil liquefaction potential during earthquakes can be found in the paper titled "Liquefaction Resistance of Soils: Summary of Report from the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation (NSF) Workshops on Evaluation of Liquefaction Resistance of Soils" [5]. The SPT-based and CPT-based liquefaction analysis procedures summarized in their paper will hereafter be referred to as the Youd et al. [5] procedures. Lately, Youd et al. [5] approach has been examined, and liquefaction susceptibility assessment of silts and clays from the CPT-based correlation has been under scrutiny, mostly because of (1) increased volume of field-based records, (2) enhanced assessment of peak ground accelerations at sites and (3) better understanding of the liquefaction behavior of silts and clays [6].

2 Site Description

For the present paper, a liquefaction susceptible site located in Darbhanga, Bihar, has been studied where an electrical powerhouse or substation is going to be constructed. Before construction, geotechnical investigations were carried out. At two borehole locations in the site, standard penetration tests (SPTs) were carried out, whereas tri-axial and direct shear tests were performed at laboratory for determination of the shear strength properties. For soil classifications, grain size distribution analysis, liquid limit and plastic limit values were also determined. The soil properties or characteristics are described below for the two boreholes (Tables 1 and 2).

The groundwater table depths at borehole 1 and 2 locations have been observed at 4.0 m and 1.5 m below the existing ground levels, respectively.

3 SPT N-Value-Based Liquefaction Potential Analysis

As per [7], the site at Darbhanga is located in the highest risk-prone earthquake zone V of the country and is suspected to have liquefaction potential. Hence, it is of utmost importance to evaluate the liquefaction potential of it prior to any construction. Also, if the design of foundations needs liquefaction hazard mitigation measures to provide

Table 1 Soil characteristics from borehole 1 (BH-1) location

N-value	Depth meters	Sample No.	Soil description	Grain size analysis					Liquid limit W_L	Plastic limit Wp	Dry density g/cc	Water content %	Shear parameter		
				I.S. classification	Gravel %	SAND %	SLT %	CLAY %					Test method	C kg/cm^2	Ø degree
	G.L.	DS													
4	1.50	SPT	Filled up soil												
	2.00	UDS	Clay of medium plasticity	CI	0.0	14.0	64.0	22.0	45.0	26.0	1.47	18.90	TST	0.45	4°
6	3.00														
6	3.00	SPT													
	4.00	UDS	Loose clay, sample not collected												
10	4.50	SPT													
	5.00	UDS	Clay of medium plasticity	CI	0.0	16.0	64.0	20.0	40.0	25.0	1.49	20.30	TST	0.50	4°
7	6.00	SPT													
	7.00	UDS	Silty sand	SM											
9	7.50		Silty sand	SM	0.0	72.0	28.0	0.0	N	P	1.50	20.80	DST	0.0	30°
	8.00		Silty sand												
15	9.00	SPT													

(continued)

Table 1 (continued)

N-value	Depth meters	Sample No.	Soil description	Grain size analysis					Liquid limit W_L	Plastic limit Wp	Dry density g/cc	Water content %	Shear parameter		
				I.S. classification	Gravel %	SAND %	SLT %	CLAY %					Test method	C kg/cm^2	Ø degree
	10.00	UDS	Silty sand												
11	10.50	SPT	Silty sand	SM	0.0	80.0	20.0	0.0	N	P					
	11.00	UDS	Silty sand	SM											
12	12.00	SPT													
	13.00	UDS													
15	13.50	SPT	Silty sand												
	14.00	UDS	Silty sand												
16	15.00	SPT													
	/														
	15.45														

Table 2 Soil characteristics from borehole 2 (BH-2) location

N-value	Depth meters	Sample No.	Soil description	Grain analysis					Liquid limit W_L	Plastic limit Wp	Dry density g/cc	Water content %		Shear parameter	
				I.S. classification	Gravel %	SAND %	SLT %	CLAY %					Test method	C kg/cm^2	Ø degree
	G.L.	DS													
	1.50	SPT													
6	2.00	UDS	Clay of medium plasticity	CI	0.0	15.0	65.0	20.0	42.0	25.0	1.48	18.50	TST	0.50	3°
	3.00	SPT													
5	4.00	UDS	Loose clay, sample not collected												
	4.50	SPT	Clay of medium plasticity	CI	0.0	17.0	61.0	22.0	43.0	26.0					
8	5.00	DS													
	6.00	SPT													
8	7.00	SPT	Clay of medium plasticity	CI	3.0	20.0	57.0	20.0	40.0	25.0	1.56		TST	0.65	3°

(continued)

Table 2 (continued)

N-value	Depth meters	Sample No.	Soil description	Grain analysis					Liquid limit W_L	Plastic limit *Wp*	Dry density g/cc	Water content %		Shear parameter	
				I.S. classification	Gravel %	SAND %	SLT %	CLAY %					Test method	C kg/cm^2	Ø degree
	7.50	UDS													
12	8.00	UDS	Clay of low plasticity												
	9.00	SPT													
20	10.00	UDS	Clay of low plasticity with gravels	CL	10.0	24.0	53.0	13.0	34.0	23.0					
	10.50	SPT													
13	11.00	UDS													
	12.0	SPT	Clay of low plasticity with gravels	CL	13.0	26.0	49.0	12.0	33.0	22.0					
11	13.00	UDS													
	13.50	SPT													
16	14.00	UDS													

(continued)

Table 2 (continued)

N-value	Depth meters	Sample No.	Soil description	Grain analysis					Liquid limit W_L	Plastic limit Wp	Dry density g/cc	Water content %		Shear parameter	
				I.S. classification	Gravel %	SAND %	SLT %	CLAY %					Test method	C kg/cm^2	Ø degree
	15.00	SPT	Clay of low plasticity	CL	17.0	30.0	44.0	9.0	28.0	22.0	1.64		TST	0.50	5°
16	/	SPT													
	15.45														
	15.45														

safety against liquefaction, then that component is also required to be incorporated in the final design.

In the present study, SPT values of two boreholes for Darbhanga site have been determined, and the same are utilized for the liquefaction potential analyses as per the method prescribed by [5].

Abbreviations used for the analyses are: γ = Bulk unit weight of soil, σ_{vo} = Total overburden stress, α'_{vo} = Effective overburden stress, N_m = Measured SPT value at field, C_{R} = Correction factor for drilling rod length, C_E = Correction factor for hammer energy ratio, C_S = Correction factor for sampling method, C_{B} = Correction factor for borehole diameter, C_N = Correction factor to normalize Nm to a common reference effective overburden stress, $(N_1)_{60}$ = Corrected standard penetration resistance, r_{d} = Stress reduction coefficient to account for flexibility in soil profile, CSR = Calculated cyclic stress ratio generated by the earthquake shaking, FC = Fines content (in percentage) of soil passing through US sieve no. 200, $(N_1)_{60}$ (c.s.) or $N_{1,\ 60,\ \text{cs}}$ = Value of $(N_1)_{60}$ adjusted to equivalent clean-sand value, CRR $_{(\text{m} = 7.5)}$ = Cyclic resistance ratio for $M_{\text{w}} = 7.5$ earthquakes, MSF or K_{m} = Magnitude scaling factor, K_σ = Correction factor for soil layers subjected to large static normal stresses, K_α = Correction factor for soil layers subjected to large static shear stresses, CRR = Cyclic resistance ratio and FoS = Factor of safety against liquefaction potential. The results for the liquefaction potential analyses for the two boreholes are described below (Tables 3 and 4).

Groundwater depth from top = 4.0 m, maximum horizontal acceleration (MHA) assuming Zone V area of IS 1893: 2002 = 0.36 (g), Design moment magnitude = 7.5, MSF or K_m factor = 1.0.

Groundwater depth from top = 1.5 m, maximum horizontal acceleration (MHA) assuming Zone V area of IS 1893: 2002 = 0.36 (g), design moment magnitude = 7.5, MSF or K_m factor = 1.0.

From the above analyses, it can be inferred that the entire length of soil up to the depth of exploration in borehole location 1 is liquefiable or susceptible to liquefaction. On the other hand, except the soil strata between the depths 9.0 m and 10.5 m, the entire soil strata up to the depth of exploration in borehole location 2 are liquefiable or susceptible to liquefaction. Hence, there is a need to provide liquefaction resistant foundation design as well as liquefaction measures to minimize its adversity if it occurs during the design life of the structure.

4 Recommendations to Mitigate Liquefaction Hazard

4.1 Approaches for Liquefaction Hazard Mitigation

The basic approach for earthquake disaster mitigation can be broadly classified into two major categories: (i) preventing or minimizing the probability of liquefaction (ground improvement) and (ii) minimization of damages in the event of

Table 3 Liquefaction analysis for borehole location: 1

Depth (m)	(γ (kN/m^3)	α_{vo} (kPa)	α'_{vo} (kPa)	N_m	C_R	C_E	C_s	C_B	C_N	$(N(1)_{60}$	r_d	CSR	FC	$((N_1)_{60}$ (c.s.)	CRR (m = 7.5)	K_σ	K_α	CRR	FoS	Remarks
1.5	17.15	25.719	25.719	4	0.8	1.1	1.2	1.05	1.70	7.5	0.990	0.232	86	14.0	0.151	1	1	0.151	0.65	Liquefiable
3.0	17.15	51.439	51.439	6	0.8	1.1	1.2	1.05	1.39	9.3	0.979	0 220	86	16.1	0.172	1	1	0.172	0.75	Liquefiable
4.5	17.58	77.815	72.91	10	0.8	1.1	1.2	1.05	1.17	13.0	0.969	0.242	84	20.6	0.223	1	1	0.223	0.92	Liquefiable
6.0	17.78	104.48	84.858	7	0.8	1.1	1.2	1.05	1.09	84	0.958	0.276	84	15.1	0.161	1	1	0.161	0.58	Liquefiable
7.5	17.78	131 14	96.807	9	0.8	1.1	1.2	1.05	1.02	10.1	0.943	0.299	28	16.1	0.171	1	1	0.171	0.57	Liquefiable
9.0	17.78	157.81	108 76	15	0.8	1.1	1.2	1.05	0.96	15.9	0.923	0.313	28	22 7	0.253	0.98	1	0.246	0.79	Liquefiable
10.5	17.78	134.47	120 7	11	0.8	1.1	1.2	1.05	0.91	11.1	0.894	0.320	20	15.6	0.166	0.95	1	0.157	0.49	Liquefiable
12.0	17.78	211.13	132.65	12	0.8	1.1	1.2	1.05	0.87	11.6	0.857	0.319	20	16.1	0.171	0.92	1	0.157	0.49	Liquefiable
13.5	17.78	237 8	144 6	15	0.8	1.1	1.2	1.05	0.83	13.8	0.811	0.312	14	16.6	0.177	09	1	0.158	0.51	Liquefiable
15.0	17.78	264.46	156 55	16	0.8	1.1	1.2	1.05	0.80	14.2	0.761	0.301	32	21.4	0.234	0.87	1	0.205	0.68	Liquefiable

Table 4 Liquefaction analysis for borehole location: 2

Depth (m)	(γ (kN/m^3)	α_{vo} (kPa)	α'_{vo} (kPa)	N_m	C_R	C_E	C_s	C_B	C_N	$(N(1)_{60}$	r_d	CSR	FC	$((N_1)_{60}$ (c.s.)	CRR ($m = 7.5$)	$K\sigma$	$K\alpha$	CRR	FoS	Remarks
1.5	17.20	25.807	25.807	6	0.8	1.1	1.2	1.05	1.70	11.3	0.990	0.232	85	18.6	0 198	1	1	0 198	0.86	Liquefiable
5.0	17.20	51.614	36.899	5	0.8	1.1	1.2	1.05	1.65	9 1	0.979	0.321	85	16.0	0 170	1	1	0 170	0.53	Liquefiable
4.5	17.20	77.422	47.992	8	0.8	1.1	1.2	1.05	1.44	12.8	0.969	0.366	83	20.4	0 220	1	1	0 220	0.60	Liquefiable
6.0	18.43	105 06	60.915	8	0.8	1.1	1.2	1.05	1 28	11.4	0.958	0.387	83	18.6	0 199	1	1	0 199	0.51	Liquefiable
7.5	18.43	132.7	73.838	12	0.8	1.1	1.2	1.05	1.16	15.5	0.943	0.397	77	23.6	0 266	1	1	0266	0.67	Liquefiable
9.0	18.43	160 34	86.761	20	0.8	1.1	1.2	1.05	1.07	23.8	0.923	0.399	n	33.6	$(N_1)_{60}$ < (c.s.) > 30	1	1	$(N_1)_{60}$ < (c.s.) > 30	$(N_1)_{60}$ < (c.s.) > 30	Non-Liq.
10.5	18.43	137 97	99.685	13	0.8	1.1	1.2	1.05	1.00	14.4	0 894	0.395	66	22.3	0 247	1	1	0 247	0.62	Liquefiable
12.0	13.43	215 61	11,261	11	0.8	1.1	1.2	1.05	0.94	11.5	0.857	0.384	61	18.8	0 201	0.97	1	0 194	0.50	Liquefiable
13.5	19.64	245 08	127 36	16	0.8	1.1	1.2	1.05	0.89	15.7	0.811	0.365	61	23.9	0 271	0.93	1	0 252	0.69	Liquefiable
15.0	19.64	274.54	142.11	16	0.8	1.1	1.2	1.05	0.84	14.9	0.761	0.344	53	22.9	0.255	0.9	1	0.229	0.67	Liquefiable

liquefaction (structural improvement). Among the various techniques or remediation methods against liquefaction, more widely used methods for liquefaction mitigation are: vibro-methods (vibro-rod, vibro-compaction and vibro-replacement), deep dynamic compaction, compaction grouting, deep soil mixing, jet grouting and drainage through granular piles or stone columns. All other methods are costly to very costly except the drainage through granular piles method. Also, granular piles are effective in mitigating liquefaction damage due to their reinforcement effect and provision of water drainage facility.

Granular piles (GP) function as drains and permit rapid dissipation of earthquake-induced pore pressures by virtue of their high permeability. The generated pore water pressure due to repeated loading may get dissipated almost as fast as they are generated. In addition, they tend to dilate (through bulging) as they get sheared during an earthquake event. Seismic forces which tend to generate positive pore pressures in these deposits cause an opposite effect of dilation in dense granular piles. One of the main benefits of ground treatment with granular piles is the densification of the in-situ ground by which the in-situ geomechanical properties (such as the shear strength) of the ground get enhanced to mitigate the seismic risks, especially, to reduce its liquefaction potential. In addition to it, granular piles provide increased bearing capacity and significant reduction in settlement, besides achieving cost economy. These benefits of granular piles make them a natural choice for liquefaction hazard minimization.

In order to substantiate the above statement, design calculations have been made to compare cost of construction of granular piles and conventional RCC piles for borehole 2 location of Darbhanga site where the substation building is going to be constructed.

4.2 Design of Granular Piles

The substation building has a load bearing area of $= 28\ \text{m} \times 22\ \text{m} = 616$ sq. m., and the super-structure design load is expected to be of 1850 Ton, which makes the load intensity as 3 T/m^2.

The diameter of granular piles to be installed at site is assumed to be of 0.3 m for the preliminary design. Hence, cross-sectional area of each granular pile becomes $A_p = 0.07065\ \text{m}^2$. The passive earth pressure coefficient, K_p, is 2.94 for the stone ballasts. Generally, the critical length of granular pile is 5 times the diameter of pile, which makes it 1.5 m for the present case. Critical length of a granular pile is that length beyond which the piles will not have any significant contribution in minimizing the liquefaction hazards in terms of the additional bearing capacity they may provide. From the soil samples collected from the borehole, the unit weight of soil is found to be 18.0 kN/m^3.

The design of load carrying capacity of each granular pile is obtained from the empirical equation provided by Hughes and Withers [8]:

$$Q_d = K_p * (8\ C_u + O_e + O_v) * A_p \quad (1)$$

From the above equation, Q_d is obtained as = 10.16 T for each of the granular piles. Now, assuming a FoS of 2.5, the safe bearing capacity of each granular pile, Q_{safe}, becomes 4.06 T.

Hence, total number of granular piles required to be installed = 1850/4.06 = 456 no's (approx.).

Finally, the approximate spacing between granular piles, if installed in a zig-zag triangular pattern, becomes 1.25 m.

4.3 RCC Pile Design

For comparison with the granular piles, let us take the RCC pile diameter as 0.35 m and length of pile required = 12.0 m (as higher SPT values obtained beyond 9.0 m depth from the ground). Here, undrained cohesion = 50 kN/m^2 (from Table 2).

Hence, load carrying capacity of each pile = $c_{\mathrm{ub}} * N_c * A_b + \alpha * c_{\mathrm{u}} * A_s$ = 50 * 9 * (3.14 * 0.352) + 0.7 * 50 * (3.14 * 0.35) * 12.0 = 504.85 kN = 50.48 T.

Safe bearing capacity (assuming a FoS of 2.5) = 20.19 T.

Hence, total number of piles required = 1850/(0.8 * 20.19) (assuming a pile group efficiency of 80%) = 115.

4.4 Cost Comparison Between Granular Piles and RCC Piles

The cost comparison between the granular and RCC piles designed is shown Table 5.

Hence, total savings in cost of construction of granular piles over RCC piles = [(10, 21,200 − 2, 05,200)/10, 21,200] * 100 = 79.9%.

Additionally, other advantages of granular piles over RCC piles are as follows:

- Overall settlement control,
- Liquefaction mitigation through drainage, soil compaction and reinforcement,
- Use of natural materials like stone ballasts over steel and concrete, thus reducing carbon footprint,

Table 5 Cost comparison between granular piles and RCC piles

Type of pile	Length of each pile (m)	Total number of piles required	Total running length (m)	Cost per unit length[a], Rs.	Total cost, Rs.
Granular piles	1.5	456	684.0	300.00/–	205,200/–
RCC piles	12.0	115	1380.0	740.00/–	1,021,200/–

[a]Cost of construction per unit length is computed as per Delhi Schedule of Rates (DSR - 2014)

- Ease of construction at site, does not require skilled labors and heavy machineries like rigs/cranes.

5 Conclusions

In the present study, a liquefaction susceptible site is analyzed, and liquefaction potential of the same has been evaluated. Accordingly, both granular piles and conventional RCC piles have been designed for the design load of an electrical substation building. The cost comparison between the two shows that granular piles can be a better alternative for the studied case.

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Influence of Textile Polyester Waste Fiber on Strength and Subgrade Properties of High to Medium Plastic Clay

Pratima A. Patel and Yati R. Tank

Abstract In India, a major portion of total land area is covered by high to low plastic clayey soil. Of this, a large proportion is expansive soil. Structures constructed over this expansive soil may be severely damaged due to its high swell-shrinkage behavior. So, such soils need to improve its strength, durability and to prevent erosion. Various studies have been carried out on expansive soils to improve its properties. Soil stabilization is one of the promising techniques used to improve the geotechnical properties of soil and has become the major practice in construction engineering. This study works to evaluate the improvements in properties of CH–CI soil of south region of Surat by adding textile polyester waste fiber. For improvement of engineering properties, this fibers are used as reinforcement by varying percentage of 1, 2 and 3%. The soil parameters such as shear strength, subgrade characteristics tested under UCS and CBR. These values are compared to that of a control specimen. Author critically remarked as CH soil has different characteristics as compared to CI soil under this test. Experiment results show this fact that using textile polyester waste fibers leads to increasing shear strength, dry density, CBR value and reduction in plasticity index and free swell index in CI soil as compared to CH soil. Analysis of result obtained from experiments may be proving the effectiveness of this product to construction site having this both types of soil. The expansive soil can be successfully stabilized by the combined action of fibers with soil.

Keywords Stabilization · Textile polyester waste fibers · UCS · CBR · Free swell index

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1 Introduction

1.1 Preface

Urbanization leads to non-availability of good ground condition for construction of new infrastructure throughout the world. Hence ground improvement is becoming the integral part of the infrastructure development in both developed and under developing countries. Expansive soil is considered as most problematic soil as it shows the increase and decrease in volume in presence and in absence of water, respectively. The major problem associated with the consulting geotechnical engineer in the last few decades is to design a foundation on expansive soil which threat to public safety and the potential damage to property.

Expansive soil can exert the force and cause the movement to a heavily loaded structure that can be more than those experienced from the ordinary soil. Along with this, cost incorporated in development of expansive soil site is very much high compared to non-expansive soil site. In contrast with normal soil site, expansive soil requires more extensive testing and analyses during the site investigation and design phase, and also, the construction phase needs continuous supervision and observation as compared to normal available soil. Additionally, the cost associated with the reconstruction of a structure damaged by expansive soil may be restrictive because the cost of reconstruction is often more than the original cost (initially estimated cost) of the structure.

It can be concluded that estimated lost caused by expansive soils may be more than the annual damage from the earthquake, floods, hurricanes and tornados (column of air which is rotated rapidly, extends from the thunderstorm to ground).

In dry state, expansive soil has the consistency of stiff clay and has the excellent shear strength which diminished significantly with the ingress of water. High swelling and shrinkage characteristics of the expansive soil are due to the presence of a clay mineral known as montmorillonite, Illite, Kaolinite, etc.

Expansive soil deposits and the problem caused by this soil are reported from all six continents except Antarctica and more than 40 countries worldwide [1]. The mentioned below countries in which expansive soil has been reported are Argentina, Australia, Burma, Canada, Cuba, Ethiopia, Ghana, India, Israel, Iran, Mexico, Morocco, Rhodesia, South Africa, Spain, Turkey, U.S.A. and Venezuela [2]. So, the problem of expansive soil is worldwide. Expansive soils are in abundance where the annual evapotranspiration exceeds the precipitation. This follows the theory that in semi-arid zones, the lack of leaching has aided the formation of montmorillonite [3]. The above-mentioned countries belong to the semi-arid area only.

The problem of expansive soil is not only related to economy of infrastructure but also with environmental perspective of living being. Expansive soil exerts great uplift pressure on the structure built on it when it is in contact with water which results in excessive heaving, cracking of the structural and non-structural members of the building and destruction of the road pavements, underground pipeline system, lined and unlined canals and airfields. These are the major area where expansive soil deposits required great care in all phases of construction.

1.2 Problem Summary

In this project work, purpose of addition of waste textile fiber is to improve penetration resistance and shear characteristics.

In this present study, the stabilization of two types of soil, i.e., CH and CI with the artificial fiber, i.e., polyester which is in the form as a waste product from textile industries has been carried out. This polyester fiber is to be mixed with both the types of soil by partial replacement of soil by weight in 1, 2 and 3%. The soil is naturally available from the ongoing site from Vesu, situated at southwest zone of Surat city. This fiber increases the dry density of soil and thus increases the shear strength and load bearing capacity of soil by conducting unconfined compressive strength, California bearing ratio and compaction tests. CH soil has different characteristics as compared to CI soil. The varying dry density, shear strength and load bearing capacity give the variation in strength and subgrade properties.

Study shows the effect of textile waste recycled polyester fiber on soil engineering properties by conducting standard proctor test, unconfined compression strength and California bearing ratio test.

1.3 Aim and Objective of Present Study Work

In our study, an attempt is made to stabilize expansive soil with addition of textile fibers. The strength parameters like CBR, UCS are determined to know the suitability of materials.

In the process of soil stabilization and modification, emphasis is given for maximum utilization of local material so that cost of construction may be minimized to the minimum extent.

Soil stabilization is the treatment done on the soils to improve their properties so that it becomes suitable for construction.

The aim of proposed study is to assess and evaluate all basic properties of CH and CI soil before adding the artificial fiber, i.e., textile waste recycled polyester fiber. Evaluated properties of untreated soil with the artificial fiber on addition of 1%, 2% and 3% by weight as per Indian standard code and compared with natural untreated soil. By gradual increase in the percentage of fiber in the CH and CI soil, the increment in strength, dry density and penetration capacity of soil is to be determined. Mainly modified compaction test, UCS and CBR tests will be conducted on treated soil sample at laboratory in control volume. Results of all tests were compared with untreated soil sample.

1.4 Problem Specifications and Background

Soil stabilization has evolved innovative techniques of utilizing locally available environmental and industrial waste material for the modification and stabilization of deficient soil.

Surat city is known for its textile business, so plenty of textile polyester fibers waste is available. In this study, stabilization of clayey soil has been done using textile waste recycled polyester fiber to improve engineering properties such as unconfined compressive strength and California bearing ratio. This fiber is used as a non-traditional reinforcement. The influence of randomly oriented polyester fiber on the engineering behavior of soil has not been reported to some extent. Ease of application and reduction in cost are making this treatment more popular. Polyester fiber is not 100% prone to restrict the water, and therefore, it absorbs a very small amount of water. Maybe a little water could be on the surface of the fiber to make it wet, it does not necessarily need to absorb.

2 Contribution of Researchers

Avoiding the site is not a better solution when site is located in urban area as scarcity of land. Adopting Raft and Pile foundation is very much costly affairs for lightweight structures. So, soil stabilization as a ground improvement is better solution for lightweight structure on high to low expansive soils.

The identification of expansive soil can be done through the understanding of its microscale aspects and macroscale aspects. The microscale aspects of the expansive soil include mineral composition, morphology, caution exchange capacity and specific surface area. The macroscale aspects include Atterberg properties, compressibility characteristics, swelling properties and strength characteristics.

The stabilization of expansive soil is most effective, while it is carried out using the addition of traditional and non-traditional additives. The traditional additives include lime and cement, whereas non-traditional additives include majorly available industrial wastes such as fibers, fly ash, ground granulated blast furnace slag (GGBS), copper slag, cement kiln dust (CKD) and rise husk ash (RHA). Literature study is highlighted only the review of soil stabilization using non-traditional additives.

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3 Experimental Investigation

3.1 *Test Material Used as Expansive Soil (CH and CI), Water, Textile Polyester Waste Fiber*

Expansive Soil (CH&CI)

The soil is classified as CH and CI soil as per IS: 1498–1970 and has a high to low free swell index (30–45%) according to the classification. The black cotton soil is mixed with textile polyester fiber in both the types of soil by partial replacement by weight in 1, 2, 3% to increase the reliability of experiments results and thereby achieving the intended purpose of the study. (i.e., effect of fibers on strength and subgrade properties of expansive soil).

These different proportions of mixtures (black cotton soil and fibers) are examined by a series of tests (UCS, CBR) to decide one suitable proportion for future investigation.

The high plastic clay as shown in Fig. 1 and medium plastic clay as shown in Fig. 2 have been obtained at a depth of 1 m and 2.5 m respectively from the construction site of Surat Multispecialty Hospital near Shanti Niketan Appt., Vesu, (southwest zone of Surat city) for investigation.

Two types of expansive soil (CH&CI) sample were collected as consideration for the study work for evaluation of behavior of various proportion of fibers (Fig. 3) with this soil.

Water

Water used for mixing and curing shall be clean and free from injurious amounts of oils, salts, acids, alkalis, sugar, organic materials or other deleterious materials. The pH value of water shall be not less than 6. Potable water or distill water is generally considered satisfactory for mixing.

Textile Polyester Waste Fibers

Source: Shree Balaji Textile, Navjivan Circle Bhatar, Surat. A manufactured fiber in which the fiber forming substance is any long-chain synthetic polymer composed of

Fig. 1 CH soil sample

Fig. 2 CI soil sample

Fig. 3 Textile polyester waste fiber

at least 85% by weight of an ester of a substituted aromatic carboxylic acid, including but not restricted to substituted terephthalic units, p(–R–O–CO–C_6H_4–CO–O–)x and para substituted hydroxy-benzoate units, p(–R–O–CO–C_6H_4–O–)x.

Characteristics of Polyester Fiber: Strong, resistant to stretching and shrinking, resistant to most chemicals, quick drying, crisp and resilient when wet or dry, wrinkle resistant, mildew resistant, abrasion resistant, retains heat-set pleats and crease, easily washed.

Cut Length: Cut lengths available are 32, 38, 44, 51 and 64 mm for cotton type spinning and a blend of 76, 88 and 102 mm-average cut length of 88 m for worsted spinning. The most common cut length is 38 mm.

These fibers are generally used on worsted system and 1.4 D for knitting.

Tensile Properties: usually in 2.0/3.0 D for suiting endues with tenacities of 3.0 to 3.5 GPD (grams per denier).

3.2 Experimental Programme

Test schedule of untreated soil sample of CH and CI as: Grain size analysis, liquid limit and plastic limit test, shrinkage limit. Specific gravity, standard compaction test, California bearing ratio, unconfined compressive strength as per I.S. codes specifications.

4 Results and Discussion

Test results as follows: (Without addition of Fibers).

The clay concentration is high in both the soil samples from the study site of Vesu, Surat. (from sieve analysis).

The CH soil has high expansive property, and CI soil has moderate expansive property. From the test results, properties of CH and CI are justified as per their classification.

Table 1 shows all index properties of CI and CH soil. These all properties verified and confirmed with IS specification.

Results of Compaction test, CBR test and UCS test (CH and CI soil with fibers and without fibers).

Table 2 highlights test result of compaction CBR and UCS, which shows variation in each value in comparison with untreated soil samples.

Remark of Compaction Test

Experimental results on compaction test show that maximum dry density decreases and optimum moisture content increases with the increase in polyester fiber content.

Table 1 Various index properties of CH and CI soil sample

Index properties assessed	For CH soil	For CI soil
Liquid limit	60.49%	44.33%
Plastic limit	20.04%	17.08%
Plasticity index	40.45%	27.25%
Free swell index	45.45%	30%
Specific gravity	2.33	2.51
Shrinkage limit	12.13%	23.65%
OMC	23%	20.60%
MDD	1.54 g/cc	1.66 g/cc

Table 2 Comparative result of all tests

Soil	Mix type	Sample name	MDD (g/cc)	OMC (%)	UCS (kg/cm^2)	CBR (2.5 mm) (%)	CBR (5 mm) (%)
CH	Untreated	CH (U)	1.54	23.00	1.50	1.87	1.58
	CH + 1% fiber	CH (1%)	1.55	21.20	2.34	1.75	1.85
	CH + 2% fiber	CH (2%)	1.54	21.40	2.18	1.69	2.04
	CH + 3% fiber	CH (3%)	1.46	26.80	1.64	3.70	3.23
CI	Untreated	CI (U)	1.66	20.60	3.04	3.73	4.57
	CI + 1% fiber	CI (1%)	1.59	22.00	2.82	6.37	6.90
	CI + 2% fiber	CI (2%)	1.54	22.50	4.50	6.96	7.16
	CI + 3% fiber	CI (3%)	1.49	22.80	4.52	4.73	7.03

The decrease in MDD is due to the results of the fiber having less specific gravity in comparison with the soil grains, and fibers prevent the soil particles to approach each other. The increase in OMC is due to result of fibers having greater water absorption capacity than the surrounding soil.

Figure 4 shows with constant value of MDD of untreated soil, addition of fiber MDD value decreases.

Figure 5 shows with constant value of OMC of untreated soil, addition of fiber

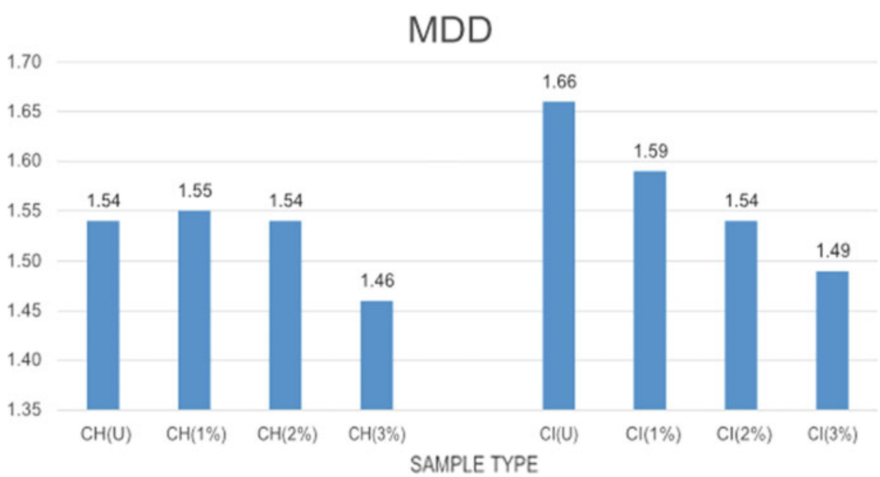

Fig. 4 Comparative result graph of MDD

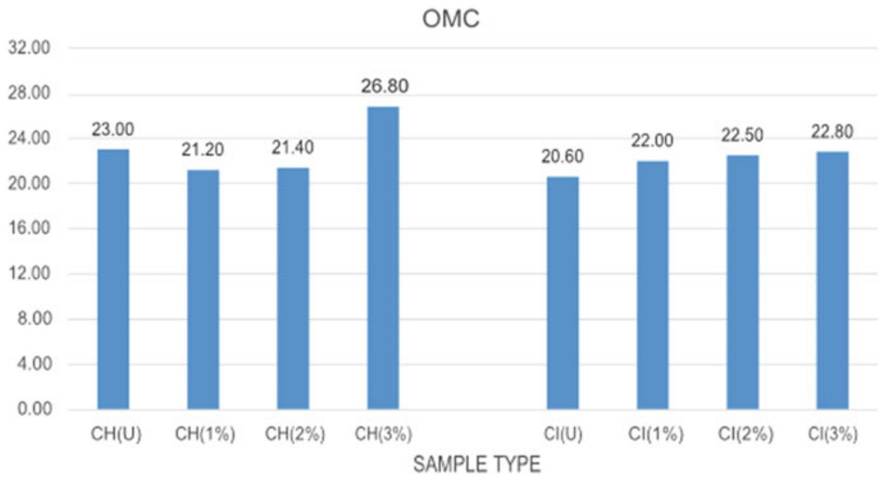

Fig. 5 Comparative result graph of OMC

OMC value increases.

Figure 6 shows UCS result improves in low plastic soil than high plastic soil.

Figure 7 shows with addition 3% of fiber CH soil gives maximum CBR value at 2.5 mm penetration but CI. This value is increased at 5 mm penetration.

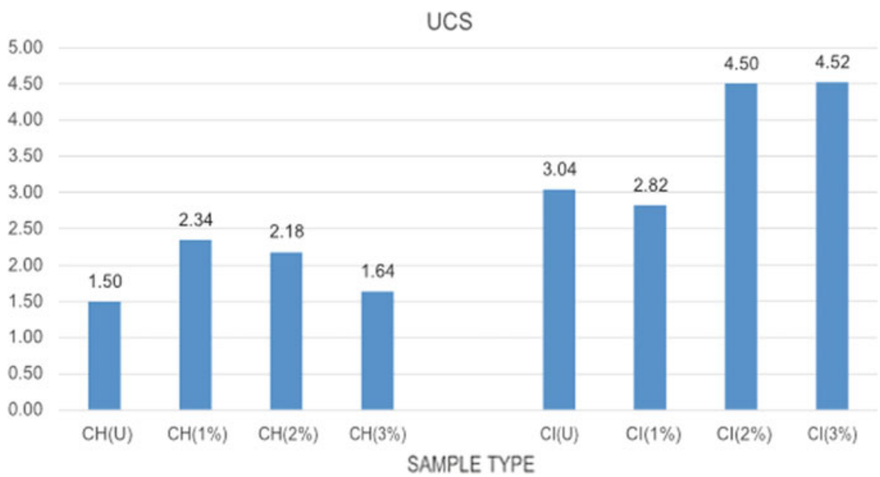

Fig. 6 Comparative result graph of UCS test

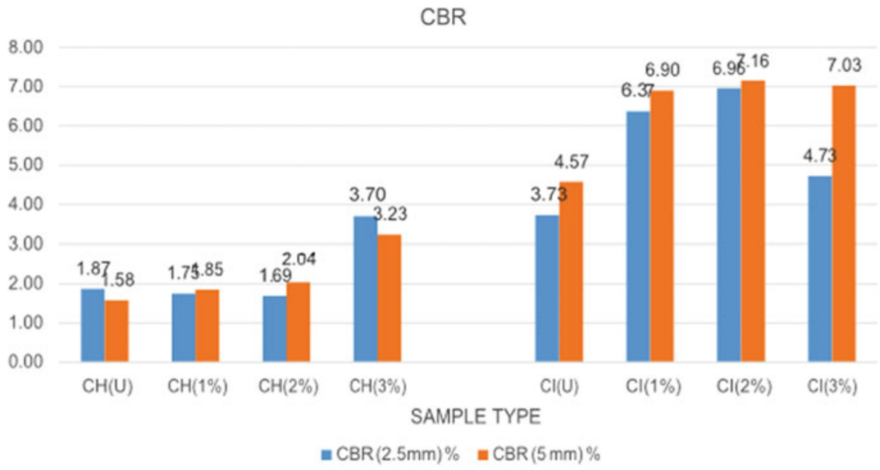

Fig. 7 Comparative result graph of CBR test

5 Concluding Remarks

Detailed conclusions as results of the study have been discussed. The analysis of the results are shown in tabular form. Also, these work focused on the effect of varying percentage of fibers on both soils.

- It has been observed that medium plastic clay covered most of the part in study area and hence important to enhance its properties for construction on it.
- Due to mineral composition of CI soil, it gives best results as compared to CH soil. CI soil can be used for pavements, soil stabilization and bearing capacity. By gradually increasing percentage of fiber, the strength behavior of CI soil also increases. CH soil is highly plastic soil due to high value of plastic limit and liquid limit and high free swell index, which affects the desire properties of soil.
- As during the soil exploration, CH soil is to be found generally in the upper crust, and CI soil is at deeper depth is been ignored, but here, the results show that CI soil has good strength behavior as compared to CH. So, the CI soil is preferable for construction purposes.

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Effect of Silica Fume and Induction Furnace Slag in the Compaction and Strength Characteristics of Black Cotton Soil

Winny Joseph and R. Sridhar

Abstract The present investigation brings out the experimental outcomes of the influence of two industrial solid wastes on black cotton soil. It evaluates the changes in compaction and strength characteristics of the black cotton soil on addition of these solid wastes. Disturbed soil sampling had been done, and as per Indian classification system, the soil is classified as CH soil. The virgin soil is mixed with varied percentages of silica fume and induction furnace slag independently, and its effect on compaction characteristics was ascertained. The proportions identified for silica fume and induction furnace slag are in the order of 5–20% of dry weight of soil with an incremental increase of 5%. The optimum percentage of silica fume was obtained from the compaction characteristics of the soil. After choosing optimum percentage of silica fume, further, the compaction and strength characteristics of the soil were ascertained with optimum percentage of silica fume and varying percentage of induction furnace slag, i.e., 10 to 20% with 5% increase. The results shows that the maximum dry density for BC soil + 10% SF + 20% IFS is 1.63 g/cc, and UCS for BC soil + 10% SF + 15% IFS is 138.76 kPa.

Keywords Black cotton soil · Silica fume · Induction furnace slag

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1 Introduction

The high volumetric changes of black cotton soil (BC soil) with respect to moisture content make it unfavorable for foundation. The expansion and shrinkage of BC soil with respect to moisture content cause pressure and settlement to structures founded on it, especially in lightly loaded buildings, canal linings, road pavements, etc. It has been found that the loss of property due to damage caused by BC soil is quite immense. Sometimes, the cost of retrofitting exceeds even the cost of the construction. Hence, it is very essential to control the volumetric changes of the soil [1]. The process of improving the soil property is known as stabilization and can be done by several techniques such as by mechanical means—soil replacement, mechanical compaction, surcharge loading and moisture control; by using chemical additives—natural, artificial and by addition of solid wastes. The first chemical stabilization in history was done by Mesopotamians and Romans. They mixed weak soil with pulverized lime stone or calcium for the construction of road. Since then, usage of lime in soil stabilization has been common. Disposal of industrial wastes to a safer zone is becoming a burning issue nowadays. The idea of controlling the expansion of soil or improving its properties by adding solid waste is not only economical but also environmental friendly. Solid wastes can be generally classified into three groups, namely agricultural waste, municipal waste and industrial waste apart from other categories of wastes. Agricultural wastes like rice husk ash, groundnut shell ash, coconut fibers, locust bean ash, etc., are very effectively used as soil stabilizers recently. Studies on rice husk ash as stabilizers show improvement in swell and shrinkage property of BC soil. The conclusion of the study was 10% RHA + 10% silica fume which was effective combination in improving the properties of BC soil. Industrial wastes like quarry dust, marble dust [2], fly ash and granulated blast-furnace slag [3–5]; (Shenbaga et.al 2004; Dutta 2011), Granulated blast furnace slag [6], silica fume, steel slag, rice husk ash, etc., [7] are also used widely as soil stabilizers. The inclusion of quarry dust and marble dust on BC soil shows better results on index properties and CBR values at 40% addition of these additives. Various researches on the combined effect of fly ash and GGBS concluded that with 10% FA + 20%GGBS gave better performance. Investigation on the effect of silica fume on BC soil reveals the improvement of swelling and strength characteristics of the soil. It was effective with 10% to 20% SF addition. All research studies projected the improvement of soil properties with the inclusion of solid wastes to the BC soil. In the present study, induction furnace slag and silica fume had been selected to study its effect on strength and optimum characteristics of black cotton soil.

India stands second in the world in castings and also one of the top 10 in terms of average production per plant. A large amount of waste has been generated by ferrous foundries. The Indian foundry produces around 6 million tonnes of castings annually. The annual production of slag is around 1.7 million, and it is estimated that around 5000 foundries are functional all over India. During the cast iron production, the slag generated is hazardous to the environment. The slag that formed in induction furnace melting is the result of complex reactions between silica, oxide of iron from steel scrap, other oxidation by-products from melting and reactions with refractory linings. The resulting slag consists of complex liquid phase. Slag contains Al_2O_3, MgO, SiO_2, Fe_2O_3, CaO & MnO [6]. The large amount of induction furnace slag is considered as waste. The skimmed off slag is often air cooled near production unit. This air-cooled slag, when powdered and the metal part separated, can be potentially used as stabilizers.

Silica fume (SF) is obtained as the bi-product in the reduction of high-purity quartz with coal in electric furnaces in the production of silicon and ferrosilicon alloys. SF is also obtained as a by-product in the production of other silicon alloys such as ferrochromium, ferromanganese, ferromagnesium and calcium silicon. When the high-purity quartz is reduced to silicon at temperatures up to 2000 °C, it produces SiO_2 vapors, which oxidizes and condenses in the low-temperature zone to fine particles containing silica. It consists of very fine transparent particles with high surface area. Because of its extreme fineness and high silica content, silica fume can be effectively used as pozzolanic material. This acts as a bonding agent and hence can be effectively used in soil for stabilization.

2 Materials Used

2.1 Black Cotton Soil

Black cotton soil was obtained from Bhalki, Karnataka (INDIA). According to Indian classification system, this soil was classified as inorganic clay of high plasticity (CH), and the properties of BCsoil are given in Table 1.

Table 1 Index properties of Black cotton soil

Index properties of black cotton soil	
Field moisture	34.5%
Liquid limit	65.4%
Plastic limit	20.3%
Shrinkage limit	14%
Plasticity index	45.1%
Specific gravity	2.6
Free swell index	40%
I.S. classification	CH
MDD	1.56 g/cc
OMC	25
UCS	95.36 kPa

Table 2 Physical properties of Silica fume

Physical properties of silica fume	
Particle size (typical)	<1 micron
Bulk density (as produced) (densified)	130–430 kg/m^2 480–720 kg/m^2
Specific gravity	2.2
Specific surface	15,000–30,000 m^3/kg
Chemical composition of micro-silica fume (%)	
Silica (SiO_2)	98.84
Alumina (Al_2O_3)	0.04
Calcium oxide (CaO)	0.63
Iron oxide (Fe_2O_3)	0.03
Potassium oxide (K_2O)	0.07
Magnesium oxide (MgO)	0.01

2.2 Silica Fume

Silica fume is also known as micro-silica, condensed silica fume, volatilized silica or silica dust. Silica fume is a very fine non-crystalline product available in wet or dry form. The physical properties and chemical composition of silica fume as obtained from the manufacturer are listed in Table 2 (Fig. 1).

2.3 Induction Furnace Slag

Induction furnace slag is a non-metallic co-product produced in the process of recycling of scrap iron. They are air cooled naturally and therefore crystalline. The crystallization takes place slowly, and hence, they are very hard. The induction furnace slag is powdered; metallic part separated and made it pass through 850 micron IS sieve. This slag was procured from Pathencheru Industrial Park in Hyderabad (Figs. 2 and 3).

In the present investigation, the effect of silica fume, induction furnace slag and their combinations on black cotton soil is being investigated. Optimum percentage of these combinations was obtained (Tables 3 and 4).

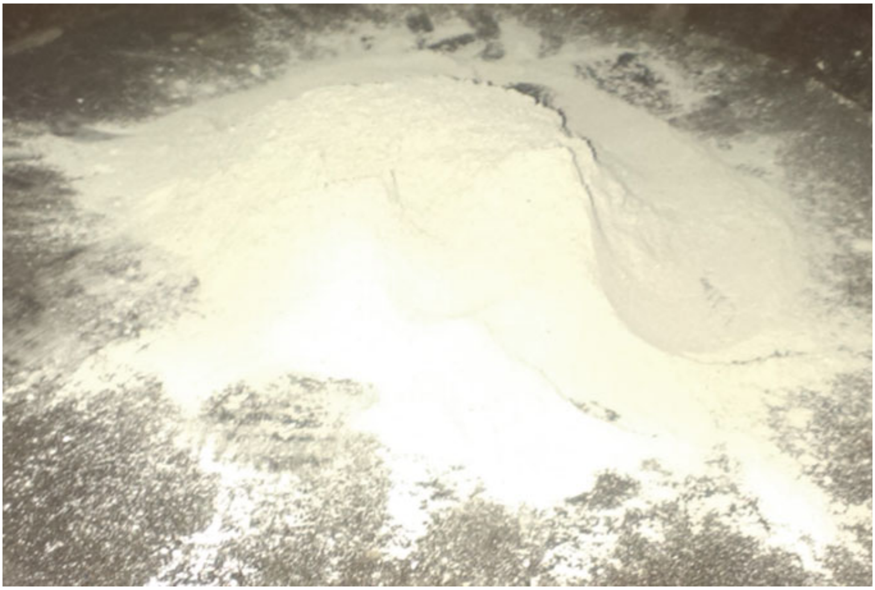

Fig. 1 Silica fume as obtained from manufacture

Fig. 2 Induction furnace slag

Fig. 3 Powdered slag

Table 3 Experimental program on compaction characteristics for black cotton soil

Sample composition
Plain BC soil
BC soil + 5% silica fume
BC soil + 10% silica fume
BC soil + 15% silica fume
BC soil + 20% silica fume
BC soil + 5% induction furnace slag
BC soil + 10% induction furnace slag
BC soil + 15% induction furnace slag
BC soil + 20% induction furnace slag
BC soil + 10% silica fume + 5% IFS
BC soil + 10% silica fume + 10% IFS
BC soil + 10% silica fume + 15% IFS
BC soil + 10% silica fume + 20% IFS

Table 4 Physical properties of induction furnace slag

Color	Black
Structure	Crystalline
Specific gravity	2.78

3 Methodology

3.1 Sample Preparation

Soil samples were collected from an excavation at a depth 1 m below the ground level. The organic matters, like tree roots and pieces of bark, were removed from the sample. Then, it is pulverized properly to make it drying faster. For different proportions of mixes, the additives were taken according to certain percentages by dry weight of soil as shown in Table 5. After the preparation of sample, compaction test and unconfined compression test were conducted on virgin soil and soil mixed with additives and the results were obtained.

3.2 Experimental Procedure

Basic laboratory tests for the identification of the soil, its index properties and strength properties were carried out on virgin black cotton soil. Then, the soil properties were again ascertained by mixing it with silica fume, induction furnace slag and its combinations with varying percentages of two additives. The proportions of additives were

Table 5 General chemical composition of slag

Na_2O	6.07
MgO	1.31
Al_2O_3	12.76
SiO_2	45.82
P_2O_3	0.09
SO_3	0.59
Cl	0.03
K_2O	0.27
CaO	4.19
TiO_2	0.55
MnO	1.79
Fe_2O_3	8.59
LOI	18.2

Table 6 Experimental program on UCS for black cotton soil

Sample composition
BC soil alone
BC soil + 10% SF + 10% IFS
BC soil + 10% SF + 15% IFS
BC soil + 10% SF + 20% IFS

taken in the order of 5, 10, 15 and 20% for ascertaining its compaction characteristics. Experimental program for compaction and UCS is given in Tables 5 and 6, respectively.

3.3 Test Procedure

All tests were conducted in laboratory as per the Indian Standard Code. The basic properties of the soil like free swell index, specific gravity, moisture content, liquid limit, plastic limit and shrinkage limit were determined as per IS 2720 Part-1,2,5,6, respectively, in the laboratory. The compaction characteristics of the soil are determined by Proctor test conducted in the laboratory as per IS: 2720 (Part VIII-*1980). The* unconfined compressive strength of the soil was determined by conducting unconfined compression test on the soil as per IS 2720 Part –X.

3.3.1 Standard Proctor Test Confirming to iS 2720 Part-VIII-1980

The prepared soil specimen is mixed with varying percentage (5, 10, 15, 20% of dry weight of soil) of silica fume, induction furnace slag independently and then with optimum percentage of silica fume and varying % of induction furnace slag to ascertain the compaction characteristics in each combination. The soil is mixed with additives in its dry form; after mixing thoroughly, initially, a water content of 10% is added, and the corresponding dry density is found out, and the procedure is repeated for 15, 20, 25, 30% for obtaining the compaction curve for each mixed proportions. The experimental program is given in table 5.

3.3.2 Unconfined Compressive Strength Test Confirming to iS 2720 Part-X.

The unconfined strength of the prepared soil is ascertained by making a cylindrical mold of 35 mm diameter and 76 mm long. It was then statically compacted to its OMC and MDD. The sample was then kept in a desiccator for about an hour, and the sample is subjected to UCS test.

Since the soil did not show relevant changes in its OMC and MDD characteristics after 10% percentage inclusion of SF in the soil, optimum percentage is considered as 10% for silica fume. And since the compaction characteristics improved after the addition of IFS, the lowest percentage ie 5% had been omitted. Along with optimum percentage of silica fume and 10% of dry weight of soil as constant, the variation of 10%, 15% and 20% percentage of IFS were added to the soil for the determination of UCS characteristics of the soil.

The prepared soil was mixed thoroughly with 10% of SF at first and then added the variation of IFS. Cylindrical soil specimen with dimensions 35 mm diameter and 80 mm long had been prepared by compacting it to its OMC and MDD. The prepared specimens were kept in a desiccator for 3 days, 5 days, 7 days, 9 days and 12 days as curing period and then UCS strength ascertained for each specified curing day. The procedures were repeated for other combinations of soil and 10% SF with different percentages of 15% IFS and 20% IFS.

4 Results and Discussion

Variation of compaction properties and UCS properties of black cotton soil with the addition of silica fume, induction furnace slag and their combination is shown in Figs. 4, 5 and 6, respectively.

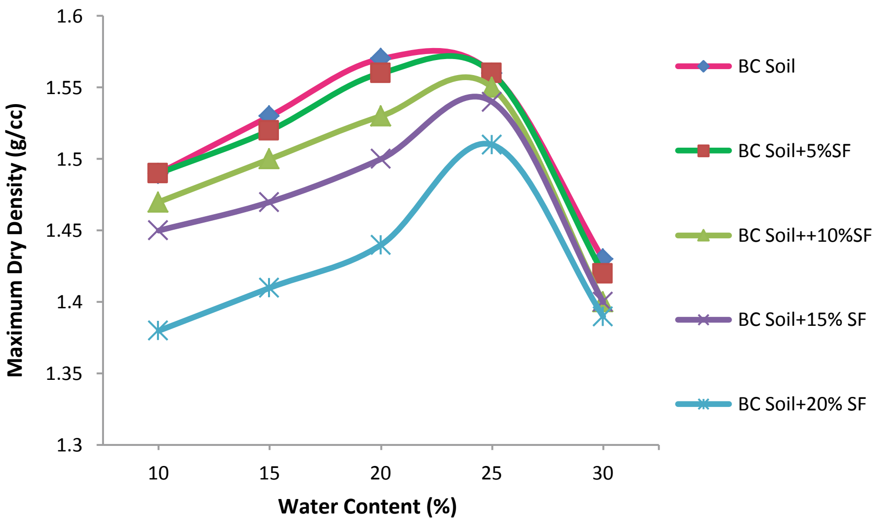

Fig. 4 Dry unit weight against water content of BC soil treated with silica fume

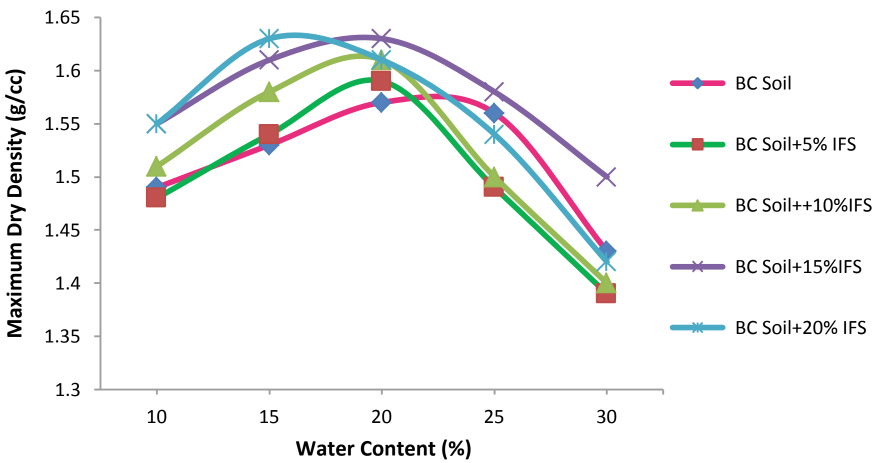

Fig. 5 Dry unit weight against water content of BC soil treated with induction furnace slag

4.1 Effect of Silica Fume on the Compaction Characteristics of Black Cotton Soil

The variation in OMC and MDD is shown in Fig. 4. When silica fume was treated with black cotton soil, there was a gradual increase in the OMC and decrease in maximum dry density of soil. When 20% silica fume was added to BC soil, the

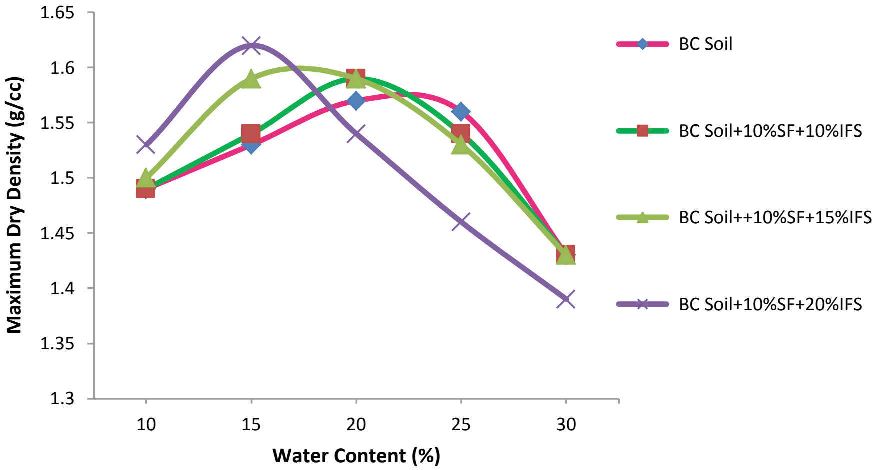

Fig. 6 Dry unit weight against water content of BC soil treated with silica fume and induction furnace slag

percentage increase in OMC is about 27% and percentage decrease in MDD is 3.8%. This may be due to the increase in percentage fines in the soil.

4.2 *Effect of Induction Furnace Slag on the Compaction Characteristics of Black Cotton Soil*

The variation in OMC and MDD is shown in Fig. 5. It was inferred that there was a gradual decrease in the OMC and increase in maximum dry density of soil, with increasing percentage of IFS in black cotton soil. The percentage decrease in OMC observed is 27% and increase in MDD is 3.7%, when 20% induction furnace slag was introduced. This may be due to the increase in coarse grains in the soil. These may also be due to the soil type, the related interchangeable cations and the relative amount of silicate clay mineral in the samples.

4.3 *Effect of Silica Fume and Induction Furnace Slag on the Compaction Characteristics of Black Cotton Soil*

The variation of OMC did not show much variation with percentage increase in silica fume from 5 to 15%, and there was a constant decrease in MDD with increase in percentage of silica fume. Hence, 10% silica fume was considered as optimum percentage. Thereafter, the BC soil was treated with 10% SF and varying percentage

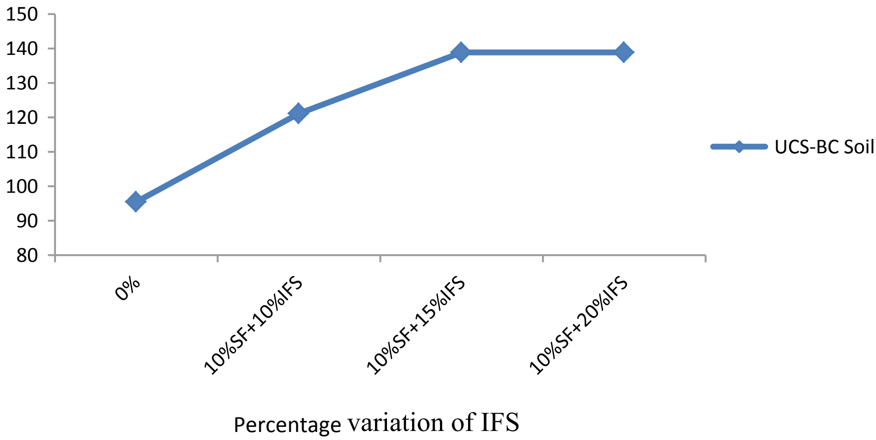

Fig. 7 Variation of UCS (kPa) with different percentages of additives

of IFS to obtain the compaction and strength characteristics. The variation of the combination of additives is shown in Fig. 6. When the soil was treated with silica fume and IFS, OMC of the soil decreased gradually. The MDD shows constant increase with increasing % of SF and IFS in the soil by the combined effect of silica fume and induction furnace slag.

4.4 *Effect of Silica Fume and Induction Furnace Slag on the UCS of Black Cotton Soil*

The effect of UCS is shown in Fig. 7; there is a constant increase in strength for black cotton soil with percentage of additives and with increase in curing period. There were not significant changes observed after (10% SF + 15% IFS) addition. Hence, the optimum percentage of silica fume and induction furnace slag in the soil was found to be 10% SF + 15% IFS for UCS. The percentage increase in strength is 45.5% compared to virgin soil.

5 Conclusion

1. Due to addition of 20% silica fume with BC soil, the percentage increase in OMC is about 27% and percentage decrease in MDD is 3.8% compared to the BC soil alone. When IFS was used to treat the soil, OMC of the soil decreased and MDD was increased with % increase in IFS.

2. OMC of the soil decreased gradually, when the soil is treated with silica fume and IFS. The MDD shows constant ascend with increase in % of SF and IFS in the soil. The maximum dry density for (BC soil + 10% SF + 20% IFS) is 1.63 g/cc.
3. The percentage increase in strength for (BC soil + 10% SF + 15% IFS) mix is found to be 45.5% compared to its virgin soil.
4. Optimum value of UCS was observed for (BC soil + 10% SF + 15% IFS) mix.

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Experimental Studies on Influence of Alccofine and Calcium Chloride on Geotechnical Properties of Expansive Soil

R. Suresh and V. Murugaiyan

Abstract Expansive soil deposits occur in the arid and semi-arid regions in the world. They cover a major portion on the geographical area in the world and about one-fifth the area of India (approximately 300,000 km^2); such soils are popularly perceived as black cotton soils and found extensively in Andhra Pradesh, Gujarat, Karnataka, Madhya Pradesh, Maharashtra and Tamil Nadu. The present study is to elucidate and efficacy of materials as an additive in improving the engineering characteristics of expansive soils. An experimental programme has evaluated the effects of alccofine-1203(3%, 6% and 9%) and $Cacl_2$ (0.25%, 0.5%, and 1.0%) contents on the FSI, swelling potential, swell pressure, plasticity, compaction, strength, hydraulic conductivity, SEM, XRD, cation exchange capacity (CEC), characteristics of expansive soil. Both admixtures were added independently and blended to the expansive soil. Mixing of both admixtures into expansive soil results shown that plasticity index, hydraulic conductivity, swelling properties of blends decreased and dry unit weight and unconfined compressive strength is increased in combination of soil + 6% of alccofine-1203 + 1% $Cacl_2$, but further more addition of alccofine-1203 and $Cacl_2$ leads to decrease in the unconfined compressive strength. It was found that the optimum quantity of material for favourable combination of soil + 6% of alccofine-1203 + 1% $Cacl_2$ was taken for further study in view of its economy due to lower $Cacl_2$ content.

Keywords Expansive soil · Alccofine-1203 · Calcium chloride · CEC

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1 Introduction

Expansive soils are known worldwide for their volume change behaviour due to moisture fluctuation because of their intrinsic mineralogical behaviour [1]. These types of soils are found mainly in the arid and semi-arid region [2] such as Australia, Canada, China, India, South Africa and the USA. India has extensive track of expansive soils known as black cotton soil covers about twenty percentage of the total land area [3]. Expansive soils presence in black colour, due to its black colour which is a result of high iron and magnesium minerals acquire from basalt [4]. Expansive soils are extensive specific surface area and high cation exchange capacity [5, 6]. Expansive soil contains clayey minerals such as montmorillonite which increases in volume during wetting. This change in volume can exert sufficient stress on a building, side walk, driveways, basement floors, pipelines and foundations to cause damages. Since the expansive soils are found worldwide, the challenges to the civil engineers in one felt around the globe. If not adequately treated, expansive soils may act as a natural hazards resulting in several damages to structures [7, 8]. The annual cost of damages to the civil engineering structures is estimated as 150 million in the USA and many billions of dollars worldwide [9]. Under the moisture ingress and digress, a building founded on expansive soil undergoes differential movements caused by alternate swell/shrink behaviour of soil causing several structural damages. Many reported data are available on the heave profile of soil at the surface, at various depths from the ground surface, and on covered areas [2, 10, 11], it is generally observed that the amplitude of soil movement decreases with depth and there is an increase in time lag with movement at depth compared with that at the surface. To date, distress problems related to this type of soils is quite immense have ensue in the loss of billions of dollars in repairs and rehabilitation [12].

Various innovative techniques such as special foundations that include belled piers, drilled piers, friction piles and moisture barriers have been developed to mitigate the problem posed by the expansive soils [13]. Apart from this techniques, stabilization of expansive soils with various additive including fly ash, lime, cement, calcium chloride, has also met with considerable success [14–16]. Physical–chemical mechanism of the lime treatment of soil are well-established [17] four mechanisms [cation exchange, flocculation, carbonation, pozzolanic reactions] which are generally associated with the modification and stabilization of lime treated soils. Further, there has been an increased in the awareness of environmental and ecosystem degradation due to huge production, and storage of waste materials is also initiated to stabilize problematic soil, alone or in combination with lime, effectively and economically [18–25]. It has been felt by researchers that strong electrolytes such as potassium chloride, magnesium chloride, zinc chloride, sodium hydroxide, ferric chloride and calcium chloride could be tried instead of lime [26–30] and strong electrolytes are readily soluble in water and hence could supply adequate cations for exchange reactions. Industrial by-product material such as fly ash [31–33], GGBS [34, 35], cement kiln dust [36, 37], lime stone dust [38] as additive is becoming more popular due to their relatively low cost; additionally, CO_2 emission can be reduced significantly by the increased use of such supplementary cementing materials currently wasted in lagoons and landfill sites. The most important feature in the stabilization of clay soils is the ability of stabilizer to provide a sufficient amount of calcium [39]. Granular pile-anchor (GPA) technique has been a recent innovation over the conventional granular pile, modified into anchors [40]. Stabilization of expansive soil with admixtures controls the potential of soils for a change in volume. In recent years, a number a stabilizers from various industries have been developed for the purpose of soil stabilization. Stabilizers can be amended with activators like lime or cement to enhance their cementitious and pozzolanic properties. [41] improved the unconfined compressive strength of expansive soil by stabilizing them with alccofine. A study by [3] used the different percentages of $Cacl_2$ and RHA mixtures to stabilize clay soil as cushions (CNS) in below footings, pavement slabs and behind canal lining.

The purpose of this study is to investigate the influence of inclusion of alccofine in conjunction with calcium chloride ($Cacl_2$) in the stabilization of expansive soils. In India, an industrial product alccofine material is manufactured by Ambuja cement private limited. The majority of this material is utilized in the high-performance concrete structures either as a cement replacement or as an additive to improve concrete properties in both fresh and hardened states and soil stabilization purpose [3, 41], while $Cacl_2$ is mainly used to reduce the swelling and increase the shear strength of expansive soil for soil stabilization. The main reason for their underutilization is the lack of pozzolanic reactivity [42]. Alccofine is ultrafine ground granulated blast furnace slag (UFGGBS) which performs superior than all other mineral admixtures used in India. It is a micro-fine material of particles size (range 0–17 microns) much finer than other hydraulic materials like cement, lime, fly ash [42]. On the other hand, $Cacl_2$ is the hygroscopic material and hence is pre-eminently suited for stabilization of expansive soils, because it absorbs water from the atmosphere and prevents shrinkage cracks occurring in expansive soils during summer season [2]. The combination of the two materials can be more beneficial when used as a stabilizing agent than using them individual. However, no studies on the joint activation of alccofine and $Cacl_2$ as stabilizing agents for expansive soils have been published to date. An attempt has been made in this study to utilize mixture of alccofine and $Cacl_2$ as binder to stabilize expansive soil. The influence of the binder on FSI, swelling potential, swell pressure, plasticity, compaction, strength, hydraulic conductivity, SEM, XRD, cation exchange capacity (CEC) characteristics of expansive soil has been taken into account for evaluating performance.

2 Materials and Methodology

2.1 Materials

Expansive Soil: The expansive clay soil is collected from Kirumambakkam, which is located in Puducherry, India. The soil is collected in a dry condition at a depth of 1 m below the ground level and preserved in the laboratory. The expansive soil is identified the index and engineering properties of expansive soils as shown in Table 1.

Alccofine: Alccofine is ultrafine ground granulated blast furnace slag (UFGGBS), which performs superior than all other mineral admixtures used in India. It is manufactured by Ambuja cement private limited in India. Chemical composition and physical properties are tested by alccofine micro-materials, Pissurlem, Goa. Alccofine-1203 properties are given in Table 2.

Calcium Chloride: The chemical composition of calcium chloride is $Cacl_2$. It is a hygroscopic material; it also absorbs water from the air and releases heat when it is dissolved in water.

Table 1 Physical properties of soil

Properties of soil	Results
Sand (%)	12
Silt (%)	30
Clay (%)	58
Specific gravity	2.60
Liquid limit (W_L)	59%
Plastic limit (W_P)	29%
Shrinkage limit (W_S)	12.5%
Free swell index (FSI)	25%
Water absorption (W_A)	53.69%
Cation exchange capacity (CEC) meq/100 g	55
Unified soil classification (USCS)	CH
OMC (%)	18.19
MDD (kn/m^3)	15.73
UCS (kPa)	157
Swell potential (%)	5.29
Swell pressure (kPa)	150
Hydraulic conductivity cm/sec	1.58×10^{-6}

Table 2 Physical and chemical properties of Alccofine-1203

Properties	Results
Physical properties	
Particle size Distribution	
D10	1.5
D50	4.3
D90	9.0
Specific gravity (g/cc)	2.88
Bulk density (kg/m^3)	680
Chemical properties	
SiO_2	35.6%
Al_2O_3	21.4%
Fe_2O_3	1.3%
CaO	33.6%
SO_3	0.12%
MgO	7.98%

2.2 Testing Methodology

Different tests can be used to characterize the index and engineering properties of stabilized soils. The present study focuses on evaluating the physical properties, compaction, strength and swell/shrink behaviour. Experimental investigations have been carried out on expansive soil with the addition of varying percentages of calcium chloride (0.25%, 0.50% and 1.0%) and alccofine (3%, 6% and 9%).

The specific gravity, Atterberg limits, compaction, unconfined compressive strength (UCS), consolidation and swelling characteristics of clay soil sample was determined according to the Indian Standards [43–47].

The water absorption (W_A) of the soil mixed with both $Cacl_2$ and alccofine-1203 was added independently and blended to the expansive soil.

A water absorption (W_A) equation is developed and recommended by [48]. Water absorption equation is

$$W_A = 0.91\, W_L$$

where W_L is liquid limit.

To determine the specific surface area (SSA) of the soil binders mixtures, a specific surface area is developed and recommended by [49]. The cation exchange capacity (CEC) of the soil admixtures blended samples is determined. The cation exchange capacity (CEC) is developed and recommended by [50]. Finally, detailed micro-analysis (XRD, SEM and FTIR) has been carried out to elucidate the mechanism of strength variation through change in mineralogy and microstructure, respectively.

3 Results and Discussions:

3.1 Index and Compaction Characteristics

The influence of alccofine and $Cacl_2$ on Atterberg limits of expansive soil is shown in Table 3. Results show that liquid limit decreases and plastic limit increases; hence, the difference between liquid limit and plastic limit is the plasticity index. Plasticity index is reduced by about 67% when the soil is blended with 6% alccofine + $Cacl_2$ 1%.

The compaction characteristics of untreated and treated soils are shown in Table 3. The results of compaction show that the MDD increases from 15.73 kN/m^3 to 16.92 kN/m^3 and optimum moisture content is reduce from 18.19% to 16.5% with increase of 6% alccofine and 1% $Cacl_2$ binder, that is, for sample which shows maximum strength.

Table 3 Effects of soil-admixtures blended on index and engineering properties

$Cacl_2$	Alccofin	W_L %	W_P %	W_S %	PI %	MDD kN/m^3	OMC%	UCS (kPa)	S%	S_P kPa	W_A %
0	0	59.0	34.5	12.5	24.5	15.73	18.19	157	5.29	150	53.69
	3	55.0	35.0	13.3	20.0	15.85	17.75	216	3.22	120	50.05
	6	49.0	35.5	16.0	13.5	15.95	17.45	245	1.23	095	44.59
	9	47.0	36.0	22.5	11.0	16.15	17.24	241	0.75	075	42.77
0.25	0	54.0	37.0	14.0	17.0	15.85	18.05	245	2.17	115	49.14
	3	51.0	38.5	15.0	12.5	16.05	17.76	300	1.02	098	46.41
	6	49.0	39.0	19.0	10.0	16.30	17.25	327	0.59	065	44.59
	9	48.0	39.0	24.5	9.0	16.45	17.10	324	0.46	096	43.68
0.5	0	52.5	38.0	14.5	14.5	15.90	17.65	306	1.47	045	47.77
	3	50.4	38.5	16.0	11.9	16.34	17.34	359	0.90	038	45.86
	6	48.0	40.0	18.8	8.0	16.70	16.80	384	0.34	022	43.68
	9	49.0	41.0	24.0	8.0	16.90	16.40	376	0.11	018	44.59
1.0	0	51.0	39.0	18.0	12.0	15.80	17.40	352	0.78	032	45.68
	3	49.0	41.0	21.0	8.0	16.30	16.80	401	0.17	012	44.59
	6	47.0	39.0	22.5	8.0	16.92	16.50	418	0	0	42.77
	9	49.0	42.0	22.8	7.0	16.95	16.24	406	0	0	44.59

Note W_L = Liquid limit; W_P = Plastic limit; W_S = Shrinkage limit; PI = Plasticity index; MDD = Maximum dry density; OMC = Optimum moisture content; UCS = Unconfined compressive strength; W_A = Absorption water content; S = Swelling potential; S_P = Swell pressure

Table 4 Properties obtained for optimum soil-alccofine-$Cacl_2$ mix

Properties	Soil	93% soil + 1%$Cacl_2$ + 6% alccofine-1203
Sand (%)	12	10.3
Silt (%)	30	35.2
Clay (%)	58	54.5
Specific gravity	2.60	2.72
Liquid limit (W_L)	59%	47%
Plastic limit (W_P)	29%	39%
Shrinkage limit (W_S)	12.5%	22.5%
Plasticity Index (PI)	25%	8%
Water absorption (W_A)	53.69%	42.77%
Cation exchange capacity (CEC) meq/100 g	55	18
Unified soil classification	CH	CI
OMC (%)	18.19	16.5
MDD (KN/m^3)	15.73	16.92
UCC (kPa)	157	418
Free swell index (FSI)	25%	0
Swell potential (%)	5.29	0
Swell pressure (kPa)	150	0
Hydraulic conductivity cm/s	1.58×10^{-6}	4.3×10^{-5}

3.2 Unconfined Compression Strength

Unconfined compressive strength (UCS) tests were conducted with alccofine-1203, and $Cacl_2$ was added independently and blended to the expansive soil samples. UCS test was performed on both intrinsic soil and chemically treated soil. The UCS value for intrinsic soil is 157 kPa. The percentage of alccofine (3, 6 and 9%) and $Cacl_2$ (0.25, 0.5 and 1.0%) was added by dry weight of the soil. The UCS values are shown in Table 3. Optimum increase was noticed at 6% alccofine and 1% $Cacl_2$. The UCS strength was increased from 157 to 418 kPa. Beyond 6% of alccofine with 1% $Cacl_2$ resulted in a slight decreased in UCS values shown in Table 4.

3.3 Swell Behaviour

The swell behaviour of soil and mixed with different percentages of alccofine and $Cacl_2$ is presented in Table 3. The maximum swell potential of intrinsic soil is 5.29% and swell pressure is 150 kPa. With addition of various percentages of alccofine and

$Cacl_2$, swell of soil decreases gradually and completely brings to halt beyond addition of alccofine 6% with 1% of $Cacl_2$. Beyond alccofine 6% with 1% of $Cacl_2$, complete elimination of swell is due to availability of adequate calcium, not only for cation exchange reaction but also for the formation of pozzolanic reaction compounds.

Cation Exchange Capacity: Exchangeable cations (i.e. Ca, Mg, Na and K) are determined displacing these from soil colloids with NH_4. This is done by shaking the soil with 1 N NH_4OAc adjusted to pH 7.0.

3.4 Mineralogical and Microstructural Analysis

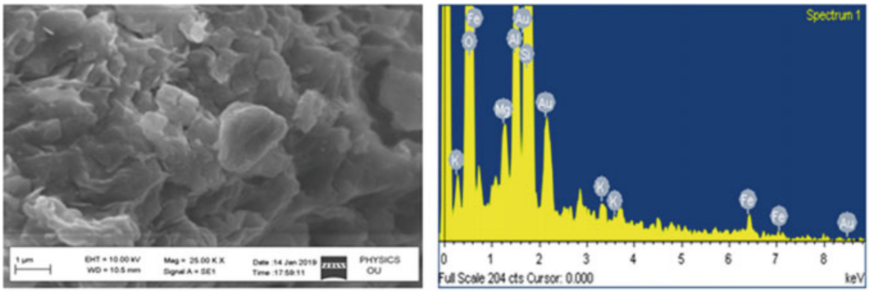
SEM with EDAX Analysis: SEM and EDAX spectrum analyses for clay soil, alccofine and clay soil + alccofine 6% + $Cacl_2$ 1% are shown in Fig. 1a–c. The microstructural studies were carried out in order to observe the individually, and changes in the soil is blended with admixture of 0 days. Eminent peaks Fe, Au, Al are observed in (1a), and Fe, Au, Al, Si are observed in clay soil. In alccofine (1b), Ca, Mg, Si, Al eminent peaks are observed. In combination of soil blended with admixture (1c) is observed eminent peaks are Fe, Au, Si, o, Al. The test was performed mainly for the identification of the various cementations compounds on the soil stabilized with 6% alccofine + $Cacl_2$1% binder, that is, for sample which shows maximum strength.

XRD: X-ray diffraction peaks identify for clay soil, alccofine and clay soil + alccofine 6% + $Cacl_2$ 1%. The most important peak traced was related to CH which was identified at $2\Theta = 26°$ to $36°$ [51] as can be seen from Fig. 2a–c; the addition of alccofine-1203 and $Cacl_2$ in the soil causes CH-related peaks to appear at the aforementioned 2Θ. It has been carried out to confirm the formation of new minerals which can play a significant role of strength improvement behaviour calcium stabilized for soil admixture. The intensity has increased for calcium chloride and alccofine-1203 materials treated when compared with the clay soil, which is all evident from X-ray data (Table 4).

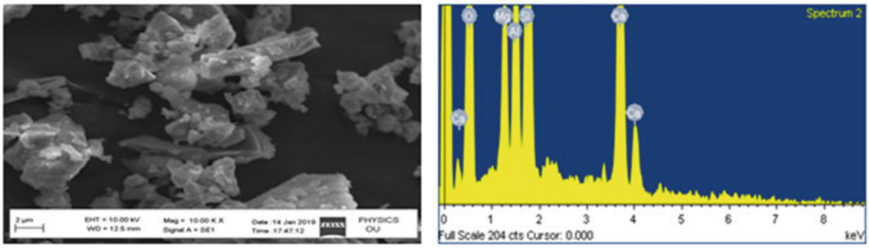
4 Conclusion

In this study, based on the laboratory investigation, a series of tests were performed to study the effect of alccofine-1203 and $Cacl_2$ on the swelling properties and strength behaviour of soils. Based on the results presented in this paper, the following conclusions are made:

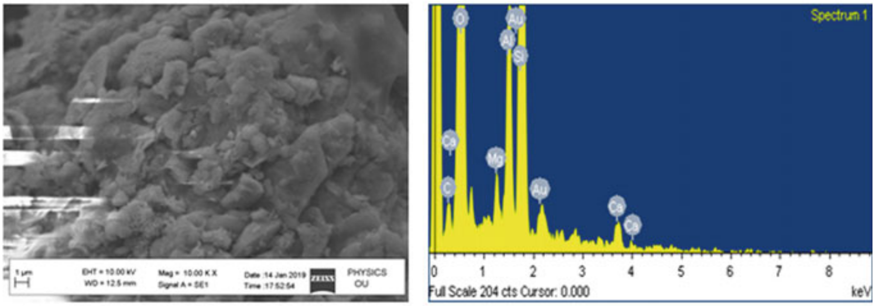
1. The addition of alccofine-1203 and $Cacl_2$ to the soil decreased liquid limit and plasticity index while increasing the shrinkage limit. The optimum moisture content (OMC) was found to decrease, while the maximum dry density (MDD) increased with increasing with binding content.



(a) SEM with EDAX analysis for soil



(b) SEM with EDAX analysis for alccofine-1203



(c) SEM with EDAX analysis for Soil + Alccofine-1203 6% + 1% $Cacl_2$ for 0 days

Fig. 1 **a** SEM with EDAX analysis for soil; **b** SEM with EDAX analysis for Alccofine1203; **c** SEM with EDAX analysis for soil + Alccofine 1203 + $Cacl_2$ 0 days

2. Unconfined compressive strength (UCS) tests were conducted with alccofine, and $Cacl_2$ was added independently and blended to the expansive soil samples. The UCS value for intrinsic soil is 155 kPa. Optimum increase was noticed at 6% alccofine and 1% Cacl2. The UCS strength was increased from 155 to 482 kPa. Beyond 6% of alccofine with 1% $Cacl_2$ resulted in a slight decreased in UCS values.
3. The swell behaviour of soil: swell potential is reduced from 5.29% to 0, and swell pressure is reduced from 150 kPa to completely bringing to halt beyond addition of alccofine 6% with 1% of $Cacl_2$.

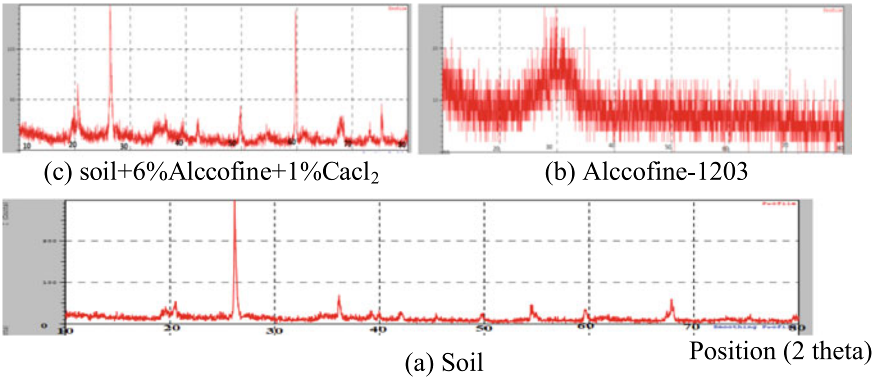


Fig. 2 XRD analysis for **a** soil; **b** Alccofine; **c** soil + 6%Alccofine + 1%Cacl$_2$

The results presented that the type and amount of additives play a crucial role in the stabilization process. It is immensely important to select the additive based on different properties, and there chemical composition is the most important among these properties. The use of cohesive non-swelling soils (CNS) cushions below the lightweight structures is well accepted. It is to be noted that CNS layer is not present on expansive soil; there is a possibility of differential heave and loss of shear strength at the edges of foundation. In the view of severe scarcity for suitable cohesive non-swelling soils (CNS) at several project sites, an alternative cushion material is proposed to be prepared at the site using the intrinsic soil (expansive soil) by admixing with it 6% alccofine and 1% Cacl$_2$ by dry weight of the soil. Based on the favourable results obtained, it can be concluded that the expansive soil with alccofine and Cacl$_2$ can be considered as an effective cohesive non-swelling soil (CNS) for pavements, sidewalks and floorings.

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Effect of Various Factors Affecting Electrokinetics Dewatering of Soil Using Conductive Geotextile

Kalpana P. Patel, L. S. Thakur, and D. L. Shah

Abstract Electrokinetic principles are being used in many geotechnical engineering application such as dewatering, decontamination, solid enhancement. Soils in the presence of high water content tend to loose the interparticle bonds, resulting in high compressibility of the soils, low bearing capacity and low permeability of the soil. All these issues lead to increase in settlement values or differential settlements. The presented paper used the process of electrokinetic dewatering to enhance the draining of water and thereby improve the strength of soft soil. The electrokinetic geosynthetics used give technical benefits over conventional electrodes in that they can be formed as strips, sheet, blankets, etc. They are light and easy to install. This paper describes laboratory experiment using polyester cotton-based materials intertwined with copper wires as a cathode and carbon electrode as anode with three types of soil under present investigation. The various factors studied for the same are experimental duration, electrode, voltage gradient and re-run analysis. It is observed that there is an increase in the percentage removal of water content with increase in value of each of the factor with increase in shear strength of the soil in case of effect of electrode.

Keywords Dewatering · Copper wire · Polyester cotton · Electrokinetics

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1 Introduction

Geotechnical properties of problematic soils such as soft fine-grained and expansive soils are improved by conventional remediation methods which have been known successful in minimizing several damages; however, they are expensive, time-consuming and may be difficult to implement in some existing structures. The problematic soil is removed and replaced by a good-quality material or treated using mechanical and/or chemical stabilization. Different methods can be used to improve and treat the geotechnical properties of the problematic soils (such as strength and the stiffness) by treating it in situ. These methods include densifying treatments (such as compaction or preloading), pore water pressure reduction techniques (such as dewatering or electro-osmosis), the bonding of soil particles (by ground freezing, grouting, and chemical stabilization), and use of reinforcing elements (such as geotextiles and stone columns) (William Powrie 1997). Amongst these methods, we preferred method of dewatering of soil using electrokinetic and/or conductive geotextile. In this regard, electrochemical or electrokinetic (EK) treatment method can be used as an alternative soil treatment method for remediation of those deficiencies underneath building foundations, roads, railways or pipelines. The use of this technique involves an approach with minimum disturbance to the surface while treating subsurface contaminants and improving the engineering characteristics of subsurface soils. The ability of electrokinetic phenomena to transport water, charged particles and free ions through fine-grained soils has been well established following their discovery by Reuss (1809). He discovered that under the influence of an applied electrical potential, water moves through the soil capillaries from the positive side to the negative side of the cell. The phenomenon of electro-osmotic flow was presented by Helmholtz in 1879, modified by Pellat in 1904, and refined by Smoluchowski in 1921. The theory presented by Smoluchowski is widely known as the Helmholtz–Smoluchowski theory. This theory deals with the electro-osmotic velocity of a fluid in soil media under the application of an electrical gradient. In 1930s, electro-osmosis was applied to fine-grained materials for soil stabilization in earthworks and foundation engineering (Gent 1998). In 1939, Casagrande demonstrated that applying electrokinetics to fine-grained soils with high water contents resulted in an increase in the effective stress within the soil through the generation of negative pore water pressures. He used this to increase soil shear strength and thus stabilize steep railway cuttings. In this technique, conductive geotextile was used to pass electricity through soil media for draining water through it. Nowadays, in civil engineering, geotextiles are vastly used and their modern uses have started with the advancement of synthetic and polymeric products and their ever-increasing application in different forms and areas of civil engineering was initiated only a few decades ago. Their use has expanded rapidly into nearly all areas of civil, geotechnical, environmental, coastal and hydraulic engineering. Geotextiles are effective tools in the hands of the civil engineer that have proved to solve a myriad of geotechnical problems.

Table 1 Index properties of soil

Type of Soil	LL*	PL*	SL*	FSI*	IS classification	MDD (gm/cc)	OMC (%)
Kaolinite	56%	31%	16%	25%	CI	1.48	26.91
Black cotton Soil	56%	27%	11%	75%	CH	1.64	18.42
Silt	30%	21%	21.63%	11%	MI	1.72	12.12

LL liquid limit; *PL* plastic limit; *SL* shrinkage limit; *FSI* free swell index

2 Materials

The silty soil was collected from the technology college campus of Maharaja Sayajirao University at Vadodara, Gujarat. The kaolinite clay was procured near from commercial market, Bhavnagar, Gujarat. The black cotton soil was procured near Parekha Village, Kayavarohan, Vadodara. The index properties of silty soil, kaolinite clay and black cotton soil are listed in Table 1.

3 Experiment Test Set-Up

For the experiments, three different types of soils were selected. For the simplicity in understanding the project, the whole quantum of work is being divided into various phases. Experiment details are given in Table 2.

Table 2 Schedule of experiments

Phase	Effect		Soil	Type of supply	Voltage	Experimental duration	Type of geotextile
1	Voltage gradient		Soil 1	DC	8, 15, 19.5, 23, 30	3	Polyester Cotton and Copper Filament
			Soil 2				
			Soil 3				
2	Experimental duration		Soil 1	DC	30	3, 7, 10, 14	
3	Effect of electrode		Soil 1	DC	30	3	
4	Effect of re-run (GR-reotextile for re-run)	GR-1 GR-2 GR-3	Soil 1	DC	30	3	

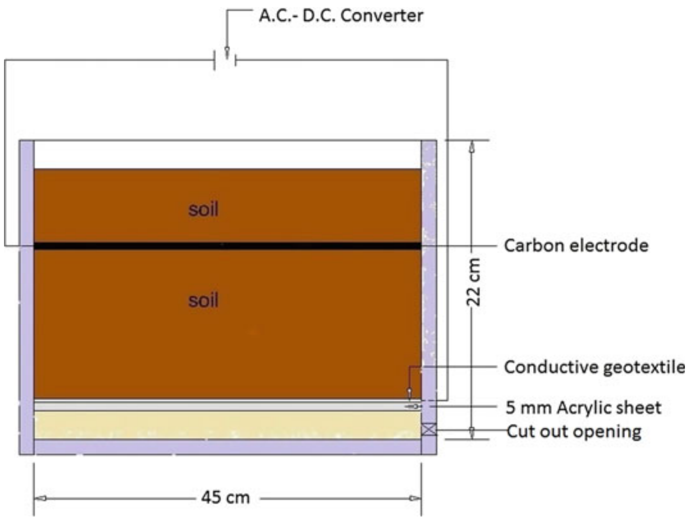


Fig. 1 Schematic diagram of dewatering set-up

3.1 Laboratory Models

Four laboratory scale models of size 45 cm × 30 cm × 22 cm were prepared using 12 mm waterproof PVC sheet (Fig. 1). In this experiment work, for the electrokinetic, dewatering should be done by using conductive geotextile (polyester cotton with copper wire), which were prepared by handloom weaving (Figs. 2, 3).

3.2 Set-Up and Conduction of Experiments

Each consisted of a filter chamber provided at a distance of 1.5 cm from the bottom using an acrylic sheet as a separator for allowing free flow of water during the process (Fig. 4). Standard quality filter paper was placed over this partition to restrict the flow of soil paste and soil particles into the filter chamber which was filled with white cement and OPC. Before using the model for experiments, they were checked for water tightness. Proper remedial measures were taken to avoid leakage, if any. Conductive geotextile (Fig. 5) was placed on the acrylic sheet to act as cathode with a carbon electrode (Fig. 6) being place at a distance of 15 cm from the bottom to act as an anode after the mould was filled with soil at its liquid limit. The electrodes were then connected using standard flexible copper wire to an AC-DC convertor unit. A 10 mm diameter and 30 cm long scarifying carbon electrode as anode and conductive geotextile as cathode were used for the conduction of current. The model had an arrangement to collect water at the bottom (Fig. 7) which was used to measure the volume of water flow as a cross check for water removal measured with respect

Fig. 2 Conductive geotextile with copper wire

Fig. 3 Experimental model

to water content removal calculated using temperature transmitter. The test duration was three days for all experiments. Various readings such as ambient temperature, ambient humidity, and current were taken on daily basis at fixed time.

Fig. 4 Separator disc (acrylic sheet) on base of model

Fig. 5 Placement of conductive geo textile (cathode)

Fig. 6 Placement of rod (anode) in soil

Fig. 7 Arrangement to collect water at the bottom

4 Experiment Test Result

Four phases of experiments investigation of electrokinetic dewatering carried out on the different soil have been analysed and discussed here. In these experiments, various factors considered for study in the work included effect of voltage gradient, electrode, experimental duration, re-run analysis, etc.

4.1 Effect of Voltage Gradient

Twelve experiments were performed for this phase using four different soils with same type of woven geotextile. The maximum temperature was observed in soil 2 at 30 V, and minimum temperature was observed in soil 1 at 15 V. The maximum percentage of removal of water was observed 21.14% in Soil 1 (Fig. 8), 67.07% in Soil 2 (Fig. 9), 37.59% in Soil 3 (Fig. 10), 26.23% in Soil 4 (Silt + Kaolinite) at 30 V up to three days.(Fig. 11) The maximum water removal was observed in Soil 2 at 30 V, and minimum water removal was observed in soil 1 at 15 V.

4.2 Effect of Experimental Duration

In this phase, five experiments were performed with and without electrokinetic at different durations (days) of test using same geotextile for Soil 1. The percentage of removal of water was observed without electrokinetic as 6.35%, 27.84%, 32.78%,

Fig. 8 Comparison of removal of water effect of voltage gradient (soil 1)

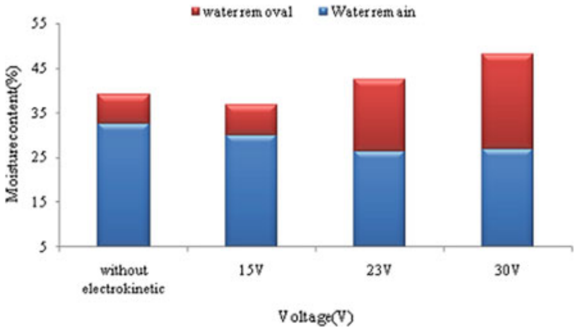


Fig. 9 Comparison of removal of water-Effect of voltage gradient (soil 2)

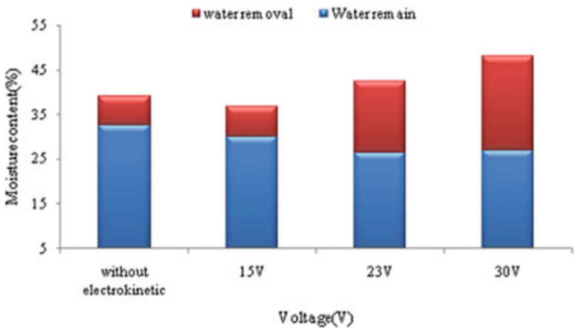


Fig. 10 Comparison of removal of water effect of voltage gradient (soil 3)

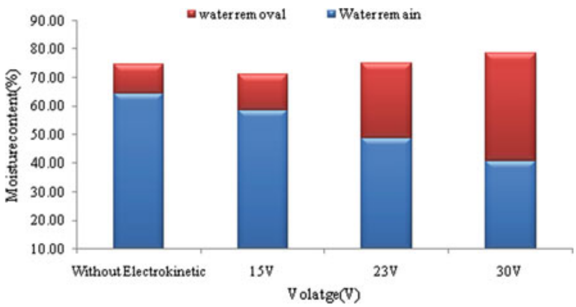
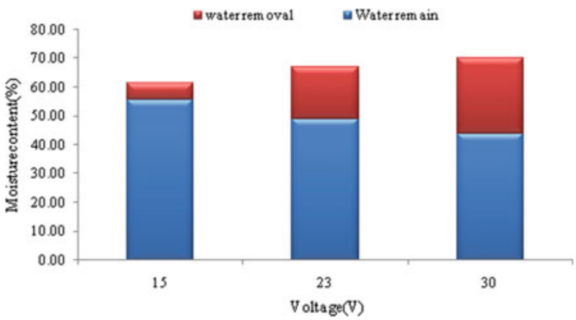


Fig. 11 Comparison of removal of water-effect of voltage gradient (soil 4)



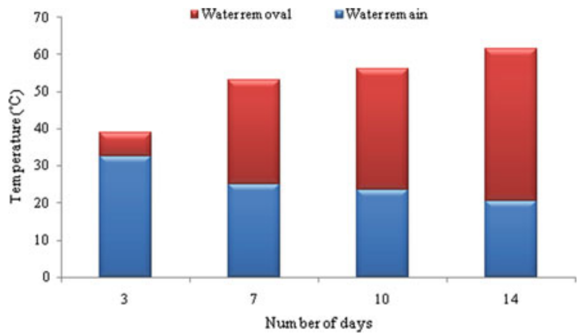


Fig. 12 Comparison of removal of water-effect of experiment duration (without electrokinetic) (soil 1)

and 40.76% at the end of day 1, 7, 10, and 14, respectively. (Fig. 12). The percentage of removal of water was observed with electrokinetic as 21.14%, 56.12%, 57.79%, and 71.54% at the end of day 1, 7, 10, and 14, respectively (Figs. 13, 14 and 15).

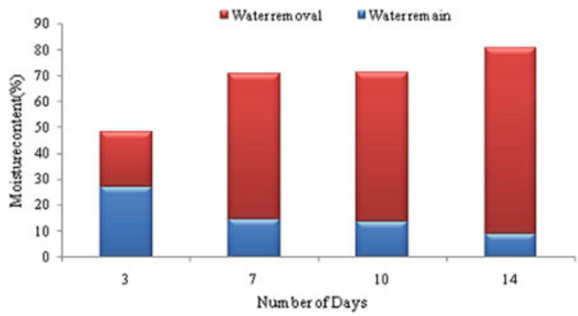


Fig. 13 Comparison of removal of water-effect of experiment duration (with electrokinetic) (soil 1)

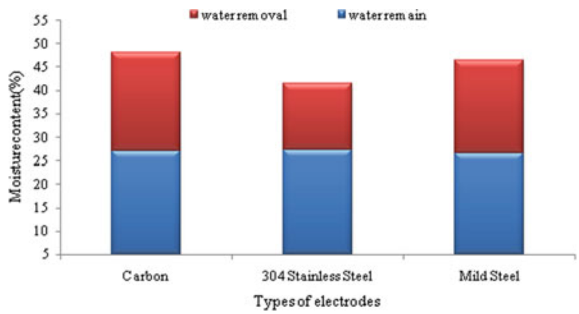


Fig. 14 Comparison of removal of water-effect of electrodes (soil 1)

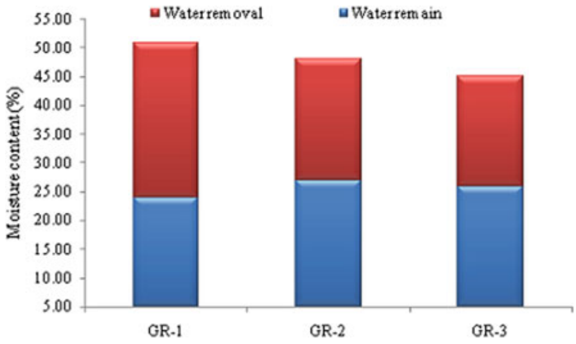


Fig. 15 Comparison of removal of water-effect of re-run analysis (soil 1)

4.3 Effect of Electrode

Three experiments were performed using different types of electrodes for this phase in Soil 1 with same geotextile at constant 30 V. The percentage removal of water was observed 21.14%, 14.14%, 19.71% in carbon electrode, 304 stainless steel and mild steel electrode, respectively.

4.4 Effect of Re-Run Analysis

Three experiments were performed for this phase using Soil 1 with same geotextile. The percentage of removal of water was observed 26.93%, 21.14%, 19.23% in GR-1, GR-2, and GR-3, respectively.

5 Conclusion

Based on the results, we can conclude that.

- the percentage of removal of water can be increased by increasing the voltage gradient rather than duration of test.
- the maximum removal of water was observed in soil 1 with electrokinetic (30v) up to day 14. Thus, the removal of water is increased by increasing processing time with electrokinetic.
- the removal of water in carbon electrode is much better than 304 stainless steel and mild steel are much less there by reduced resistance to water movement under same voltage. As we know that carbon has good conductivity as compared to other mild steel and 304 stainless steel. Here, results clearly show that dewatering rate was faster if carbon used as anode with compare to mild steel and 304 stainless steel. But only one problem arise was every two or three days we have to change

it because of reaction of current on carbon rod get melted so connection was lost. In carbon, corrosion problem was less as compared to 304 stainles steel and mild steel.
- the re-run experiment with (polyester cotton with copper filament) conductive geotextile shows change in the removal of water at first run (GR-1) was better result as compared to second (GR-2) and third run (GR-3). Efficiency of conductive geotextile decreased after first run since dewatering rate was slow at second and third run. Results show that the removal percentage of water increased at first run of conductive geotextile.

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Numerical Analysis of Geogrid CFG Pile-Supported Embankment on Soft Soil

N. B. Umravia and C. H. Solanki

Abstract This paper presents an optimum design and analysis of CFG pile-supported embankment for the national highway in India. Cement flyash and gravel piles are new approach of ground improvement column technology for the India. These techniques were used in china in many projects as highway and railway embankment for ground improvement. A couple two different dimension embankments 2D FEM model are used in this paper over the expansive soil near Surat City, Gujrat. The present study does with this observation by employing CFG pile support embankment composite foundation and screening its reflection through 2D FEM analyses. The present study on CFG pile variable length affects the slenderness ratio, material properties (Grade) as well as effects of area replacement ratio. The slenderness ratios of pile and thickness of sand layer on the proportion of loads are shared by CFG piles and its settlement response of composite foundation on different levels of vertical load by highway embankment. Results showed the thickness of sand mattress on CFG pile was transmission load effectively as significant thickness of it. CFG pile was efficiently in improving the bearing capacity and reducing vertical settlement of expansive soft soil. Other way results showed that incremental lengths of CFG pile reducing the settlements of composite foundation and increment in diameter of CFG pile effectively improved in bearing capacity of composite foundation. CFG pile material friction angle improves the bearing capacity of foundation system.

Keywords CFG (cement, fly ash, gravel) · Pile · 2D FEM · Slenderness ratio · Area replacement ratio

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1 Introduction

Soils and piles bearing capability can be absolutely developed in composite foundation, which had higher social and economic advantages. CFG pile-supported embankment was used as composite foundation with and without reinforcing support in china engineering practice. Recent trends in soil improvement are with low-strength flexible pile composite foundation which is significantly increasing foundation soft soil strength and decreasing foundation settlement. CFG pile is semi-rigid pile in nature which is high strength column technology. Application of CFG pile could be fully utilization soil; enhancing bearing capacity and decreased the settlement in additional reduced the blurred effect of the soils and the piles interaction. Moreover the application of CFG pile in consolidating soft soil foundation is gradually applied in soft soil treatment.

This paper is based on the numerical analysis of highway embankment of the Surat City, Gujarat India. This numerical model was simulated the consolidation behaviors by a couple of hydraulic and mechanical model. The parametric study of CFG pile influence with length, which effect on maximum bending moment of the performance of CFG pile-supported embankment with and without reinforcement.

The Surat region soils are expansive nature therefore during the construction of highway important element is subgrade. The soil load distribution of a highway subgrade engineering was difficult, mainly composed of artificial filler, newly sedimentary cohesive soil, silt soil and quaternary deposition of cohesive soil, sandy soil and cohesive soil. The bearing capacity characteristic value was only 100 kPa. Natural subgrade bearing capacity characteristic value was less than the subgrade bearing capacity characteristic value (design value), so natural subgrade cannot be the bearing stratum. The new approach of cement fly-ash gravel (CFG) pile was adopted in to improve the bearing capacity. Bearing capacity characteristic value was required to 200 kPa after the foundation treatment. In developing the proposed theoretical method, the following simplifications are used for embankment design.

1. The embankment fill is homogeneous, isotropic.
2. There is only one layer of geosynthetic which is considered.
3. The soft soil and the embankment fill deform only vertically, and no relative displacement and slippage between geosynthetic and subsoil.
4. The height of embankment fill is larger than 0.5 times pile spacing.

Literature Survey

The performance of an embankment constructed on a cement deep mixing-formed column–slab system improved soft clay deposit was analyzed using field measurements and finite element simulation results [1]. CFG pile contributed to less than 21% of the total settlement [2]. Cement, fly-ash, gravel (CFG) pile is consisted with macadam, gravel, sand, fly ash mixed with cement and water. In sand, silt, clay and muddy soil and miscellaneous fill foundation, there have been many applications of CFG technology [3]. Geosynthetic-reinforced piled embankments have been increasingly used to stabilize embankments over soft soils. The presence of the reinforcement reduces the stresses transferred to the soft foundation and improves the efficiency of the transference of loads to the piles [4]. The laboratory results in the form of vertical load intensity-settlement behavior were compared with that obtained from FEM software [5].

2 A CFG Pile-Supported Embankment FEM Model

The finite difference software Plaxis 2D was adopted for this numerical analysis. The cross section of the numerical model for this case study is presented in Fig. 1 which shows the cross-sectional of the CFG pile-supported highway embankment, which is 1.8-m high with an 8-m wide crest and 1(V): 2(H) side slopes. The elastic modulus and shear strength of the embankment fill were determined from laboratory tests. The CGF pile under the highway embankment has been installed in a square pattern at a spacing of 1.3 m using a pile installation method. The water–cement ratio of the mix design and the cement content were 0.59 and 324 kg/m^3 of soil, respectively. All this standard data have been used for CFG pile for design optimization C-20.

The calculation model was shown as Fig. 2. Mesh partition adopted 15 nodes triangular element, and the program automatically subdivided mesh. Especially, the

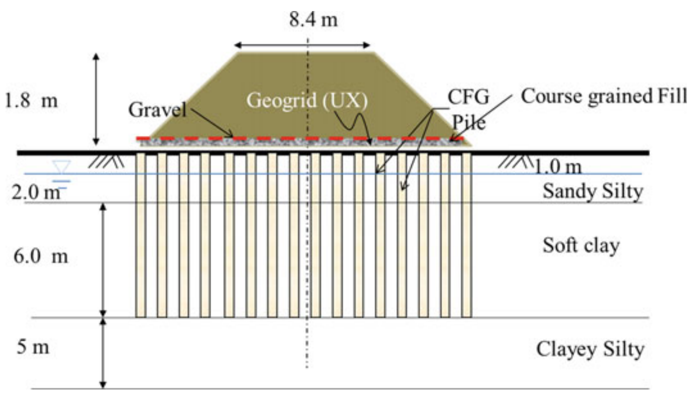


Fig. 1 Cross section of highway embankment

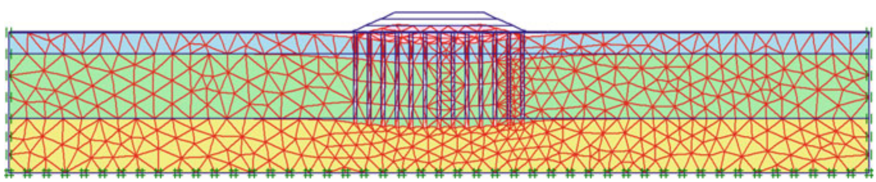


Fig. 2 FEM modeling with large vertical displacement

Table 1 Properties of embankment fill

Material	Unit weight (kN/m^3)	Friction angle (degree)	Cohesion (kPa)	Young's modulus E (MPa)	Passion's ratio ν
Embankment fill	18.60	33.8	11.5	20	0.3
Gravel	20	36	60	70	0.3
CFG pile	24	-	-	20,000	0.2

Gravel and pile used in FE analysis

Table 2 Soft foundation soil properties used in FE analysis

Layers	Thickness (m)	Degree (φ)	Cohesive C' (KPa)	γ (kN/m^3)	γ_{sat} (kN/m3)	K_x, K_y (m/day)	E	ν
Sandy silt	2	30.6	4	17	19	6.91E − 05	2.0E4	0.35
Soft Clay	6	27	13	16	17	8.3E − 08	2.0E4	0.3
Clayey silt	5	34	0	17	20	8.9E − 06	2.0E4	0.27

mesh of reinforcement area and cushion area was resubdivided. At the same time, pile was simplified to sheet pile form on the basis of equivalent principle of axial stiffness, in which the pile had a diameter of 0.3 m. The physical and mechanical parameters of reinforcement area and substratum and cushion and CFG pile were shown in Table 1. The following presumptions were adopted when it was numerical Analyisis in Plaxis (Table 2).

(a) The pile is linear elastic material, meeting with generalized Hooke's law;
(b) Soil transition zone and cushion were elastic–plastic materials, adopting Mohr–Coulomb constitutive model.

3 Effect of the Cushion Thickness Change on Settlement

The height of embankment was 1.8 m. The computing result was shown as Fig. 2. It can be seen from Fig. 2 that the settlement was larger as the cushion is not set up, which was contributed to the significant stress concentration in the pile under the load for no cushion. Therefore, the pile bore most or the entire load as the load was applied, which caused greater settlement than that of the cushion being set up. The settlement reached 5.0 cm in the above example; when the cushion was set up, the settlement rate got slow, which implied that the soil between piles began to bear the part of load. As the thickness of the cushion varied from 0 to 30 cm, the extent to decrease of settlement was the largest; while the thickness of the cushion exceeded

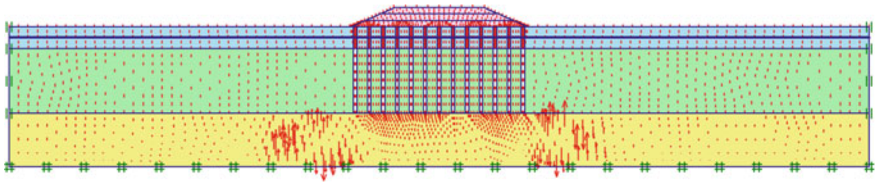


Fig. 3 Output Results of CFG Pile embankment

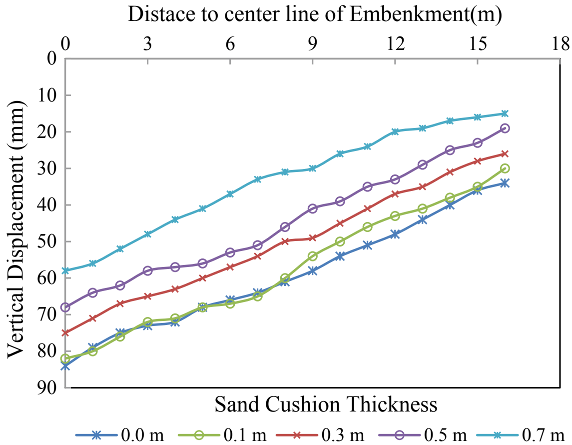
30 cm, the extent to decrease of settlement got very little; whereas the thickness of the cushion was larger than 50 cm, the increase of thickness had almost no effect on the settlement of subgrade. In view of the reliable technique and the sound economy, the reasonable thickness of cushion was from 30 to 60 cm. The cushion of certain thickness was set up to ensure that pile and soil bear load in common and fully plays the role of soil between piles, which resulted in the improvement of bearing capacity for compound foundation.

Effect of cushion sand bed is significant effect of decreasing the settlement effect, and arching effect has been created in embankment as shown in Fig. 3. The effect has been nutrised with the increasing the depth of sand cushion and Geosynthetics layers.

Results

To investigate the influence parameters change in view of the thickness of sand cushion are varied in embankment from 0 to 60 cm with the settelment behaviors of embankment. in this present study influence of sand cushion thickness variable 0,0.1,0.3,0.5, and 0.7 m has been considering for evaluating vertical settlement of embankment under static loading. Figure 4 elaborates the relation of influence of sand cushion thickness to vertical settlement; it clearly shows that with increasing

Fig. 4 Influence of the sand cushion in embankment



the thickness of sand cushion the rate of vertical settlement has been effectively decreasing in respectively.

Figure 5 shows the lateral deformation of clay and the pile group as measured from Plaxis 2D analysis. A maximum lateral pile head deflection localized to the center of every pile at considering row in CFG pile-supported embankment All piles within the pile group deflected laterally under the influence of the lateral loading from the embankment fill.

The results are shown in Fig. 6 which describes the pore water pressure of the embankment at center point of embankment pore water pressure analysis; it is maximum during the 15–80 days effect. Soil has been consolidated than pore pressure has been decreasing.

Figure 7 shows the stress at middle point of embankments with pile, without pile and with pile with single layers of geosynthetics obtained by numerical analysis under its embankment load only It is clearly observed that stress maximum in 70 days after construction than after it is became constant. When used CFG pile for ground improvement it has been significant decreeing the stress concentration, while the use

Fig. 5 Comparative lateral displacement of soft soil and CFG pile under static load

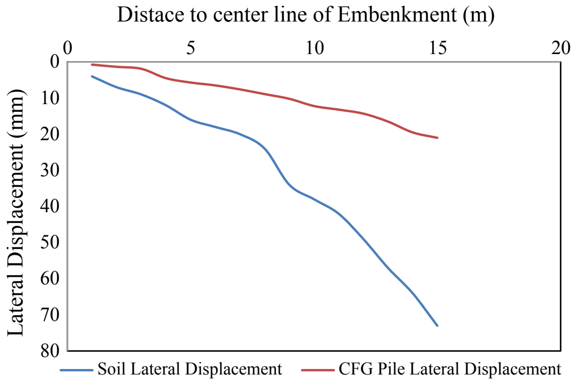
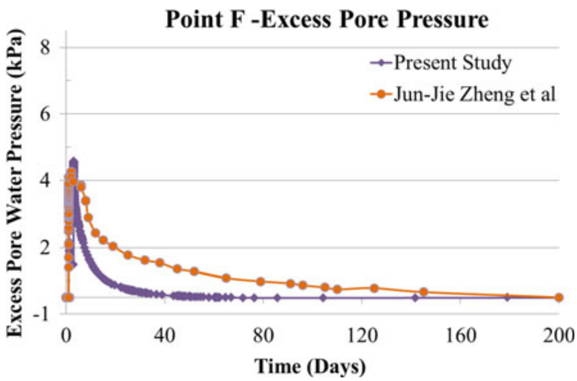


Fig. 6 Pore pressure behaviors with respect to time



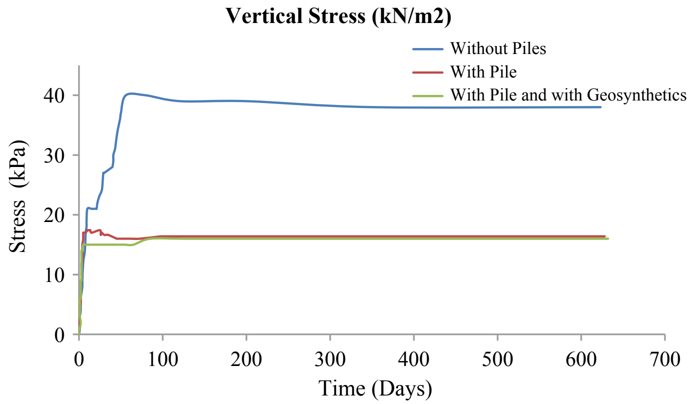
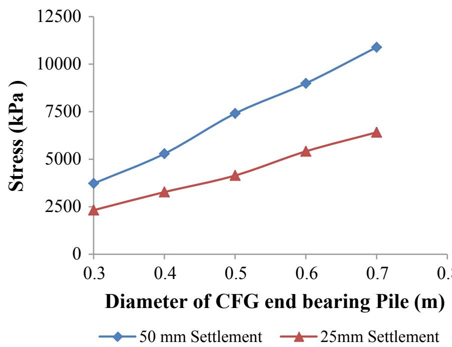


Fig. 7 Stress versus time acting vertical stress center point of embankment

Fig. 8 Influence of single CFG end bearing pile diameters on stress and vertical settlement



one layer of geosynthetics layer is also decreasing in the stress. The approximate stress concentration has been decreasing, 38% (Fig. 7).

To better understand the behavior of CFG pile, using Plaxis 2D axis symmetrical model analysis for the variation of stress on top of the CFG pile is plotted in Fig. 8 with a various diameters and a constant length 10 m of end bearing CFG piles for a vertical settlement of 25 and 50 mm. In order to increase the diameters of CFG pile effectively, the percentage increase in stress is about 70. The corresponding increase in stress is about 70% at 25 mm settlement have the same percentage it can also be observed that the stress for the smaller diameter column is higher because of higher confining pressures (consequently higher stiffness) developed in the CFG pile.

4 Conclusion

1. As the soft soil subgrade was treated with CFG pile, the bearing capacity of composite foundation increased substantially; at the same time, the settlement of embankment decreased significantly.
2. As per the results, the effect of vertical stress obtain is less affected on embankment used with CFG pile.
3. The settlement of subgrade decreased and tended to stabilize as the ratio between length and diameter of pile increased, which also showed there existed in reasonable pile length with the same height of embankment.
4. When the necessity of bearing capacity single pile CFG pile is not high, therefore the diameter of CFG pile is usually to select in composite foundation between 300 and 500 mm.
5. In CFG pile-supported embankment sand cushion are significant effect to load distribute equally the settlement of subgrade was larger while the cushion in embankment has been not constructed. In case of constructed sand, cushion settlement of subgrade has been decreasing as the increasing the thickness of cushion.

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Analysis and Modification of Engineering Behavior of Soil Using Plastic Waste Materials

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Abstract Plastic waste management becomes a serious issue all over the world. Dumping of plastic waste on land without treating it degraded the properties of parent material. The main objective of this study is to demonstrate the use of plastic waste for improving properties of soil and soil subgrade underlying by aggregate base which is applicable in construction of embankment and flexible pavement respectively. Waste plastic water bottles converted into plastic strips with an aspect ratio of 6%. The behavior of randomly distributed plastic strip-reinforced soil system and soil aggregate system was observed by conducting soaked CBR test series with different percentage of plastic strips (0%, 0.5%, 1% and 2%). Results of study show that soil reinforced with waste plastic strips has significant increase in CBR values with respect to unreinforced soil in both the cases. The results also reveal that the 1% of plastic strip-reinforced soil system and soil aggregate system show significant improvement in strength.

Keywords Plastic waste · CBR · Soil · Aggregate

1 Introduction

Improvement in stability or bearing capacity of the soil can be achieved by the use of controlled compaction, addition of suitable admixtures, use of waste materials, etc. It is more economical both in terms of cost and in energy to increase the strength of the soil rather than going deep. As day by day infrastructure is developing, along with the growth of the country, there is drastic increase in population. This increased

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population leads the problem of solid waste management. According to CPCB report 2018, out of total solid waste, plastic waste is 8.3 billion tones. The bottled water is the fastest growing beverage industry in the world. It is reported that annual consumption of plastic bottles in the world are approximately 10 million tons, and it grows about 15% every year [CPCB]. Plastic waste disposal is the sensitive issue all over the world. There is urgent need to reuse or recycle the plastic waste. A common problem with recycling plastics is that they are often made up of more than one kind of polymer (heterogeneous character) or some sort of fibers added to the plastics (a composite) to give added strength. This characteristic are helpful for using the waste materials in soil of poor strength to improve geotechnical properties of soil. This study of using raw plastic bottles is an alternative method for the improvement of strength parameters of soil.

1.1 Background Study

Rapid improvements in the engineering world have influence a lifestyle of human beings in utmost extends, but day-to-day activities of mankind are augmenting risk in the environment in the same proportion. Plastic wastes have become one of the major problems for the world. The harmful gas which is being produced by this agent leads to tremendous health-related problems. So, effective engineering implementation of this has become one of the challenging jobs for engineers. Engineer are seeking for astute implementation of these wastes in ample amount, and implementing these wastes in soil stabilization helps to reduce the risk of natural destruction which is caused due to rainfall or other aspect, and also, it aids in reducing the waste in an ample amount. Plastic is considered as one of the major pollutants of environment as it would not decay or cannot be destroyed. So, implementing this for some good purpose helps to reduce its effect also.

Plastic can be one of the materials which can be used as a soil stabilizing agent, but the proper proportion and aspect ratio of it matters in CBR value of soil. Study shows that that benefits of reinforcement increase to certain level and after that it will decrease the strength so careful observation must be done. All this shows that plastic strips can be used as a reinforcing material in stabilization of the sub grade soil if used in right proportion [1].

Chebet et al. (2014) conducted experiments to determine the increase in shear strength and bearing capacity of two samples of locally available soil due to random mixing of strips of high-density polythene material from plastic shopping bags. Strips of shredded plastic material were used as reinforcement inclusions at concentration of up to 0.3% by weight. These results indicate that the increased strength of soil was due to tensile stresses mobilized in the reinforcements [9].

Achmad Fauzi et al. (2016) used two soil samples R2 and R24 collected from various sites of KUANTAN. Waste cutting HDPE and crushed waste glass were used as additives. The variations of additive contents were 4%, 8%, 12% by dry total weight of soil sample, respectively. They evaluated engineering properties like sieve analysis, Atterberg limit, specific gravity, standard compaction, soaked California bearing ratio, and tri-axial test of the soil sample before stabilization and after stabilization. The result showed that on addition of waste HDPE and glass, there was an increase in PI, about 10% for R24 and 2% for R2 samples, respectively. The value of optimum water content decreases and MDD increases when content of waste HDPE and glass was increased, but there was an increase in CBR value. Authors also observed that there was a decrease in the value of cohesion and increase in friction angle of R2 and R24 samples with additives [10].

Dhatrak et al. (2015) calculated the engineering properties by mixing waste plastic. It was observed that for construction of flexible pavement to improve the sub-grade soil of pavement using waste plastic bottles chips is an alternative method. In a proportion of 0.5, 1, 1.5, 2, and 2.5% of the weight of dry soil, plastic waste was added to calculate CBR value. He concluded that using plastic waste strips will improve the soil strength and can be used as sub-grade. It is economical and eco-friendly method to dispose waste plastic [11].

Anas Ashraf (2011) studied on the possible use of plastic bottles for soil stabilization. The analysis was done by conducting plate load tests on soil reinforced with layers of plastic bottles filled with sand. The bottles cut to halves placed at middle and one-third position of tank. The test results showed that cut bottles placed at middle position were the most efficient in increasing strength of soil [12].

Rajkumar Nagle 2014 conducted various experiments to compare CBR of soil-reinforced with natural waste plastic. They mixed polyethylene plastic bottles food packaging and shopping bags, etc. As reinforced with three soil samples of expansive soil (black cotton soil), silt clay and sandy soil. Their study showed that MDD and CBR value increases with increase in plastic waste [13].

2 Experimental Program

2.1 Materials

Soil: Soil sample used has been collected from big bazaar near vesu Surat. Soil is classified as CH the soil.

Waste Plastic: For making the plastic chips collecting the plastic waste bottles and for the experiment work, cut it by proper size and 6 aspect ratios. Approximate percentage of plastic chips is between 0%, 0.5%, 1%, and 2%. At the time of the CBR test, plastic chips are mixed in very proper manner; thus, soil is not changing.

Aggregate: Aggregates are used as it passes through the 10 mm sieve and retains on 6.5 mm sieve.

2.2 Compaction Test

Compaction test of soil is carried out using Proctor's test to understand compaction characteristics of different soils with change in moisture content. Compaction of soil is the optimal moisture content value 23.797 at which a given soil type becomes most dense and achieves its maximum dry density value 14.130 by removal of air voids.

2.3 California Bearing Ratio (CBR) Test

It is the ratio of force per unit area required to penetrate a soil mass with standard circular piston at the rate of 1.25 mm/min to that required for the corresponding penetration of a standard material.

2.4 Aggregate Impact Test

The aggregate impact test is carried out to evaluate the resistance to impact of aggregates. Aggregates passing 12.5 mm sieve and retained on 10 mm sieve is filled in a cylindrical steel cup of internal diameter 10.2 mm and depth 5 cm which is attached to a metal base of impact testing machine.

$$\text{Aggregate impact test } (\%) = 15.18 < 30 \text{ (satisfy)}$$

2.5 Abrasion Value Test

The principle of Los Angeles abrasion test is to find the percentage wear due to relative rubbing action between the aggregate and steel balls used as abrasive charge.

Los Angeles machine consists of circular drum of internal diameter 700 mm and length 520 mm mounted on horizontal axis enabling it to be rotated; an abrasive charge consisting of cast iron spherical balls of 48 mm diameters and weight 340–445 g is placed in the cylinder along with the aggregates.

$$\text{Abrasion value test } (\%) = 9.72 < 30 \text{ (satisfy)}$$

2.6 Flakiness Index

Flakiness Index is the percentage by weight of particles in it, whose least dimension (i.e., thickness) is less than three-fifths of its mean dimension.

$$\text{Flakiness index }(\%) = 10.81 < 35 \text{ (satisfy)}$$

2.7 Elongation Index

Elongation index is the percentage by weight of particles in it, whose largest dimension (i.e., length) is greater than one- and four-fifths times its mean dimension.

$$\text{Elongation index }(\%) = 23.59 < 30 \text{ (satisfy)}$$

3 Results and Discussion

3.1 Compaction Test

Figure 1 shows the plot between percentages of fibers verses optimum moisture content. Figure 2 shows the plot between percentages of fibers verses maximum dry density. It shows as the percentages of plastic chips increase, optimum moisture content and maximum dry density decrease.

Fig. 1 Materials (soil and plastic)

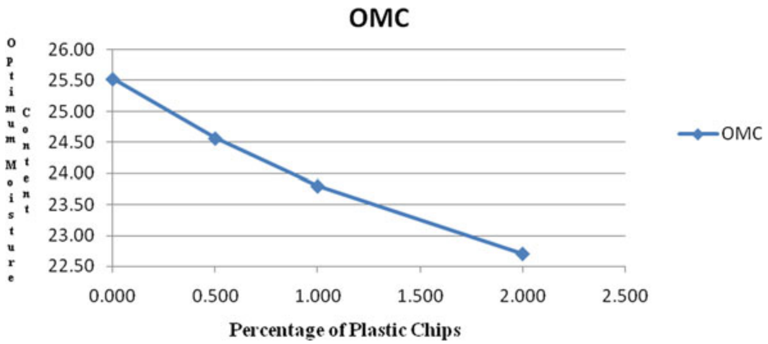

Fig. 2 Percentage of plastic chips verses optimum moisture content

3.2 California Bearing Ratio (CBR) Test

Figure 3 shows plot between bearing pressure verses settlement. In 1% of fibers, the CBR value increases and after that suddenly CBR value decreases. It shows that plastic strips are effective in some proportion. Figure 5 is the plot between pressure and settlement of soil aggregate system, which also shows the same response (Fig. 4).

Figure 6 shows that the comparison of CBR value between soil and soil aggregates system. In both the cases, the percentage of plastic waste is effective at 1 after that there is decrease in CBR value. The difference in increase of CBR value is almost in same rate for both the systemsQuery.

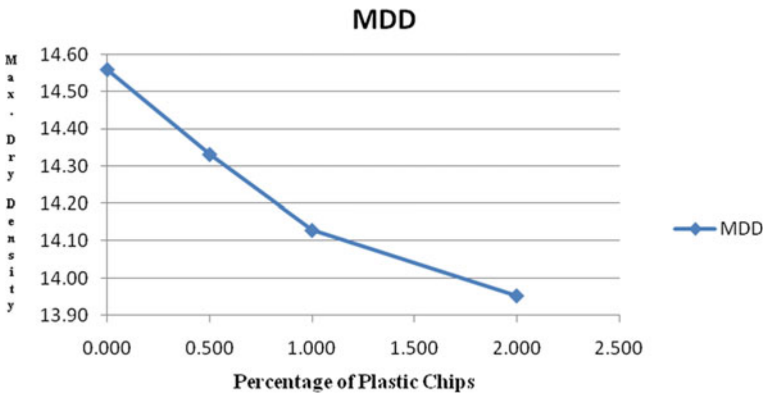

Fig. 3 Percentage of plastic chips verse maximum dry density

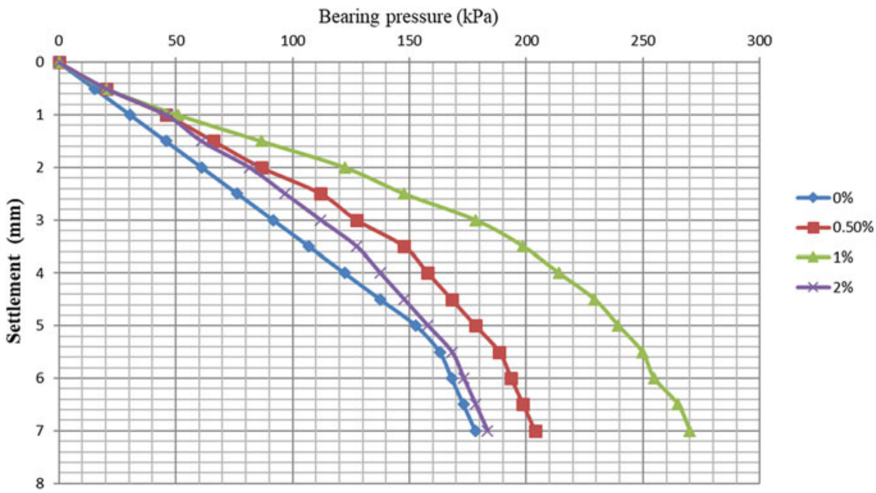

Fig. 4 Pressure verses settlement plot of soil system

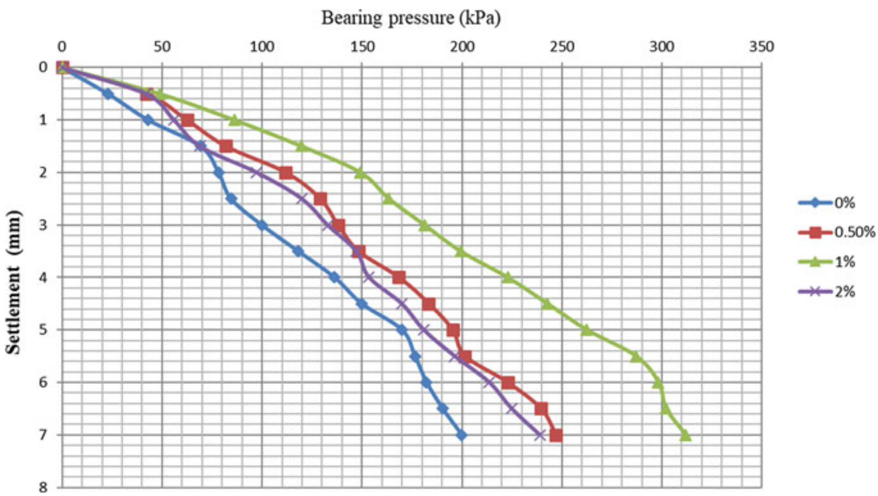

Fig. 5 Pressure verses settlement plot of soil aggregate system

4 Conclusions

From the California Bearing Ratio result, it has been concluded:

- By first observation it conclude that the value of CBR increases while plastic strips percentage increases.
- 1% of plastic fiber gives the optimum result in soil and soil aggregate system.
- After 1%, there is sudden decrease in CBR value again in both systems.

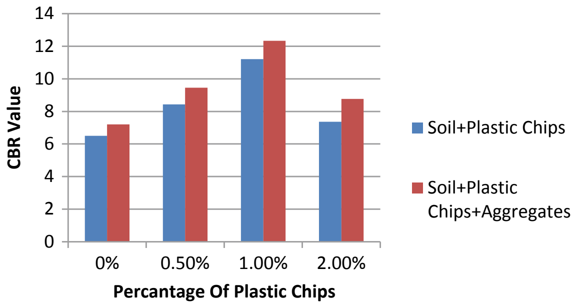

Fig. 6 Comparison of soil and soil aggregates system

- Soil and soil aggregate system shows the same response under loading because it provides good strength.
- So this type of technique can be used for embankment and pavement which may lead to solve problem of plastic waste management.

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Stabilization of Soil Using Terrazyme for Road Construction

Varun Sushil Chaurasia, Pawan Prabhunath Pandey, Atulkumar Vinod Mishra, Sandeep Shyamnarayan Gupta, and Anuj U. Pawar

Abstract In developing countries like India, the most important requirement of any project after performance criteria is its economical, feasibility and serviceability criteria. The traditional methods are not economically feasible also time consuming. Hence, it has created a need to discover the other possible ways to satisfy the performance as well as economical criteria. The present paper describes a study carried out to check the improvements in the properties of black cotton soil and red soil with a bio-enzyme, named Terrazyme. Recently, some bio-enzyme stabilized roads were constructed in various parts of India, which are being performing very well. Bio-enzyme improves the engineering qualities of soil, facilitates higher soil compaction densities and increases stability. Bio-enzyme helps in easy mixing with water at optimum moisture content (OMC), and then it is sprayed over soil and compacted. Soil with varying index properties has been tested for virgin as well as stabilized soil with different dosages. The test results indicate that stabilization improves the soil

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strength up to great extent, which implies that the bearing capacity and the resistance to deformation increase in stabilized soil. The locally available material can be used, and in case of scarcity of granular material, only bio-enzyme stabilized thin bituminous surfacing can fulfill the pavement design requirement. Adopting the IRC method based on soil CBR, the pavement design thickness on stabilized soil also reduces 25–40%. The use of bio-enzyme in soil stabilization is not very popular due to lack of awareness between engineers and non-availability of standardized data.

Keywords Bio-enzyme—Terrazyme · Soil stabilization · OMC · IRC

1 Introduction

As we all know that population of India is increasing day by day which has created a need for better and economical vehicular operation which requires good highways/roads having proper geometric design, pavement condition and maintenance. In many parts of India, soil consists of high silt contents, low strengths and minimal bearing capacity. When poor quality soil is available at the construction site, the best option is to modify the properties of the soil so that it meets the pavement design requirements. This has led to the development of soil stabilization techniques which improve the strength and durability of soil. The main aim of stabilization is cost reduction and to efficiently use the locally available material. Most common application of stabilization of soil is seen in construction of roads and airfields pavement.

2 Objectives

- To study change in the properties by stabilizing with enzyme.
- To optimize the use of local materials in the design and construction of roads by improving their engineering properties.
- To optimize the quantity of Terrazyme to be used as a stabilizing agent.
- To increase the durability, strength and stiffness of soil, improve workability and constructability of the soil and reduce the plasticity index.

3 Significance of the Study

- Output of this research will enhance the development of India economies, particularly rural economies.
- Useful to policy makers in decision making and to economists in budgeting purposes.

4 Literature Review

Lacuoture and Gonzalez (1995) conducted a comprehensive study of the Terrazyme soil stabilizer product and its effectiveness on subbase and subgrade soils. The reactions of the soils treated with the enzyme were observed and recorded and compared to the untreated control samples. The variation in properties was observed over a short period only, and it was found that in cohesive soils there was no major variation in properties during the early days but the soil showed improved performance progressively [1].

Vijay Rajorial, Suneet Kaur (2014) carried out a theoretical evaluation of enzyme. Reduction of about 18 to 26% is seen in cost of construction of roads by using Terrazyme as a soil stabilizer, constructed by public work department in Maharashtra. Structures made of bio-enzyme are economical and have greater strength [2].

Priyanka Shaka et al. (2016): Based on IS classification, red soil is classified as clayey sand and the black cotton soil as highly compressible clay. Laboratory testing showed that decrease in liquid limit and plasticity index was observed with the increase in dosages of Terrazyme. Also, the Terrazyme dosage of 200 ml/0.75m3 of dry soil garnered the best result. Further increase in the dosage does not alter the plasticity characteristics of soils substantially. CBR value of the soil sample was increased by 2.75%. 3.345%. 3.47% and 3.56% by application of the bio-enzyme with a dosage of 200 ml/0.75m3. With further increase in the dosage of the enzyme, no substantial increase was recorded [3].

5 Materials

5.1 Terrazyme

Terrazyme is a liquid enzyme which is organic in nature and is formulated from the vegetable and fruit extract. It is brown in color with smell of molasses and can be easily used without the need of masks or gloves. It is easily mixed with water, and for optimal results should be diluted with optimum moisture content of that soil. This decreases the swelling capacity of the soil particles and reduces permeability (Table 1).

The required quantity of Terrazyme was supplied by Avijeet Agencies, Chennai.

[1-2-4-5-6-7].

5.1.1 Mechanism of Terrazyme

In clay water mixture, positively charged ions (cations) are present around the clay particles, creating a film of water around the clay particle that remains attached

Table 1 Properties of Terrazyme supplied by manufacturer (source—Avijeet Agencies)

Identify (as it appears on label)	N-Zyme
Hazardous components	None
Boiling point	100 °C
Specific gravity	1.05
Melting point	Liquid
pH value	4.4
Evaporating rate	Same as water
Solubility in water	Complete
Appearance/odor	Brown liquid, non-obnoxious

or adsorbed on the clay surface. The adsorbed water or double layer gives clay particles their plasticity. Terrazyme replaces adsorbed water with organic cations, thus neutralizing the negative charge on a clay particle. The organic cations also reduce the thickness of the electrical double layer. This allows Terrazyme treated soils to be compacted more tightly together. Terrazyme resists being replaced by water, thus reducing the tendency of some clay to swell. Terrazyme promotes the development of cementitious compounds using the following general reaction:

$$H_2O + \text{Clay} \xrightarrow{\text{Terrazyme}} \text{Calcium Silicate Hydrates} \quad \text{(i)}$$

[1-4-5-6-7]

5.1.2 Calculation of Terrazyme Dosage

Procedure to prepare the laboratory dilution dosage as given by the manufacturer:

- Determine quantity of soil to be treated with 1 L of Terrazyme in cubic meters, based on plasticity and gradation.
- Read 0.01 ml of TZ concentrate per Kg of soil mix.
- Laboratory preparation = 0.01 ml of TZ concentrate + 100 ml water (1:100 dilution).
- Withdraw from the laboratory preparation that ml as required and add to the water required to bring sample to within 2% below optimum moisture content (laboratory application mixture).
- Mix water required + mls of laboratory application mixture uniformly with soil sample.

The dosages used in the experiments are 0.01 ml, 0.02 ml and 0.03 ml on trial and error basis.

Table 2 Properties of black cotton soil (BCS)

S. No.	Property	Value	IS code
1	Specific gravity	2.5	IS 2720 (part 3)
Atterberg's limit			
2	Liquid limit (%) Plastic limit (%) Plasticity Index	53.33 29.20 24.13	IS 2720 (part 5)
Grain size distribution			
3	(a) Gravel (%) (b) Coarse sand (%) (c) Fine sand (%) (d) Silt and clay (%)	52.50 28.50 13.75 03.25	IS 2720 (part 4)
4	IS soil classification	GC	
5	Free swell index %	02.00	IS 2720 (part XL)
Engineering properties			
6	(a) M.D.D. (KN/m^3) (b) O.M.C. (%)	18.25 22.50	IS 2720 (part 7) IS 2720 (part 2)
7	Coefficient of permeability	8.0×10^{-8} cm/s	IS 2720 (part 17)

5.2 Soils

5.2.1 Black Cotton Soil (BCS)

The black soil is very retentive of moisture. The black cotton soil is found to contain montmorillonite clay mineral that has high expansive characteristics, and these are mainly found in Maharashtra, Madhya Pradesh, parts of Karnataka, Andhra Pradesh, Gujarat and Tamil Nadu [6–8] (Table 2).

The black cotton soil in this experimental work was brought from the Dhule district of Maharashtra.

5.2.2 Red Soil (RS)

Red soils are usually poor growing soils, low in nutrients and humus and difficult to cultivate because of its low water holding capacity. These soils can be found around in large tracts of Western Tamil Nadu, Karnataka, Southern Maharashtra and many parts of India [9] (Table 3).

The red soil used in this experimental work is from Mumbai district of Maharashtra.

Table 3 Properties of red soil (RS)

S. No.	Property	Value	IS Code
1	Specific gravity	2.36	IS 2720 (part 3)
Atterberg's limit			
2	Liquid limit (%) Plastic limit (%) Plasticity index	60.90 31.40 29.50	IS 2720 (part 5)
Grain size distribution			
3	(a) Gravel (%) (b) Coarse sand (%) (c) Fine sand (%) (d) Silt and clay (%)	0.00 12.12 25.85 61.02	IS 2720 (part 4)
4	IS soil classification	CH	
5	Free swell index %	60.80	IS 2720 (part XL)
Engineering properties			
6	(a) M.D.D. (KN/m3) (b) O.M.C. (%)	15.20 28.00	IS 2720 (part 7) IS 2720 (part 2)
7	Coefficient of permeability	$1.6 * 10^{-9}$ cm/s	IS 2720 (part 17)

6 Problem Statement

The various problems faced due to poor quality of soil are subgrade failure, freeze and thaw action and many more. In this project, we will examine roads against traffic and try to minimize the problems faced by road users which are generally attributed to **poor subgrade conditions** (Fig. 1). We will reduce or eliminate the thickness of the different layers of road by treating the subgrade layer with the Terrazyme (Fig. 2) as per the design requirement. The purpose is to explain about the material used, i.e., Terrazyme, mechanism and its advantages by solving these consequences.

Our main focus is to develop the infrastructure of the country by providing better quality of subgrade layer.

7 Concept

- We are using Terrazyme in subgrade layer of road. The Terrazyme is mixed with water and is spread over subgrade soil which makes the subgrade layer of road almost impermeable. As the surface becomes impermeable, it will not absorb water which percolates from upper layer of road. It increases the durability of the road and prevents the formation of rut, pot holes, etc.

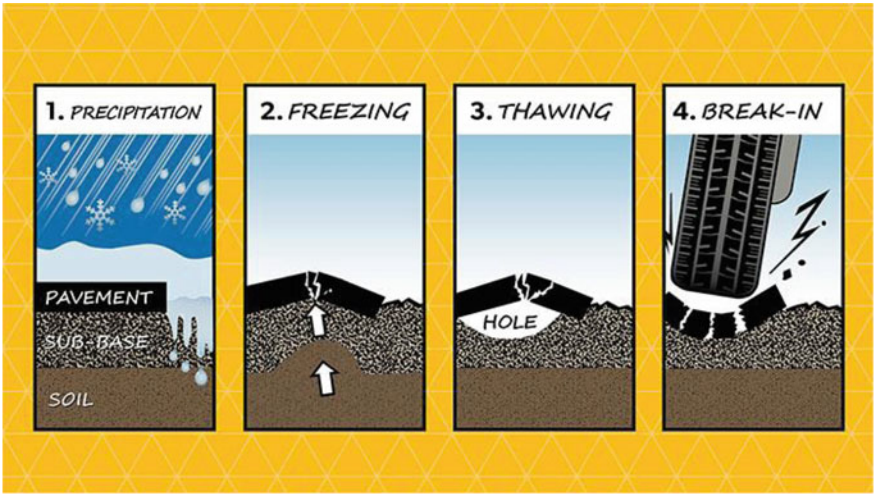

Fig. 1 Potholes. *Source* Pothole Wikipedia

Fig. 2 Proposed model

8 Methodology

It is necessary for both consultants and contractors, as executors involved in a production process that is making use of Terrazyme, to understand that significant cost savings that can be achieved through the relatively fast speed of construction with Terrazyme.

8.1 Pre-construction Phase

- For companies or organizations without prior experience with Terrazyme, it is advisable to contact a certified consultant or the manufacturer for advice on the dilution ratio and the crust thickness of the subbase and base layers of the road structure. The certified consultants or the manufacture can provide design assistance to determine the need for pavement on the stabilized soil layers and the type of pavement that can be selected.
- It is imperative to properly study the characteristics of the selected soil to insure its suitability for treatment. Depending on the characteristics of the soil information on the plasticity and load bearing capacity, it is necessary and prudent in some cases to submit the soil prior to the initiation of the project to laboratory or field trials to ensure its suitability.

8.2 Construction Phase

Step 1: After the embankment or the box cutting has been made according to the conventional construction methods, the construction team puts a layer of scarified soil on top of the subgrade soil and removes all the large stones, roots and trash from the loosened soil. Road grader or farm tractor with teeth is used.

Step 2: Pulverize the scarified soil, so that the mass is separated from particles rather than breaking down of individual particles with the help of either road grader or farm tractor with rototiller, or any other mixing equipment.

Step 3: The engineer then starts with the Terrazyme application by spraying water containing diluted Terrazyme on the road surface. After a sufficient quantity of water has been sprayed to bring the soil to OMC, the spraying of the water is stopped, which is carried out with water truck with distributor bar nozzle mounted at front or back. The soil has to be mixed thoroughly to make sure that the enzymes diluted in water are mixed through the soil and initiate the process of cation exchange with road grader or farm tractor with any mixing equipment.

Step 4: Once the soil containing the Terrazyme is properly mixed, the engineer can start with the formation of the camber, to meet design requirements with the help of road grader with blade adjustment for pitch, angle and side to side elevation. As soon as the camber formation is completed, the compaction of the soil can take place for finishing the surface. This can be done with smooth drum rollers, vibrating compactors or sheep-foot rollers depending upon the composition of the soil.

Step 5: After compaction, a spray of water containing a light concentration of Terrazyme can be used under extremely dry and hot conditions to enhance the curing.

The constructed road can be opened for traffic within two to three days after construction under dry conditions. It will also be ready for the application of pavement layers. Items being used for directing the traffic are barriers, signs, cones, tapes, etc.

8.3 Quality Control

- The quality control of structures constructed making use of the bio-enzymatic soil stabilizer, Terrazyme, has to be done by engineers who have a proper understanding of soil mechanics and road construction. Attention can be given to the fact that the proper soil that was selected is also actually used during the construction of the road structures.
- After the construction of Terrazyme layers, density test such as the proctor density test can provide information on the quality of compaction while load bearing test such as CBR or DCP test can provide information on the strength increase of the soil during its curing period.

9 Tests Performed and Results

As per BIS, Indian Standard Methods of Test for soils-IS: 2720 [10].

9.1 Liquid Limit

See Table 4.

Table 4 Liquid limit determination

S. No.	Particulars	Black cotton soil			Red soil		
1	Container no	1	2	3	1	2	3
2	No. of blows	68	72	70	56	34	27
3	Mass of empty container (m1) in gm	35	35	35	20	15	20
4	Mass of container + wet soil (m2) in gm	60	68	65	35	30	40
5	Mass of container + dry soil (m3) in gm	50	57	53	30	25	32.5
6	Mass of water (m2–m3) in gm	10	11	12	5	5	7.5
7	Mass of dry soil (m3–m1) in gm	15	22	18	10	10	12.5
8	Water content (%)	66.66	50	66.66	50	50	60
9		Average = 61.1%			Average = 53.33%		

Table 5 Plastic limit determination

S. No.	Particulars	Black cotton soil			Red soil		
1	Dosage and container no	1	2	3	1	2	3
2	Mass of empty container (m1) in gm	20	20	20	19.62	20.45	19.47
3	Mass of container + wet soil (m2) in gm	38.54	59.50	71.30	20.63	24.67	22.4
4	Mass of container + dry soil (m3) in gm	33.52	48.50	62.43	20.4	23.6	21.5
5	Mass of water (m2-m3) in gm	4.72	11	9.07	0.23	1.07	0.9
6	Mass of dry soil (m3-m1) in gm	13.34	28.5	42.43	0.78	3.15	2.03
7	Water content (%)	34.9	38.59	20.90	29.4	34	30
8		Average = 31.46%			Average = 31.2%		
9		Ip = 29.64%			Ip = 22.13%		

9.2 *Plastic Limit*

See Table 5.

9.3 *Unconfined Compression Test*

See Table 6.

Table 6 UCS determination

Curing Period In Days			Unconfined Compressive Strength (KPa)					
	RS Alone	RS + TZ(3)	RS + TZ(3)	RS + TZ (3)	BCS Alon e	BCS + TZ (1)	BCS + TZ (2)	BCS + TZ (3)
0	160	163	184	174	147	177	195	182
7	175	180	234	218	–	192	258	239
15	–	240	309	276	–	250	316	291
30	–	283	352	305	–	298	369	324

Table 7 CBR determination for RS

CBR (%)								
Penetration (mm)	RS alone		RS + TZ (1)		RS + TZ (2)		RS + TZ (3)	
	Actual load (KN)	CB R	Actual load (KN)	CB R	Actual load (KN)	CBR	Actual load (KN)	CB R
2.5	35.21	2.62	35.62	2.65	40.05	2.98	38.03	2.83
5	70.8	3.5	70.96	3.52	74.39	3.69	71.57	3.55
7.5	103.2	4	110.94	4.3	116.1	4.5	113.52	4.4

Table 8 CBR determination for BCS

CBR (%)								
Penetration (mm)	BCS Alone		BCS + TZ (1)		BCS + TZ (2)		BCS + TZ (3)	
	Actual Load (KN)	CBR	Actual load (KN)	CBR	Actual load (KN)	CBR	Actual Load (KN)	CB R
2.5	33.6	2.50	62.90	4.68	93.54	6.96	70.69	5.26
5	65.52	3.25	108.06	5.36	177.81	8.82	145.1	7.20
7.5	–	–	186.54	7.23	251.55	9.75	220.8	8.56

9.4 California Bearing Ratio

See Tables 7 and 8.

10 Advantages

- It increases the durability and shear strength and makes the subgrade layer of the soil almost impermeable.
- It reduces quantity of materials required in construction, reduction in cost of overall construction of road.
- It does not have an adverse effect on soil or on environment, hence it is an eco-friendly technique.
- This technique can save a lot of money of government required for the maintenance of road.

11 Disadvantages

- It requires skilled labor and expertise supervision.
- Proper dilution ratio should be taken to get the optimum strength.
- Excess amount may lead to the formation of cracks.

Table 9 Estimated cost

S. No.	Layer of road	Estimated cost			
		Without treatment		With treatment	
		Depth (m)	Cost (Rs.)	Depth (m)	Cost (Rs.)
1	Subgrade	0.1	2,560,000	0.1	2,560,000
2	Terrazyme				1,440,000
3	Subbase	0.2	5,120,000	0.15	3,840,000
4	Base course	0.15	3,840,000	0.1	2,560,000
5	Surface course	0.1	3,392,000	0.07	2,374,000
Total cost		14,912,000		1,2774,000	
Cost saving		2,137,600			

- It is unsuitable for small construction work.

12 Cost Comparison

Estimated cost for 1000 m length, 2 way 2 lane—8 m wide road (Table 9).

13 Conclusion

In reality and practice, addition of bio-enzyme gives better performance in the field and ultimately ensures durable and maintenance-free pavement. As we proceed in our research, we came to the conclusion that there is not any improvement in properties of red soil due to addition of Terrazyme since red soil is a non-cohesive soil. Hence, it concludes that Terrazyme improves the property of cohesive soil only.

Terrazyme eliminates the use of granular subbase, base course and surface course in case of low traffic. The material to be used is eco-friendly and saves a lot of resources. Thus, the product so formed after the application of Terrazyme is biodegradable in nature, and the affect is permanent.

Terrazyme is proved not only smarter material but also eco-friendly in coming years and is most feasible in construction work as we have discussed. It could play a pivotal role in this upcoming revolution if their remarkable properties are been exploited.

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Study of Swelling and Shrinkage Characteristics of Expansive Soil Using Silica Gel as an Admixture

Uday Chaudhari and Vanshika Muchhara

Abstract Expansive soils are characterized as soil which undergo large volume changes on availability of moisture. The continuous cyclic wetting and drying process cause vertical and horizontal movement in expansive soil which leads to failure of engineering structure erected on such soils (Masoumeh and Masoud in Electronic Journal of Geotechnical Engineering 17:2673–2682, [1]). The expansive soil has very low bearing capacity, high plasticity, high compressibility, low permeability, high swelling and shrinkage properties, due to the presence of montmorillonite mineral. The expansive soil contains the clay particle of medium to high compressibility and covers nearly 20% of geographical area in India which is the concern of study (Zhang and Cao in Journal of Wuhan University of Technology 17:73–77, [2]). In this research, silica gel as admixture is used to aid the properties of parent material. This study aims to conduct soil stabilization of black cotton soil of North Gujarat region by using silica gel in different proportion. In this research paper, silica gel is added 2, 4, 6, 8 and 10% by weight to give comparison between the soil properties of untreated soil and treated soil. Due to the addition of 10% of silica gel by mass in virgin soil, l plastic limit increased by 16.72% and liquid limit decreased by 19.23% which ultimately decreases plasticity index by 57.5. Also it was found that MDD is decreased by 4.3%, and free swell index decreased by 41.56% on addition of 10% silica gel by mass.

Keywords Expansive soil · Silica gel · Admixture · Stabilization

1 Introduction

Generally, civil engineering structures are built on or underneath of the soil. The soil on which structures are to be built load coming out due structure for design useful period of time without any kind of failure. However, structure erected on black cotton

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or say expansive soil is subjected to cracking in major components of structure which lead to failure of structure. Expansive soils are the clay that have the characteristic property to swell and soften when there is availability of moisture, or shrink and dry cracked when their moisture content is decreased. Soils containing the clay mineral montmorillonite generally exhibit these properties. Besides this if soil contains clay belonging to mica group, which includes illites and vermiculites, then soil can be of expansive nature but does not cause significant problems [1]. Problems are associated with expansive soils, which are located in many regions of Gujarat particularly talking about Bharuch, Vadodara and North Gujarat region. Besides this, expansive soils are mostly found in the arid and semiarid regions, and it covers very large area of the world. It covers nearly 20% of the landmass in India and includes almost the entire Deccan Plateau, Western Madhya Pradesh, parts of Gujarat, Andhra Pradesh, Uttar Pradesh, Karnataka and Maharashtra [3]. In these regions, damage due to swelling action has been observed clearly in the form of cracking and breakup of pavements and damage to building foundations, embankments and irrigation systems. Expansive soils can cause serious damage and distortion to structures, particularly for the light buildings and pavements built on them.

One of the most used method to control volume change is to stabilize expansive soils with the help of admixtures that reduce volume change. The engineering properties of expansive soil are dependent on many factors which include properties of soil, environmental factors, stress condition, etc., which vary from place to place in particular region. Engineering properties of soil are determined by the Atterberg's limits, index properties and other soil properties obtained by laboratory or field tests (Stated by Mitchell 1993) [4]. It is well known that the swell–shrinkage characteristics of expansive soils are closely related to soil plasticity index, free swell index and clay composition of soil.

This report presents laboratory tests to evaluate the effect of silica gel as admixture on expansive soil on geotechnical behaviour of the expansive soil in terms of grain size distribution, Atterberg's limits, specific gravity, compaction characteristics, free swell, shrinkage percent and unconfined compressive strength.

1.1 Objectives of the Study

- To study the effect of silica gel on the properties of the expansive soil.
- To compare the changes in properties of untreated soil and soil with an admixture by conducting laboratory tests.
- To find out economic and optimum percentage of admixture to be added according to weight of total soil mass.
- For the detailed investigation of engineering properties of soil and to classify the type of soil.

2 Materials and Methods

2.1 Materials

Black Cotton Soil: Expansive soil sample used in this research work was collected from Limda Village, Vadodara city, near Limda Lake from pit having depth about 1.5 m.

Silica gel: Silica gel is an amorphous and porous form of silicon dioxide (silica), consisting of an irregular tridimensional framework of alternating silicon and oxygen atoms with nanometer-scale voids and pores. Silica gel's high specific surface area allows it to adsorb water easily, making it useful as a drying agent. Silica gel is often described as "absorbing" moisture material. Silica gel may be freshly prepared from alkali silicate solutions and may vary in consistency from a soft transparent gel similar to gelatin or agar, or a hard solid.

Sample preparation: According to testing and treatment, the soil samples were prepared. Oven drying or air drying soil samples were required. Soil boulder or wet soil lumps converted into fine particles of soil with help of rubber covered mallet or hammer. The virgin soil was taken out for testing purpose and remaining mass of soil mixed with silica gel manually to get uniform mix ratio for each varying silica gel proportions.

2.2 Methods

Initial water content: The test was conducted as per IS-2720 (1980) Part(II) [5].

Specific Gravity: Specific gravity which is the relative measure of the heaviness of the soil particles and here it is determined by density bottle. This test was conducted as per IS 2720 (1980) Part (III) [6].

Atterberg's Limits: The test includes determination of the liquid limits, plastic limits, plasticity index, shrinkage limits, etc., for the natural soil and soil with an admixture. The liquid limit, plastic limit and plasticity index tests were conducted as per IS-2720 (1985) Part(V) [7]. And shrinkage limit and shrinkage index were conducted as per IS-2720 (1972) Part (VI) [8].

Free Swell Index: The free swell index is also one of the most commonly used simple tests to estimate the swelling potential of the expansive soil. The test includes determination of the free swell index of the natural soil and the soil and admixture. The tests are conducted in accordance with IS: 2720 (1977) Part XL testing procedure [9].

Standard Proctor Compaction Test: The standard proctor test is carried out for determination of the maximum dry density (MDD) and optimum moisture content for the natural soil and soil with admixture. The tests are carried out as per IS 2720(1980) PART VII [10].

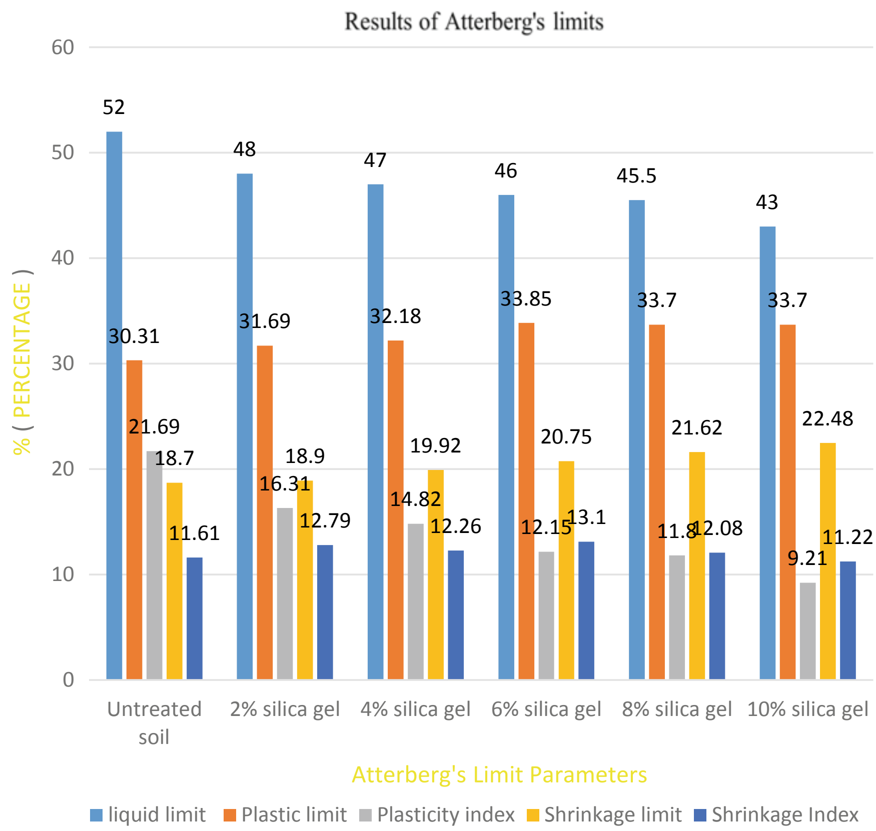

Fig. 1 Observation of Atterberg's limit test

3 Observations

See Figs. 1 and 2.

4 Results

See Table 1.

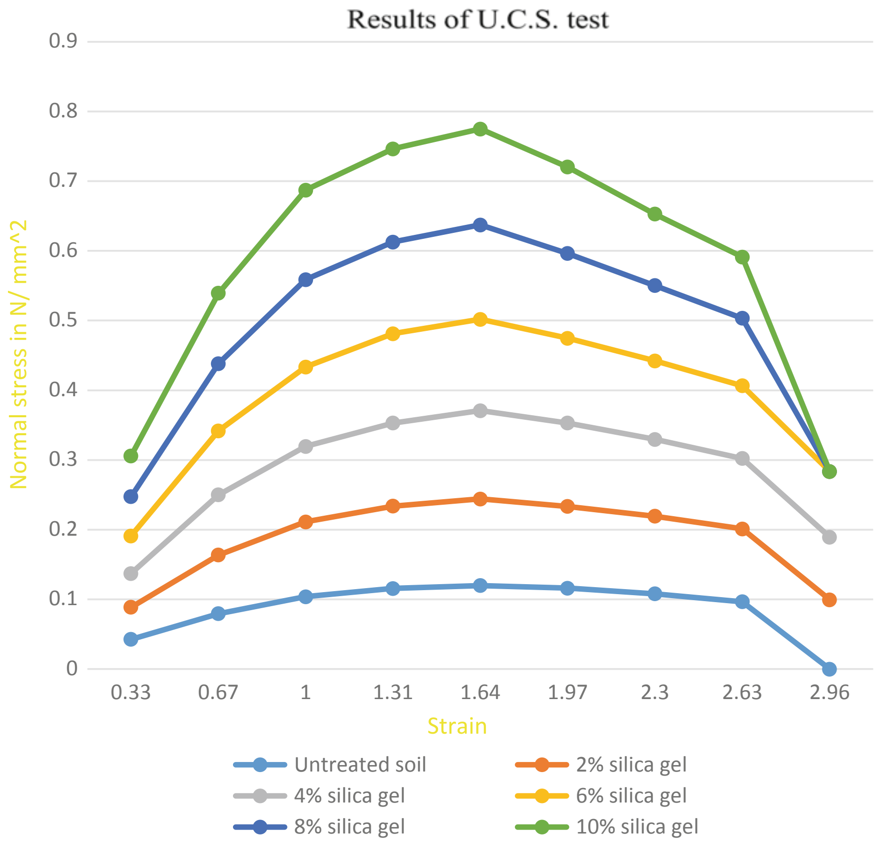

Fig. 2 Observation of unconfined compressive strength test results

5 Discussion

By adding increasing percentage of silica gel in natural soil decrease in liquid limit, plasticity index, MDD, specific gravity is observed; while there is increase in plastic limit, shrinkage limit, shrinkage index and cohesive strength of soil. This is mainly due to presence of silica gel; silica gel is mainly composed of silicon dioxide or silica (SiO_2). On addition of silica gel as a admixture in soil, this silicon dioxide reacts with calcium and other minerals present in soil and form dicalcium silicates, tricalcium silicates, etc., trisilicates compound hydrates and hardens rapidly and responsible for initial set and strength gain while other silicates hydrate slowly and increase strength gradually.

From the observations, it is clear that mixing of silica gel in expansive soil reduces the swelling properties effectively, it is clearly visible by adding 10% silica gel admixture in soil by mass, and it brings down the value of free swell index from 41.66 to 24.34%.

Table 1 Test results on virgin soil and treated soil

Test conducted	Properties	Virgin soil	2% silica gel	4% silica gel	6% silica gel	8% silica gel	10% silica gel
1. Determination of moisture content	Moisture content	7.6%	–	–	–	–	–
2. Determination of specific gravity	Specific gravity	2.51	2.49	2.26	2.22	2.0	1.98
3. Free swell index	Swell index	41.6	32.41	28.41	25.71	24.63	24.34
4. Atterberg's limit	Soil classification	MH	MI	MI	MI	MI	MI
5. Standard proctor test	OMC in % MDD in g/cc	15.7 1.59	14.9 1.585	14.7 1.575	14.3 1.57	14.2 1.565	14 1.53
6. UCS test	UCS strength in N/mm^2	0.12	0.124	0.126	0.130	0.135	0.138
	Cohesive strength in N/mm^2	0.06	0.062	0.063	0.065	0.067	0.069

Besides this, liquid limit and plasticity index are lower down by about 10%. But there is concern of worry due to decrease in specific gravity of soil on increasing percentage of silica gel admixture. Therefore, the combination of silica gel with another admixture which increases the specific gravity of soil is preferred.

6 Conclusions

Based on the experimental studies, the following conclusions were drawn.

1. Soil used in this project was classified as MH, and silt is having liquid limit greater than 50. Hence, proper treatment of compaction or use of suitable admixture is required to stabilize the soil.
2. Silica gel admixture reduces the free swell index and swelling properties but reduction in specific gravity is the concern of worry. Thus, another suitable admixture to increase specific gravity along with silica is used for the purpose of soil stabilization.
3. The decrease of the maximum dry unit weight with the increase of the percentage of silica gel is mainly due to the lower specific gravity of the silica gel compared with expansive soil and immediate hydration process which reduces the density of soil. The decrease in dry density with increase in silica gel is also not a desirable result.

4. The unconfined compressive strength of the natural soil was 0.12 N/mm^2, and on adding 10% silica gel admixture it slightly increased to 0.138 N/mm^2.

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Utilization of Geo-Grid for Improving the Strength of Subgrade Layer with Fly Ash

Paul Basudeb and Paul Sanjay

Abstract The present study aims at exploring the possibilities of utilization of fly ash and geo-grid for rural road. The performance of fly ash and geo-grid reinforced with soil is studied by conducting California bearing ratio (CBR) test. Studies have been carried out by considering different percentages of fly ash by placing the geo-grid sheets within the different depth positions. The motivation of the present work is to reach the soaked CBR value of a subgrade layer which is above 10%, as per I.R.C.: S.P.-72-2007. Under the soaked condition, the geo-grid shows the maximum penetration resistance and gives greater CBR values. The soaked CBR values of soil–fly ash mixed with geo-grid at different depths of single layer show that the CBR values decrease continuously with the increase in the depth of geo-grid. Also the optimum position of geo-grid to get higher value of CBR is 0.2*H* from the top of the specimen, where *H* is the total height of the soil specimen in CBR mould.

Keywords Soil · Fly ash · Geo-grid · CBR · Subgrade

1 Introduction

The performance of a pavement is very responsive to the characteristics of the soil subgrade, which provides base for the whole pavement structure. The present study aims at exploring the possibilities of utilization of fly ash and geo-grid for the improvement of rural road. Number of research works have been carried out to improve the performance of subgrade soil by mixing it with fly ash and geo-grid according to [3, 6, 7]. The performance of fly ash and geo-grid reinforced with soil has been studied by conducting California bearing ratio (CBR) test. In the present

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case, the studies have been carried out by adding soil with different percentages of fly ash and combining them with the geo-grids along the different depth positions.

2 Motivation and Objective

In this study, fly ash and geo-grids have been used as stabilizing materials. According to [4], the soaked CBR value of a subgrade layer is above 10%. The motivation in this paper is to achieve this CBR value by using different percentages of fly ash mixed with the soils layered with geo-grid at different single-layered depths.

3 Methodology

Different laboratory experiments have been performed to find out the physical and engineering properties of the soils collected from NIT, Agartala campus, Tripura, India (Table 1). The fly ash sample was collected from Kolaghat Thermal Power Plant, West Bengal, India, having specific gravity 1.98.

The dry soil sample has been mixed with different percentages of fly ash (10, 20, 30 and 40%) and also reinforced with geo-grid at different placement depth as shown in Fig. 1. The heavy compaction tests have been performed to find the maximum dry density (M.D.D.) and optimum moisture content (O.M.C.) of the soil. Also the California bearing ratio (CBR) values of the soil mixed with fly ash and reinforced with geo-grid in different depths (0.1H, 0.2H, 0.3H, 0.4H, 0.6H, 0.8H, where H is the thickness of the soil layer) are performed according to [5].

Table 1 Physical and engineering properties soil sample

Physical properties	Experimental data
Specific gravity (G)	2.60
Sand particles (0.075–4.75 mm, %)	53.28
Silt particles (0.002–0.075 mm, %)	35.32
Clay particles (<0.002 mm, %)	11.40
Name of the group and symbol	Silty–Sand (SW-SM)
M.D.D. (gm/cc) [Heavy compaction]	2.00
O.M.C. (%) [Heavy compaction]	11.80

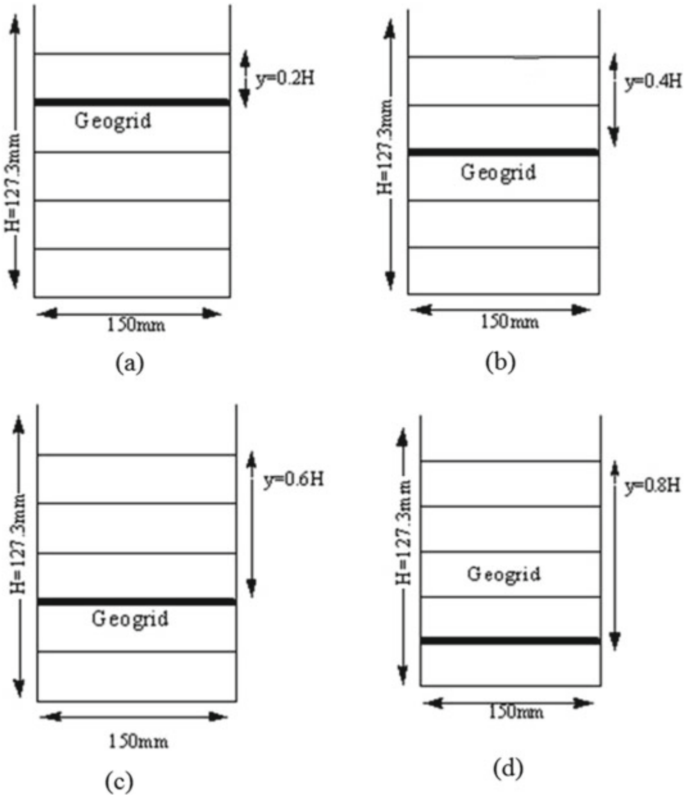

Fig. 1 Placement of geo-grid against the different depths CBR mould

3.1 Properties of Geo-Grids

Biaxial geo-grids are collected from the Kolkata new market (makers TENSAR). Thickness of this geo-grid is 1.50 mm, and grid size is 10 × 10 mm (Fig. 2).

4 Results and Discussions

The CBR values of soil and fly ash mixtures have been obtained under soaked conditions. Also the soaked CBR values have been obtained by combining soil, fly ash and geo-grid together.

Fig. 2 Photo of geo-grid

4.1 CBR Values of Soil and Soil–Fly Ash Combinations

The soaked CBR tests have been performed for the soil and the soil mixed with fly ash at different percentage which are presented in Table 2. It is observed that the maximum value of the CBR is attained after adding an optimum percentage of fly ash, beyond that the CBR value reduces.

The gain of strength of fly ash stabilized soil is primarily a result of pozzolanic reactions between silica and alumina from the soil and fly ash to form various types of cementing agent in presence of water. Similar trends have been obtained according to [1, 8].

Table 2 Variation of CBR with soil and soil–fly ash combinations under soaked conditions

S. No.	Combinations of soil and fly ash		CBR value (%)	Relative increase in CBR value (%)	Percentage increase in CBR value
	Percentage of soil	Percentage of fly ash			
1	100	0	4.65	–	–
2	90	10	5.43	0.78	16.77
3	80	20	6.7	2.05	44.08
4	70	30	7.58	2.93	63.01
5	60	40	7.09	2.44	52.47

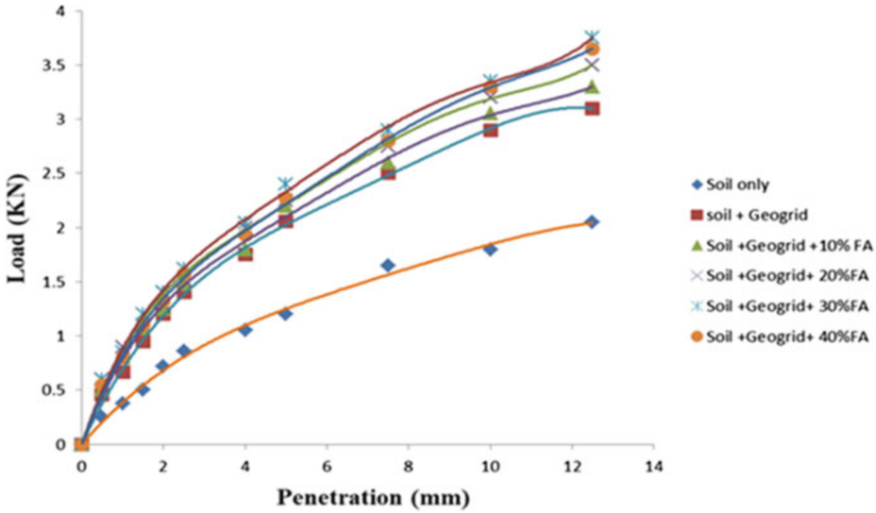

Fig. 3 Soaked CBR test result considering geo-grid at 0.1*H* depth

4.2 CBR Values of Soil, Fly Ash and Geo-Grid Combinations

The soaked CBR tests have been performed for the soil and the soil mixed with fly ash at different percentage combinations against the different depth of placement of geo-grid under soaked conditions according to Fig. 1.

Figures 3 and 4 show typical CBR test results under soaked conditions considering the geo-grid at 0.1*H* and 0.2*H* depth, respectively. The CBR test results are also presented in Table 3. The geo-grids have been placed at 0.1*H*, 0.2*H*, 0.3*H*, 0.4*H*, 0.6*H* and 0.8*H* depth from the upper portion of CBR mould.

Figure 5 shows the variation of percentage increase in CBR values against the depth of placement of geo-grid for different fly ash percentages. It has been observed that the use of geo-grid with different percentage of fly ash plays an important role in the performance of CBR values. Under the soaked condition, the geo-grid shows the maximum penetration resistance and gives greater CBR values. This is because it has a very good interlocking and frictional capability and therefore provides the high tensile resistance to any lateral movement of soil and therefore improves the strength of soil. For all placement depth, this behaviour is similar.

It can be observed that the CBR values decrease continuously with the increase in the depth of geo-grid. Thus, it is concluded that lowering the position of geo-grid actually reduces the percentage increase in CBR values of soil.

The stress applied by the CBR plunger is distributed according to Boussinesq equation of stress distribution [2]. Applying [2], stress developed at the level of different position of geo-grid is summarized in Table 4. Introduction of geo-grid at a depth of 0.2*H* where 63.70% of stress is coming will show lower strain as the grid

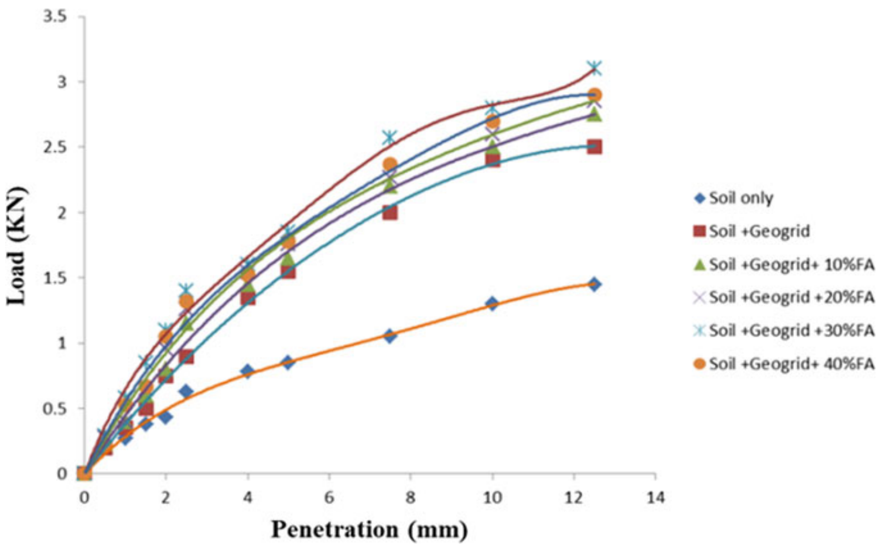

Fig. 4 Soaked CBR test result considering geo-grid at 0.2*H* depth

has got higher modulus of elasticity than soil. This justifies getting higher CBR value at this position.

The results obtained from the present study may be comparable with the finding by the previous researchers [9].

5 Conclusions

Based on the above results and details discussions, following conclusions may be made:

- The maximum value of the CBR is attained after adding an optimum percentage of fly ash, beyond that the CBR value reduces.
- Under the soaked condition, the geo-grid shows the maximum penetration resistance and gives greater CBR values.
- The CBR values decrease continuously with the increase in the depth of geo-grid, thus lowering the position of geo-grid actually reduces the percentage increase in CBR values of soil.
- Using geo-grid at a depth of 0.2*H* (*H* is soil specimen height in CBR mould) shows higher percentage of stress (63.70%), which will show lower strain as the grid has got higher modulus of elasticity than soil. This justifies getting higher CBR value at this position.

Table 3 Variation of CBR with soil and soil–fly ash (FA) combinations against the different depth of placement of geo-grid under soaked conditions

S.. No.	Depth of placement of geo-grid	Percentage of fly ash	CBR value (%)	Relative increase in CBR value (%)	Percentage increase in CBR value
1	–	–	4.65	–	–
2	0.1*H*	0	5.07	0.42	9.03
3	0.1*H*	10	5.23	0.58	12.47
4	0.1*H*	20	5.36	0.71	15.26
5	0.1*H*	30	5.61	0.96	20.64
6	0.1*H*	40	5.58	0.93	20.00
7	0.2*H*	0	7.30	2.65	56.90
8	0.2*H*	10	8.40	3.75	80.60
9	0.2*H*	20	9.20	3.95	97.80
10	0.2*H*	30	10.2	5.55	119.3
11	0.2*H*	40	9.65	4.05	107.5
12	0.3*H*	0	6.13	1.48	31.83
13	0.3*H*	10	6.27	1.62	34.84
14	0.3*H*	20	6.41	1.76	37.85
15	0.3*H*	30	6.92	2.27	48.82
16	0.3*H*	40	6.87	2.22	47.74
17	0.4*H*	0	6.80	2.15	46.23
18	0.4*H*	10	7.30	2.65	56.98
19	0.4*H*	20	7.80	3.15	67.75
20	0.4*H*	30	8.50	3.85	82.8
21	0.4*H*	40	7.95	3.3	70.97
22	0.6*H*	0	6.60	1.95	41.94
23	0.6*H*	10	7.10	2.45	52.70
24	0.6*H*	20	7.60	2.95	63.40
25	0.6*H*	30	8.02	3.37	72.47
26	0.6*H*	40	7.81	3.16	67.95
27	0.8*H*	0	6.46	1.91	41.07
28	0.8*H*	10	6.93	2.28	49.03
29	0.8*H*	20	7.30	2.65	56.98
30	0.8*H*	30	7.73	3.08	66.25
31	0.8*H*	40	7.44	2.79	60.00

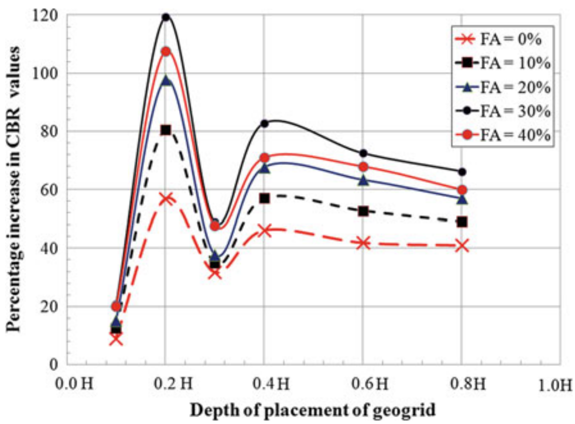

Fig. 5 Variation of percentage increase in CBR values against the depth of placement of geo-grid for different fly ash percentages (FA), where H is the thickness of the soil layer

Table 4 Stress developed at different position of geo-grid in CBR mould

Depth of reinforcing layer from the top	% stress
0.1H	7.68
0.2H	63.70
0.3H	41.03
0.4H	27.70
0.6H	14.20
0.8H	8.40

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Stabilization of Contaminated Soil by Mixing of Corn Husk Fibers

Priyanka Roy and Prasun Mukherjee

Abstract Soil contamination has always been a major cause for land degradation and deterioration of soil properties in various ways. Numerous activities involving the use of petrochemicals on a daily basis, accidents leading to oil spilling, and pipeline or reservoir leakage lead to the contamination of soil. In addition to this contamination is also leading to groundwater pollution, altering the geotechnical properties of soil. The uses of natural fibers incorporated soil as a construction material since ancient times, led to the understanding of the variations in soil properties and also the need to improve these properties to achieve desired construction proficiency. Thus, with this view, this research work deals with the improvement of compaction and strength characteristics of synthetic oil contaminated soil by random mixing of corn husk fibers at different percentages of 1, 2, 3, and 4% by weight of soil. The experimental study was conducted on soil samples which were prepared artificially by mixing synthetic oil at 5, 10, and 15% by weight of soil. Index properties of soil were tested, and then proctor compaction test and CBR tests were conducted in order to observe the changing pattern with the addition of corn husk fiber. Results showed marked improvement in compaction and strength characteristics of soil. The pH value of soil was also determined and results showed a remarkable change in soil characteristics from acidic in nature toward neutralization with addition of corn husk fiber.

Keywords Stabilization · Contaminated soil · Corn husk fiber, pH value · CBR improvement

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1 Introduction

Contamination can lead to an accumulation of contaminants in soils. It may frequently cause damage to buildings, dams, highways, etc., situated in those sites as well as can make the site unsuitable for developmental activities. The major chemicals implementing concern are petroleum hydrocarbons, polynuclear aromatic hydrocarbons (such as naphthalene and benzo(a) pyrene), solvents, pesticides, lead, and other heavy metals. Contamination is directly proportional to the degree of industrialization and severity of chemical substance. The emphasis over soil contamination stems primarily due to health risks either from direct contact with the contaminated soil, or vapors from the contaminants, also the water bodies and supplies within and underlying the soil can get contaminated.

Air, water, and land are being contaminated for short-term benefits by industrial, petrochemical, construction, and sanitary activities. Considering land contaminations, environmentalists are concerned about subsurface water aquifer contaminations, plant growth in contaminated soil, and environmental and health hazards. Nevertheless, geotechnical experts are continuously studying and experimenting the effects of soil contamination on the geotechnical properties of the soil. The soil-bearing capacity, foundation settlement, shear resistance, compressibility, and plasticity are the factors that must be taken into consideration.

Synthetic oil is a lubricant composed of chemical compounds which are artificially made. The exact process of synthesizing and composition of different additives is generally a commercial trade secret and vary among producers.

Synthetic oil is used as a replacement for petroleum-refined oils when operating in high temperatures. Hydrocarbons present in synthetic oil reduce permeability and in turn reduces the strength and compaction characteristics of natural soil.

Since ancient ages, many natural materials were recognized for use as reinforcement in soft soils. There are evidences of the use of woven mats made of reeds in temples of Babylonia, the use of tree twigs with leaves (as tensile elements) in construction of Great Wall of China, uses of straw and hay to reinforce mud blocks, and bamboo thatch in mud walls in India. In Kerala, there is an age old practice of spreading coconut leaves over sub-grades. Also stolons of trees were laid on soft marshy soils to facilitate walking in some developing countries.

However, intensive research started in early 1950s to find means of utilizing the accumulating end products during fractional distillation of crude oil refineries for gainful use and eliminating disposal problem for such end wastes. In the process polymeric materials like poly amide, polyethylene, etc., were developed for making synthetic fibers. Worldwide research started to develop methods of using such material for improving soil properties to desired levels. Thus, the concept of geosynthetics originated. The success of man-made geosynthetics was completely based on vigorous research, studies, and trials and its growth has been remarkable over the last five decades. The concept of reinforcing soil with inclusion was given by French architect-engineer Henri Vidal in 1963 and was employed in construction of retaining walls which were reinforced by inclusion of linear strips placed horizontally. Later, this concept was verified in 1969 by Vidal and Schlosser in LCPC cohesion theory, also Hausmann [1] proposed Sigma model and Tau model, dealing with bond and tensile failure of soil with reinforcement. Since then the concept of reinforced earth has been well established and present conventional reinforcement methods consist of inclusion of fibers, strips, grids into the soil mass for improving the bearing capacity, filter and drainage control in soils, etc. Recently, methods of random mixing of various types of fibers have attracted increasing attention in many geotechnical applications like airfield construction. In fact use of geosynthetics in civil engineering heralded a new revolution in civil engineering.

However, presently, the market is dominated by synthetic geotextiles, but it has certain disadvantages compared to natural geotextiles. They are:

(i) Synthetic geotextiles are not eco-friendly, particularly where they are laid open to sunshine/atmosphere.
(ii) The products are not renewable.
(iii) Increasing price of polymeric raw materials, due to the diminishing amount of available crude in earth, leads to increase in the overall cost of geotextiles.

Thus, the need of an eco-friendly, renewable, abundantly available, and economically viable alternative became important in environ conscious and economically backward countries. As a result, search for suitable natural geotextiles has been important. In the last two decades, reports on soil used with different natural geotextiles made from natural fibers are in evidence. Many natural fibers have been identified and experimented for possible use in construction.

Mainly, geotextiles made from jute, coir, and sisal are commonly been tested and used. Some of the basic advantages of such geotextiles are:

(i) High moisture absorbing capacity
(ii) High initial tensile strength
(iii) Low extension at break
(iv) Bio degradable, soil nourisher
(v) Renewable resource, easily available
(vi) Economic and eco-friendly.

Researches and experiments have been conducted by using the method of random mixing of natural fibers in order to improve the properties of contaminated soil. Though no extensive work has been reported using corn husk fibers in synthetic oil contaminated soil.

2 Literature Review

Extensive research work has been reported for improving the properties of contaminated soil by different stabilization methods. They are as follows:

(i) Al-Sanad et al. [2] and Al-Sanad and Ismael [3] conducted laboratory testing programs to evaluate the influence of oil contamination and aging effects on the geotechnical properties and behavior of Kuwaiti sand.
(ii) Aiban [4] studied the effect of temperature on the strength, permeability, and compressibility of oil-contaminated sand obtained from eastern Saudi Arabia. He found that the compressibility and permanent deformation of the oil-contaminated sand increased as the temperature increased above room temperature and that the shear strength parameters were not sensitive to the testing temperature.
(iii) Evgin and Das [5] conducted triaxial tests on clean and oil-contaminated quartz sand. They found that full saturation with motor oil caused a significant reduction in the friction angle of both loose and dense sands and a drastic increase of the volumetric strain. They also showed through a finite-element analysis that the settlement of footing increased due to oil contamination.
(iv) A laboratory testing program was performed to determine the effects of crude oil contamination on some of the geotechnical properties of clayey and sandy soils from the coastal soils from Persian Gulf beaches by Khamehchiyan et al. [6]. Their testing program examined basic properties, Atterberg limits, compaction, direct shear, uniaxial compression, and permeability on clean and contaminated soil samples that had the same density.
(v) Ur-Rehman et al. [7] conducted another laboratory testing program to compare the engineering properties of uncontaminated and contaminated clay. They reported that the contaminated clay behaved more like cohesion less materials due to the formation of agglomerates. This contamination has been shown to affect the plasticity and the cation exchange capacity (CEC) of the investigated clay.
(vi) Shroff [8] mentioned that physical, chemical, biological, and thermal methods can be used for improvement of the strength of crude oil contaminated soil. The author reported that for best results to stabilize crude oil contaminated soil, the mixing of 10% lime, 10% cement, and 5% attapulgite clay may be used.

Some work has also showed the effective use of corn husk as a reinforcement material in soil. They are as follows:

(i) The study by Akinloye et al. [9] assessed the effect of corn husk ash on the engineering properties of lateritic soil. The lateritic soil, which was modified with corn husk and lime as stabilizing agents, was obtained from Kuedenda along Kaduna-Abuja expressway, Kaduna. The optimum moisture content and maximum dry density of the laterite soil without stabilizing were 15.24% and 1.73 g/cm^3, respectively. The compressive strength was observed to be increased with curing age. Thus, it was deduced that lateritic soil strength can be improved at 10% each for lime and corn husk.
(ii) A study was conducted by Yalley [10] to investigate the potential of corn husk as an enhancer for the production of soil blocks for low cost housing. Five different levels of stabilization (0, 5, 10, 15, and 20%) using corn husk ash were adopted for this study. Fifteen blocks were molded for each stabilization

level. In all, a total of 75 blocks was molded and subjected to the compressive strength, abrasion resistance, and water absorption by capillarity tests after curing 28 days and compared with the relevant standards of compressive earth blocks. In general, there was a significant improvement in the compressive strength characteristics of the stabilized soil blocks. Soil blocks mixed with 20% corn husk ash had the highest compressive strength of 5.311 MPa followed by blocks with 15% corn husk ash with compressive strength of 4.917 MPa.

3 Scope of Work

Studies made in literature review regarding the effect of natural fiber on contaminated soil, indicate that there are possibilities for improvement in the compaction, strength and pH characteristics of oil contaminated soil. Therefore, with the aim of getting more comprehensive results, corn husk fibers have been selected in this research work as inclusion in week cohesive soil for improving its compaction, strength, and pH characteristics.

For this project work, local soil was collected from Dakhshin Gobindapur area (South 24 Parganas, West Bengal) and the sample was artificially contaminated by mixing synthetic oil (mobil) at different percentages, i.e. 5%, 10%, and 15%, respectively, by weight of dry soil sample. The samples were then put in plastic bags and were placed in desiccator for 24 h. Since the soil was being artificially contaminated, therefore, in order to optimize the experimental results, the soil was mixed with three different percentages of synthetic oil.

The corn husks were collected from Baruipur area (South 24 Parganas, West Bengal) and used as an inclusive and mixing material for studying its effects on compaction, strength, and pH characteristics of the contaminated soil sample.

For vigorous study, the prepared soil samples were subjected to different routine testing in order to determine the index properties, compaction characteristics, California bearing ratio (CBR) value at optimum moisture content (OMC), as well as pH characteristics of the soil sample.

The experimental work was conducted by randomly mixing the corn husk leaves in the contaminated soil sample. The corn husks were to be cut in three different sizes namely 1 cm × 1 cm, 2 cm × 1 cm, 3 cm × 1 cm, and 4 cm × 1 cm. For each of these sizes, the fibers were to be mixed with dry soil in different percentages of 1, 2, 3, and 4% by weight of dry soil. OMC and MDD were to be evaluated and un-soaked as well as soaked CBR value at OMC value was to be found for each of these cases. The result of all these tests was to be utilized to see the effect of mixing corn husk fibers of a particular size; at a particular percentage by weight of contaminated soil, on compaction characteristics and un-soaked CBR values at OMC. For pH determination, the corn husk fibers were grinded into a particle size similar to the soil particles and were mixed with the contaminated soil at different percentages of 2, 4, and 6%.

Table 1 Properties of the three contaminated soil sample

Soil properties	Soil contaminated with different percentages of synthetic oil		
	5% synthetic oil	10% synthetic oil	15% synthetic oil
Liquid limit	47%	40	32
Plastic limit	33%	29	19
Specific gravity	2.42	2.42	2.42
Sand (%)	3.9	3.9	3.9
Silt (%)	68.5	68.5	68.5
Clay (%)	27.6	27.6	27.6
Classification of soil	Inorganic clay of low plasticity (CL)	Inorganic clay of low plasticity (CL)	Inorganic clay of low plasticity (CL)
OMC (%)	17	15	11
MDD	1.77 g/cc	1.79	1.72
CBR (un-soaked)	4.5	4.4	4.0
CBR (soaked)	4.2	4.1	3.8

4 Materials Used for Experiment

4.1 Soil Sample

The locally available soil was collected from Dakshin Gobindapur, South 24 Parganas, West Bengal. In order to artificially contaminate the soil, for further studies, the collected soil samples were mixed with different percentages of synthetic oil. The index properties of the prepared soil samples were conducted and the results are reported in Table 1.

4.2 Corn Husk Fibers

The corn husks were collected from Baruipur area (South 24 Parganas, West Bengal) and used as an inclusive material for studying its effects on compaction and strength characteristics of contaminated soil sample. The physical properties of corn husk fibers are given in Table 2.

5 Experimental Results and Discussion

The experimental work involves the random mixing of corn husk fibers of different sizes of 1, 2, 3, and 4 cm at varying percentages of 1, 2, 3, and 4% by weight of dry

Table 2 Physical properties of corn husk fibers

Properties	Values
Width	6 cm
Thickness	0.39 mm
Mass per unit area	3.1×10^{-4} gm/mm^2
Unit weight	5.1×10^{-4} g/mm

soil. For appropriate experimental results, three different samples of contaminated soil was prepared. Sample 1 (S1), Sample 2 (S2), and Sample 3 (S3) were prepared by mixing dry soil with 5%, 10%, and 15% of synthetic oil, respectively. For each case of contamination, the soil sample was mixed with corn husk fibers of 1 cm length added at four different percentages, i.e. 1, 2, 3, and 4% by weight of dry soil and standard proctor test was conducted to determine the optimum moisture content and maximum dry density for each case of percentage variation. Both un-soaked and soaked CBR were then conducted by conventional ways at the predetermined OMC by randomly mixing the date palm leaves at different percentages. Also, the prepared soil samples were mixed with grinded corn husk fibers at different percentages of 2, 4, and 6% and their pH value was determined with a pH meter following the conventional test methods. The experimental results showing the OMC and MDD, CBR (both un-soaked and soaked) and pH values of the three soil samples are provided in Tables 3, 4 and 5, respectively.

The results indicate that there is a continuous increase in optimum moisture content with increase in percentage of fiber content for each particular length of corn husk fiber. Similarly, the maximum dry density goes on decreasing with the increase in percentage of corn husk fiber from 1 to 4% for each particular length of fiber. This pattern of increasing OMC and decreasing MDD has been noticed for all three samples of contaminated soil. As the water content increases, the pores gets filled with water, which hinders the close packing of the grains of soil and thus the dry density reduces. Also it is seen that with the increase in length of date palm leaves, when the percentage of fiber is increasing, the maximum dry density decreases. This is due to the fact that the leaves are occupying more space in the soil and is preventing proper compaction in soil.

The CBR values, however, showed an interesting orientation. The maximum CBR values (both un-soaked and soaked) were observed at a mixing percentage of 4% for 1 and 2 cm of fiber length, indicating the increase of CBR value with increase in percentage fiber. But in case of 3 cm fiber length, the maximum CBR value was observed at a mixing percentage of 3% after which it decreased with increase in fiber content. Similarly, in case of 4 cm fiber length, the maximum CBR value was obtained at a mixing percentage of 2% thereafter decreased with increased percentage of corn husk fiber. These results showed a similar trend for all three samples of contaminated soil.

The results for pH value of each soil sample showed improvement in pH value with increase in percentage of grounded corn husk fiber into the soil sample from 2 to 6%. The contaminated soil samples generally showed low pH value, indicating

Table 3 Values of optimum moisture content and maximum dry density with different length of corn husk fibers mixed at different percentages

Contaminated soil sample	% of corn husk fiber added (%)	OMC value without corn husk fibers (%)	OMC value with corn husk fibers of different length (%)				MDD value without corn husk fibers (g/cc)	MDD value with corn husk fibers of different length (g/cc)			
			1	2	3	4		1	2	3	4
	1	17	21	23	26	28		1.74	1.72	1.69	1.66
S1	2		25	27	29	31		1.72	1.7	1.66	1.63
	3		29	30	32	35	1.77	1.7	1.68	1.64	1.59
	4		32	34	36	38		1.68	1.66	1.62	1.56
	1	15	18	20	22	25		1.71	1.69	1.66	1.64
S2	2		21	24	26	28	1.79	1.69	1.65	1.61	1.59
	3		24	27	29	32		1.65	1.6	1.58	1.56
	4		27	30	33	35		1.62	1.58	1.56	1.54
	1	11	14	16	18	19		1.69	1.67	1.65	1.63
S3	2		18	19	21	23	1.72	1.66	1.63	1.6	1.58
	3		21	22	24	26		1.63	1.6	1.58	1.55
	4		25	25	27	29		1.6	1.58	1.55	1.53

Table 4 Values of un-soaked and soaked with different length of corn husk fibers mixed at different percentages

Contaminated soil sample	% of corn husk fiber added (%)	Un-soaked CBR value without corn husk fibers	Un-soaked CBR value with corn husk fibers of different length (g/cc)				Soaked CBR value without corn husk fibers	Soaked CBR value with corn husk fibers of different length (g/cc)			
			1	2	3	4		1	2	3	4
S1	1	4.5	5.2	5.7	6.0	6.3	4.2	4.9	5.2	5.6	6
	2		5.9	6.2	6.6	7.0		5.5	5.8	6.0	6.4
	3		6.5	6.7	7.8	6.6		6.1	6.3	7.3	6.1
	4		7.3	7.6	7.4	6.0		6.5	7.1	6.8	5.7
S2	1	4.4	4.8	5.0	5.3	5.7	4.1	4.6	4.7	4.9	5.3
	2		5.6	5.4	5.7	6.6		5.1	5	5.2	6
	3		6.2	6.1	6.9	5.9		5.7	5.6	6.3	5.6
	4		7.0	6.7	6.5	5.4		6.4	6.2	5.9	5
S3	1	4.0	4.4	4.8	5.2	5.5	3.8	4.1	4.3	4.7	5
	2		4.9	5.3	5.5	6.1		4.5	4.9	5.2	5.7
	3		5.3	5.9	6.1	5.7		4.9	5.3	5.7	5.4

Table 5 pH values mixed at different percentages of grounded corn husk fiber in contaminated soil

Contaminated soil sample	% of corn husk fiber added in grounded form (%)	pH value without corn husk fibers (g/cc)	pH value when soil is mixed with grounded corn husk fiber
S1	2	4.5	5.1
	4		5.9
	6		6.8
S2	2	3.8	4.6
	4		5.3
	6		6.2
S3	2	3.3	4
	4		5.2
	6		6

the sample to be acidic in nature. The addition of grounded corn husk fiber into the soil showed improvement in pH value of the soil thus indicating the neutralization of the soil sample.

Comparing the experimental values for OMC, MDD, CBR (un-soaked and soaked) and pH of Sample 1 (soil mixed with 5% synthetic oil) with that of Sample 2 (soil mixed with 10% synthetic oil) and Sample 3 (soil mixed with 15% synthetic oil), it has been clearly observed that with the increase in synthetic oil content, the overall results showed a decreasing trend.

The graphical representation of the results of OMC, MDD un-soaked CBR, and soaked CBR for soil Sample 1 is given in Figs. 1, 2, 3 and 4, respectively.

The graphical representations of the results of OMC, MDD un-soaked CBR, and soaked CBR for soil Sample 2 are given in Figs. 5, 6, 7 and 8, respectively.

The graphical representations of the results of OMC, MDD un-soaked CBR, and soaked CBR for soil Sample 3 are given in Figs. 9, 10, 11 and 12, respectively.

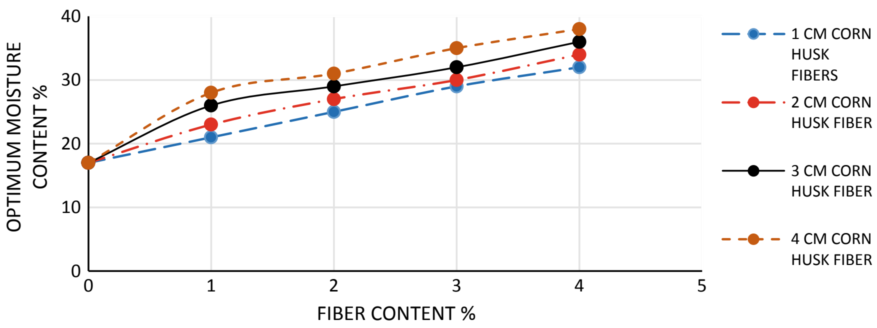

Fig. 1 OMC versus fiber content graph for Sample 1

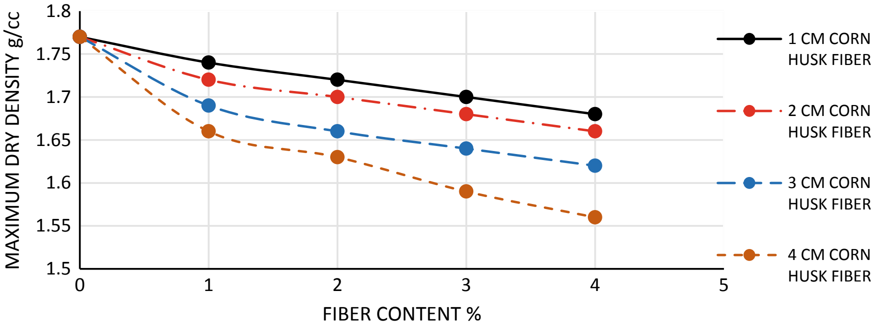

Fig. 2 MDD versus fiber content graph for Sample 1

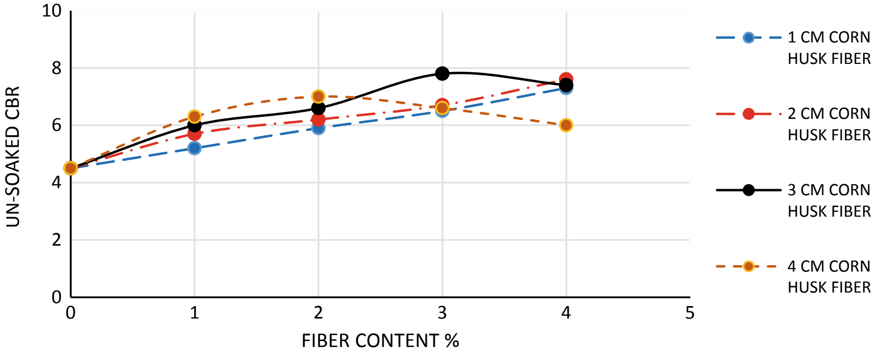

Fig. 3 Un-soaked CBR versus fiber content for Sample 1

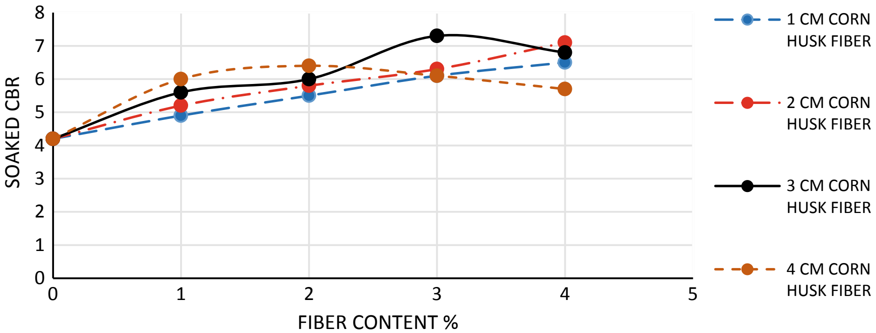

Fig. 4 Soaked CBR versus fiber content for Sample 1

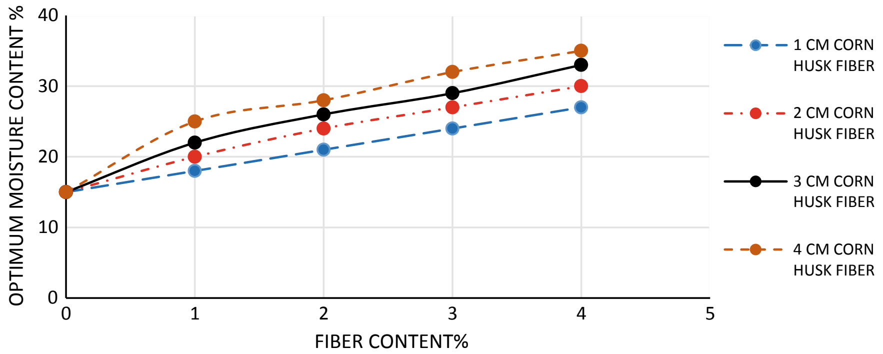

Fig. 5 OMC versus fiber content graph for Sample 2

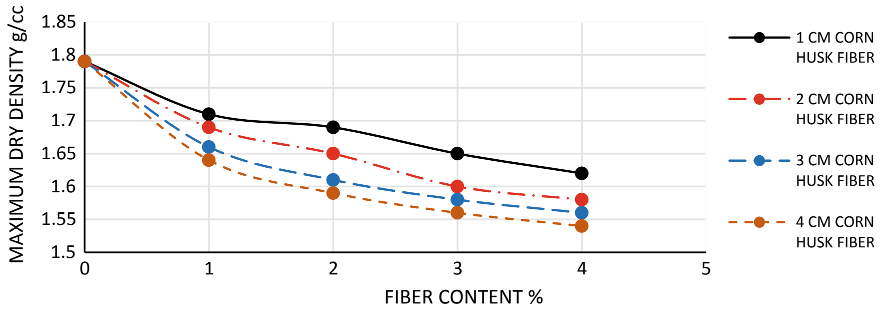

Fig. 6 MDD versus fiber content graph for Sample 2

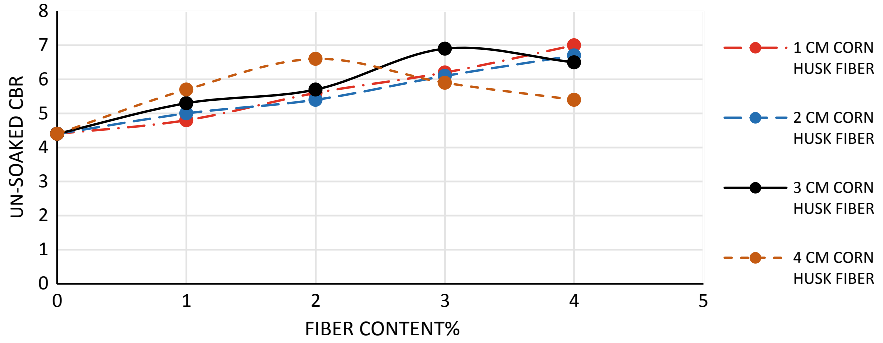

Fig. 7 Un-soaked CBR versus fiber content for Sample 2

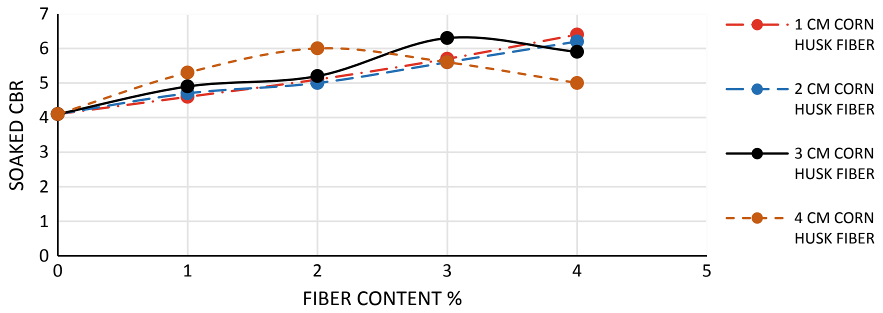

Fig. 8 Soaked CBR versus fiber content for Sample 2

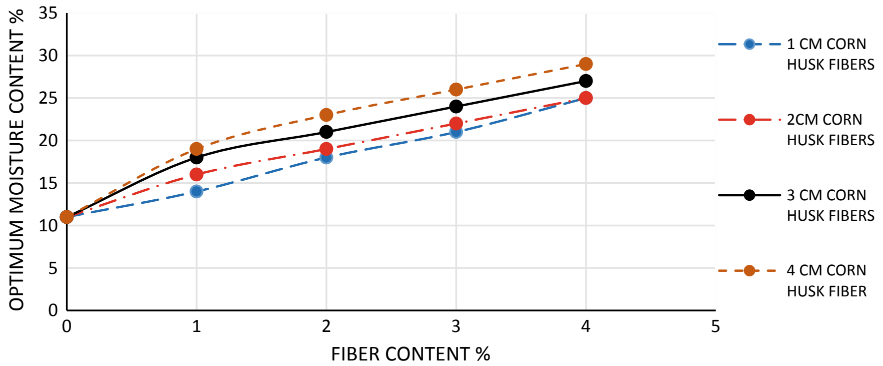

Fig. 9 OMC versus fiber content graph for Sample 3

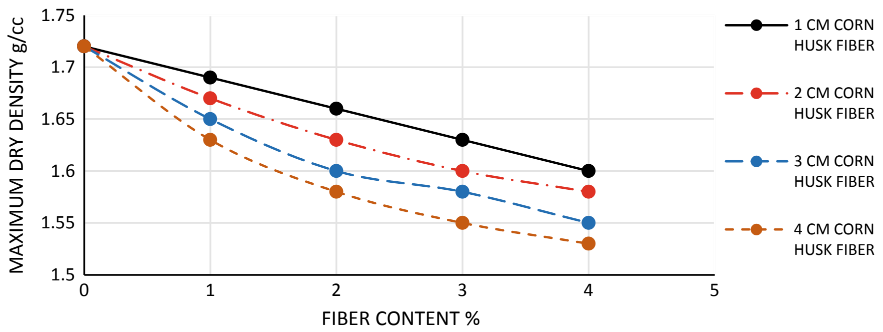

Fig. 10 MDD versus fiber content graph for Sample 3

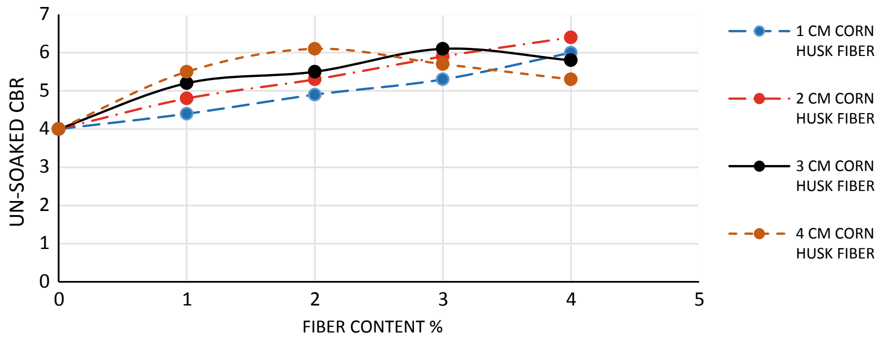

Fig. 11 Un-soaked CBR versus fiber content for Sample 3

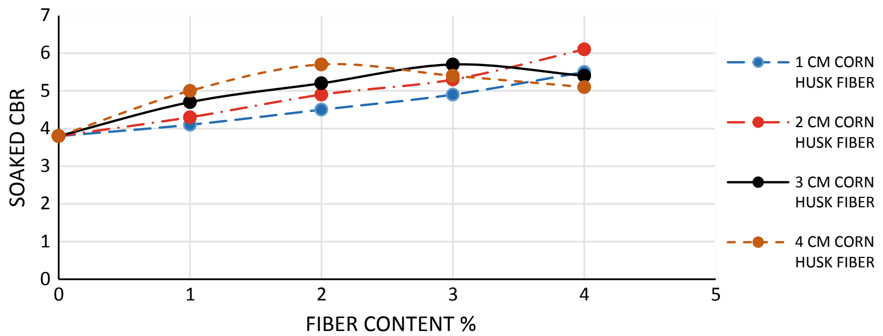

Fig. 12 Soaked CBR versus fiber content for Sample 3

6 Conclusion

On the basis of extensive study on the effect of corn husk fibers in random mixing in contaminated soil, following conclusions are drawn. These conclusions are presented sequentially initially on compacted contaminated soil randomly mixed with corn husk fibers.

(i) For all three samples of contaminated soil, there is an increase in optimum moisture content, pH value, and California bearing ratio (Soaked and un-soaked) value as well as a decrease in maximum dry density when corn husk fibers are randomly mixed within the soil sample.
(ii) For Sample 1, the maximum CBR improvement was 73% with corn husk fiber length of 3 cm mixed at a percentage of 3%.
(iii) For Sample 2, the maximum CBR improvement was 59% with corn husk fiber length of 1 cm mixed at a percentage of 4%.
(iv) For Sample 1, the maximum CBR improvement was 60% with corn husk fiber length of 2 cm mixed at a percentage of 4%.

(v) Therefore, Sample 1 showed better CBR improvement compare to the other two Samples.
(vi) For Sample 1, pH value was increased as 51% for a corn husk fiber mixing percentage of 6%.
(vii) The pH value for Sample 2 was increased by 63% when mixed with 6% of corn husk fibers.
(viii) The pH value improvement for Sample 3 was 81% at a mixing percentage of 6%. Therefore, it can be concluded that addition of more corn husk fiber will provide better improvement in pH value of the contaminated soil.
(ix) The results indicate that with the increase in fiber size, there is a decrease in maximum dry density and also in CBR value; therefore for optimum results smaller fiber sizes are appropriate for application. The results may be utilized for strengthening contaminated soil by reinforcing with corn husk fibers, which in general have less strength due to contamination by soil.
(x) Improvement in pH value reduces the acidity in soil which occurred due to contamination. As a result, the soil will improve in terms of strength and fertility, thus improving the overall index properties of the soil sample.
(xi) Thus with the improvement in strength of contaminated soil, the soil could be used as a subgrade material which in turn will reduce the other materials used in base, and surface layers can be reduced, thus reducing the thickness of the roads as well as will reduce the cost of road construction. Also the fertility of the effected soil can be improved which will also be beneficial for irrigational purposes. Therefore, the experimental values can be used beneficially for practical road construction purposes and mainly as a remedial solution for contaminated soil to be used as a construction material.

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Laboratory Investigation of Black Cotton Soil—Fly Ash—Steel Slag Mixes

Mangesh Subhash Ambekar and Hrishikesh Ashok Shahane

Abstract Black cotton soil is one of the major soil deposits in India. They exhibit high swelling and shrinking when exposed to changes in moisture content and hence has found to be most troublesome from engineering considerations. In this study, geotechnical characteristics of black cotton soil with the addition of fly ash (FA) and steel slag (SS) mixes were investigated. FA and SS were added in the varying percentage of 5, 10, 15, and 20% of the weight of black cotton soil. Atterberg's limit tests illustrated that 5–20% addition of FA and SS in the black cotton soil considerably reduces the volume change potential of the black cotton soil. Addition of FA and SS in black cotton soil also helps in reducing its swelling and shrinkage potential. The strength gain of the mixes not only depends on fly ash and steel slag content but also on mix gradation. The angle of internal friction for all mixes and the cohesion part of the shear strength was higher than that of the black cotton soil owing to strong bonding between steel slag and fly ash particles. Apart from the experimental results, the use of fly ash and steel slag has proven to be an eco-friendly material thereby reducing the dumping and disposal problems associated with fly ash and steel slag.

Keywords Black cotton soil · Fly ash · Steel slag · Swelling · Shrinkage

1 Introduction

The expansive soils occur all over the world. India has large tracks of expansive soil known as black cotton (BC) soil. These soils are suitable for growing cotton

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and black or blackish-grey in color due to the presence of titanium oxide in small concentration. This soil has a high percentage of Montmorillonite clay mineral which is mainly responsible for expansive characteristics of the soil. This soil increases in volume on absorbing water during rainy seasons which results in swelling and softening of soil. Also, the volume of soil decrease when water evaporates from it in the summer season. Due to alternate swelling and shrinkage property, the construction of the foundation for structure over a black cotton soils poses a challenge to the civil engineers. To overcome this effect, many locally available materials and industrial waste such as coir waste, quarry dust, lime, fly ash, copper slag, murum, etc., [1–7] were used in combination of black cotton soil and the geotechnical properties of BC soil were enhanced. The use of the optimum amount of additives is mainly dependent on the mineralogical composition of the soils and additives. Utilization of industrial wastes in the geotechnical engineering field is the best method for improving the performance of BC soil.

Extensive research was carried out by the geotechnical investigators to reduce the swelling and shrinkage and enhance geotechnical properties of expansive soils by using industrial wastes. Nwaiwu et al. [8] reported that quarry dust can be successfully used for road construction containing black cotton soil area. However, the mixture does not become non-plastic after addition of quarry dust content. Partial addition of shredded tire waste with black cotton soil changed the geotechnical properties which were advantageous to engineering applications and greatly decreases the swelling potential of the black cotton soil [9]. Poh et al. [2] showed that the use of basic oxygen steel (BOS) slag fines produce enhancements in strength and durability, and also a reduction in expansion characteristics. Fly ash due to its pozzolanic properties beneficial in combination with lime in improving properties of soil. With the increase in the percentage of fly ash keeping amount of lime as a constant, strength of the respective soil-lime mixture was increased [10]. Miao et al. [11] presented a method of geopolymerizing black cotton soil (BCS) to ascertain its potential use in subgrades.

2 Materials and Methodology

In this study, laboratory investigation of BC soil, fly ash, and steel slag with different proportions was carried out. BC soil was collected from the nearby area of Nashik. Geotechnical properties of BC soil are given in Table 1. It has been observed that the liquid limit and plastic limit of soil sample were greater than 50% and 30%, respectively. Therefore, according to Casagrande's A-line chart, soil was classified as CH soil.

Fly ash (FA) for the present investigation was collected from the coal-based thermal power plant located at Eklahare, Nashik. FA in dry form was transported in airtight double polythene bags. The chemical constituents of FA are given in Table 2 and it is classified as Class F fly ash as per ASTM C 618 (ASTM 2018). Steel slag (SS) was collected from the electric arc-based Bhagawati Steel Cast PVT Ltd. located

Table 1 Geotechnical properties of black cotton soil

S. No.	Parameters	Results
1	Specific gravity	2.67
2	Liquid limit (%)	58.40
3	Plastic limit (%)	30.37
4	Plasticity index	28.03
5	Shrinkage limit (%)	18.93
6	IS classification	CH
7	Free swell index (%)	103
8	OMC (%)	24.50
9	MDD (gm/cc)	1.53
10	UCS (kg/cm^2)	0.29

Table 2 Physical properties and chemical composition of the fly ash and steel slag

S. No.	Constituents	Fly ash	Constituents	Steel slag
1	Silica	58.66%	Loss of ignition at 900 °C	5.20%
2	Magnesium oxide	1.82%	Silica	44.25%
3	Sulphur trioxide	0.76%	Alumina	9.10%
4	Sodium dioxide	0.62%	Iron oxide	24.10%
5	Total alkalies	92.56%	Calcium oxide	4.60%
6	Total chloride	0.03%	Magnesium oxide	0.40%
7	Loss on ignition	1.94%	Chromium oxide	Nil
8	Moisture content	0.25%	Total alkalis	2.80%
9	Specific gravity	2.43	Manganese oxide	8.10%
10			Phosphorus pentoxide	0.32%
11			Specific gravity	2.85

in Malegaon MIDC, Sinnar, Nashik, and chemical constituents of steel slag are given in Table 2.

Series of laboratory tests were carried out to find the geotechnical characteristics of black cotton soil, fly ash, and steel slag mixes. Details of the combination used are shown in Table 3.

Table 3 Details of black cotton soil-fly ash-steel slag mixture for test conducted

$W = W_{BC} + W_{FA} + W_{SS}$	Variation of W_{BC} (% by total dry weight)	Variation of W_{FA} (% by total dry weight)	Variation of W_{SS} (% by total dry weight)
Combination 1	100, 95, 90, 85, 80	0, 5, 10, 15, 20	0
Combination 2	100, 95, 90, 85, 80	0	0, 5, 10, 15, 20
Combination 3	85, 80, 75, 70	10	5, 10, 15, 20

For evaluation of various geotechnical properties of black cotton soil, fly ash, and steel slag mixes, following laboratory tests were performed. During this study, the standard procedures and precautions have been implemented as per IS 2720 Part-3 Sec I (2016) for specific gravity, IS 2720 Part-4 (2015) for grain size analysis, IS 2720 Part-5 (2015) for liquid limit and plastic limit, IS 2720 Part-6 (2016) for shrinkage limit, IS 2720 Part-40 (2016) for free swell index, IS 2720 Part-8 (2015) for compaction test and IS 2720 Part-10 (2015) for unconfined compression test (UCS).

3 Test Results and Discussion

Figure 1 shows the effect of fly ash, steel slag, and combinations of the fly ash and steel slag on the liquid limit (LL) of black cotton soil. It was observed that as the additive content increases, the LL value decreases. The LL of black cotton soil has been found as 58.4% for 0% additives. This is a reference datum to study the effect of additives namely fly ash and steel slag on LL of black cotton soil. In present case, the LL is decreased by 1.35% for 5% FA, 2.675% for 5% SS, and 3.86% for combination of 10% FA + 5% SS. So this reduction in LL is found to be reduced gradually with the increase in the percentage of additives. Finally, at 20% gross additives consisting of 80% BC with 20% FA gives 8.59% reduction in LL. In the case of 80% BC with 20% SS gives 11.49% reduction. Whereas, for 70% BC + 10% FA + 20% SS up to 14.84%, reduction in LL was observed.

Figure 2 shows the variation of the plastic limit (PL) for different proportions of BCS + FA + SS mixtures. It was observed that PL of 30.37% has found for 0% additives. In present case, the PL is decreased by 0.72% for 5% FA, 35% for 5% SS

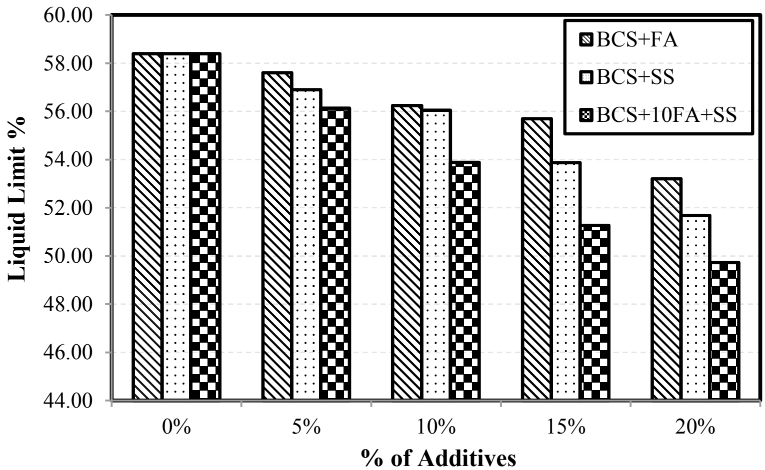

Fig. 1 Comparative effect of FA, SS, and FA + SS on liquid limit

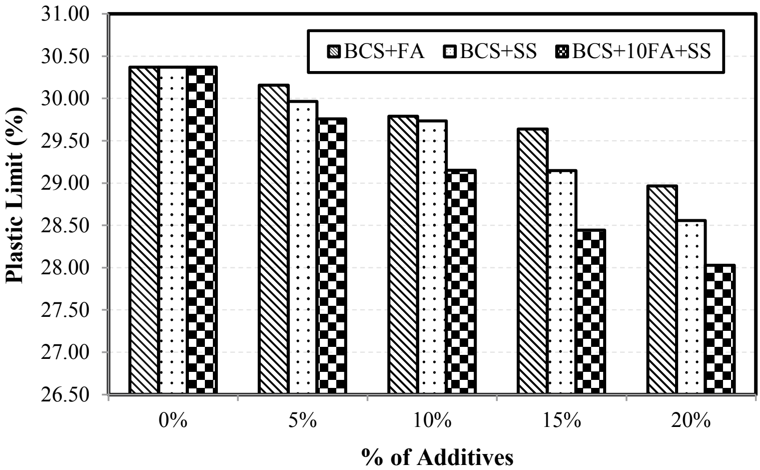

Fig. 2 Comparative effect of FA, SS, and FA + SS on plastic limit

and 2% for a combination of 10% FA + 5% SS. Similar observation as that of LL has been observed in a reduction in the PL of mixes as it reduced gradually with the increase in the percentage of additives. It is due to the reduction in the percentage of clay content in the mix [9].

From Fig. 3, it has been observed that the plasticity index (PI) of only black cotton soil is 28.03%. It was evident that the PI was decreased very rapidly with the addition of FA and SS. PI was decreased by 2.03% for 5% FA, 3.88% for 5% SS, and 5.88% for the combination of 10% FA + 5% SS. Finally, for the combination of 80% BC with 20% SS gives the highest reduction in PI of 17.49%. Liquid limit, plastic limit

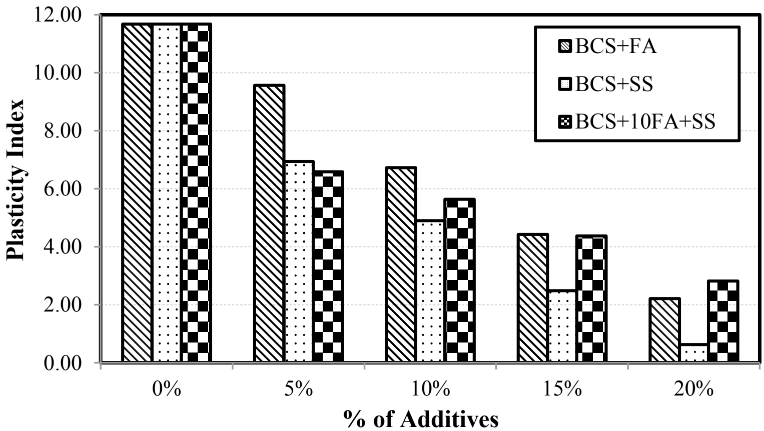

Fig. 3 Comparative effect of FA, SS, and FA + SS on plasticity index

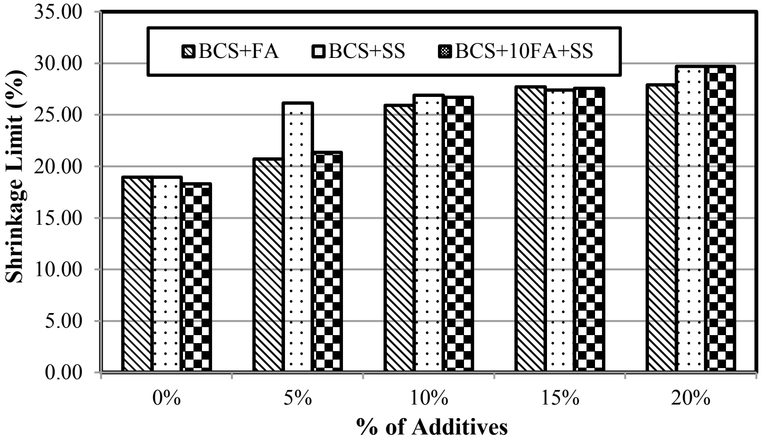

Fig. 4 Comparative effect of FA, SS, and FA + SS on shrinkage limit

well as PI values, decreased consistently at higher fly ash and steel slag contents, and similar observations were observed by Nwaiwu et al. [8] and Srivastava et al. [9].

The shrinkage limit (SL) of natural soil mainly depends on the grain-size distribution of the soil. Even though clay-sized particles play an important role in the shrinkage phenomenon, there is an optimum clay content at which the shrinkage limit of soil becomes lower. From Fig. 4, it was observed that the shrinkage limit of 18.93% has found for 0% additives in BC soil. As FA and SS content increase the SL increases. Finally, at 80% BC with 20% FA gives 47.43% increase in the SL. In case of 80% BC with 20% SS gives 56.89% increase in SL. However, for 70% BC + 10% FA + 20% SS up to 20% increase in SL was noticed. In BC soil, the clay content reduced due to the addition of fly ash and steel slag resulting in increasing the shrinkage limit.

These results clearly show potential for reducing the shrinkage behavior of black cotton soil when mixed with FA and SS. It was also observed that the addition of coarser SS waste provides relatively higher values of shrinkage limit when compared to the corresponding values obtained for FA mixed with black cotton soil. Hence, the addition of coarser category waste material is more advantageous for controlling the shrinkage behavior of black cotton soils [9].

Addition of FA and SS to black cotton soil causes a reduction in the free swell index (FSI). In Fig. 5, it is shown that additions of up to 20% FA and SS caused a reduction in free swell index from 104 to 60%, and this represents a reduction of up to 40%. The reduction in FSI was found to be reduced gradually with the increase in the percentage of additives. These trends suggest that FA and SS, which are the granular material can be used to reduce the swelling potential of expansive clay soils. The possibility of using fly ash and steel slag to reduce the swelling potential of expansive soils offer an opportunity for a reduction in the volume of this waste product which would otherwise establish an environmental problem.

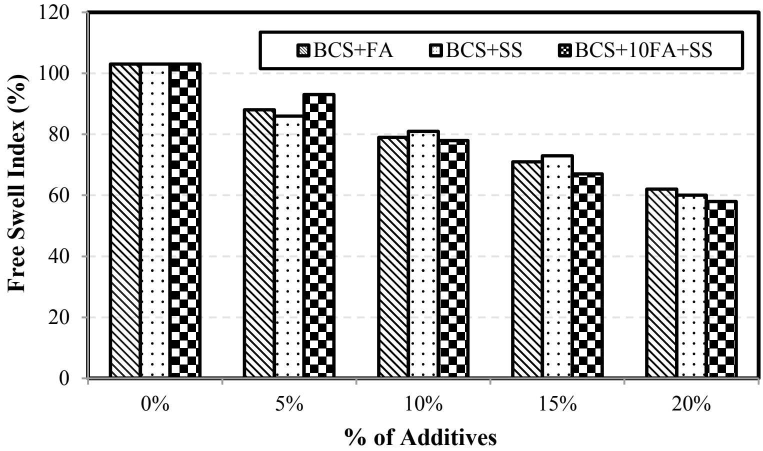

Fig. 5 Comparative effect of FA, SS, and FA + SS on free swell index

Figure 6 shows the variation of optimum moisture content (OMC) with additive content. From the results, it is clear that OMC decreases with the addition of FA and SS. OMC of 24.5% has been found for 0% additives content, i.e., for black cotton soil only. The OMC was decreased by 2.69% for 5% FA, 0.48% for 5% SS, and 7.42% for combination of 10% FA + 5% SS. So this reduction in OMC was found to be reduced gradually with the increase in the percentage of additives. Also in case of 70% BCS + 10% FA + 20% SS up to 14.61% reduction in OMC was observed. From

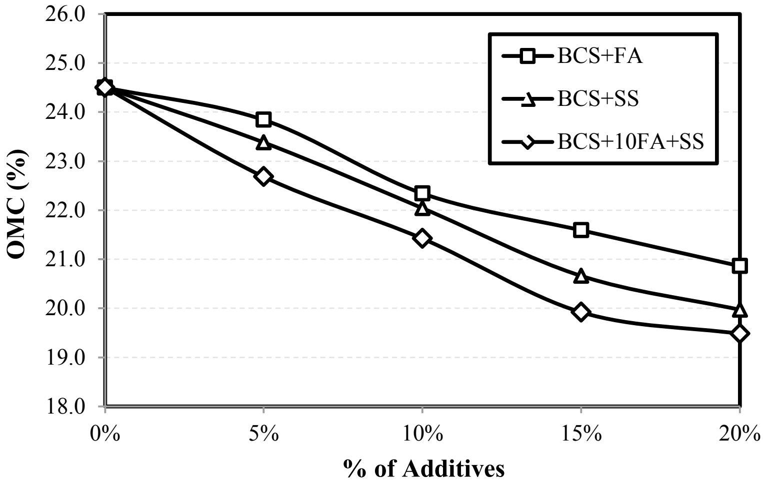

Fig. 6 Comparative effect of FA, SS, and FA + SS on OMC

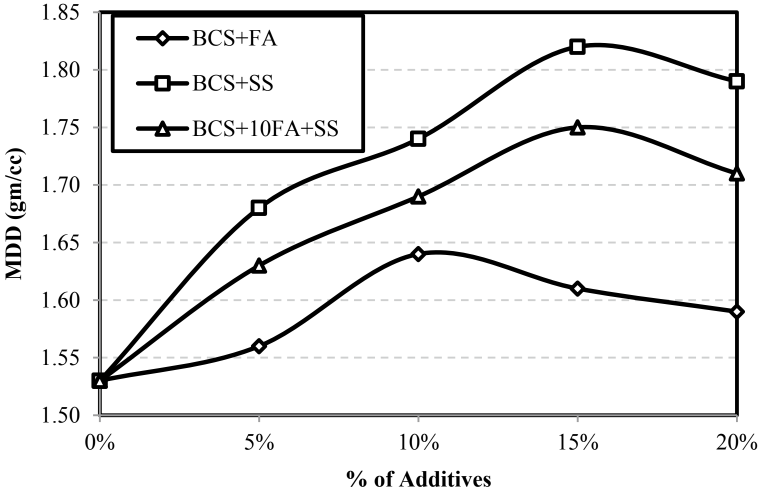

Fig. 7 Comparative effect of FA, SS, and FA + SS on MDD

the results, it was evident that as the percentage of clay from the BC soil reduces the amount of moisture required also reduces.

Figure 7 shows the variation of maximum dry density for different proportions of BCS-FA-SS mixtures. It has been observed that the maximum dry density of all mixes increased with additive content. From the results, it was observed that with an increase in fly ash content, the MDD of BCS-FA mixes increases and after 10% FA content MDD value decreases. The rise in density is more significant at lower percentages of FA. However, increasing trend of MDD value was observed in the case of BCS + SS and BCS + 10FA + SS combination. The MDD was increased by 1.96% for 5% FA, 3.26% for 5% SS, and 6.53% for combination of 10% FA + 5% SS. Finally, at 10% FA and 90% of BCS gives MDD value of 1.64 gm/cc. In case of 85% BCS with 15% SS gives 18.95% addition of MDD, i.e., 1.82 gm/cc. Also in case of 75% BCS + 10% FA + 15% SS MDD value observed as 1.75 gm/cc. In BC soil, the clay content reduced due to the addition of steel slag resulting in increasing the coarser particles and increase in the maximum dry density for BCS + SS mixes.

The UCS tests were carried out on the cylindrical specimens BCS + FA + SS mixes (38 mm × 76 mm) to for different mix proportions without curing. The test is undrained based on the assumption that there is no moisture loss during the test. The mixture was compacted in three layers to achieve the required MDD of the mix. Increase in the fly ash and steel slag content increased the cohesive and frictional property of black cotton soil. The 75% BC soil + 10% fly ash + 15% steel slag content found the optimum combination among all combinations. The UCS value black cotton soil sample was 0.291 kg/cm^2 and it was increased up to 0.552 kg/cm^2 after addition of FA + SS as shown in Fig. 8.

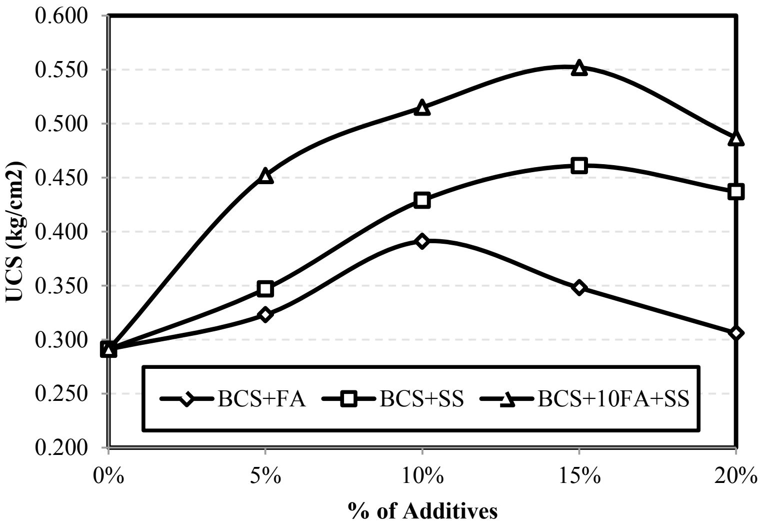

Fig. 8 Comparative effect of FA, SS, and FA + SS on UCS

4 Conclusions

From the Atterberg's limit tests, it was observed that 5–20% addition of FA and SS in the black cotton soil considerably reduces the volume change potential of the black cotton soil. Addition of FA and SS in black cotton soil also helps in reducing its swelling and shrinkage characteristics. From the combined observation of compaction tests and UCS test results, it can be noted that addition 10FA and 20% SS in black cotton soil will provide a mix having higher shear strength.

According to the test results, it appears that fly ash and steel slag can be used as an admixture in the BC soil to improve its properties. Utilization of fly ash and steel slag also has the advantages of reusing an industrial waste by-product without adversely affecting the environment and solve the dumping and disposal problems of industrial waste.

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Comparison of Compaction Characteristics of Non-conventional Conventional Stabilizers

M. S. Gayathri and Sujit Kumar Pal

Abstract Roads play an important role in connecting traffic all over the world. It is crucial as it helps to grow and develop a nation. In past decades, ingress of water in the rainy season weakens the road's soil base. From the beginning of road stabilization, water was one of the main problems. So now researchers have come with an idea of water-proofing the soil and recently, they introduced nano-chemicals. Previous literature surveys show that the presence of only small amount of nano-material in the soil could influence significantly the physical and chemical behaviour of soil. The main objectives of the study are to determine the compaction and shear strength properties of various soil combinations prepared by mixing the nano-chemical solutions, to obtain the optimum dosages of nano-chemical agent corresponding to higher MDD and shear strength values. The nano-chemicals Terrasil and Zycobond are collected from Gujarat, India, for the stabilization studies. Experimental programme is carried out on the clayey-silt soil treated with different dosages of Terrasil, Zycobond and cement. Results obtained are compared and studied. UCS strength is found to be increasing with increase in dosage of stabilizer and curing period. The optimum dosage of Terrasil and Zycobond is obtained as 0.8 kg/m^3 by weight of soil and also the strength maximum for 7% cement content. This improvement may be possible due to the reaction of the chemical with the soil particles and as a result Terrasil waterproofs the surface and Zycobond improves strength.

Keywords Terrasil · Zycobond · Compaction · Shear strength

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1 Introduction

Roads play an important role in connecting traffic all over the world. It is crucial as it helps to grow and develop a nation. From the beginning of road stabilization, water was one of the main problems. Soils are very hard in dry state, while it loses strength completely in wet state. The water ingress into soil in rainy season weakens the road's soil base, especially in rural roads. Long-term performance of pavement structures depends on the stability of the underlying soils. Application of different stabilization materials to regular soil delivers better results, especially in case of road stabilization. The engineering design of these road facilities should be such that each layer in the pavement has the minimum specified structural quality. These layers must be able to resist excessive permanent deformation, resist shear and avoid excessive deflection that may result in fatigue cracking in overlaying layers. The local soil available at the site does not always meet the minimum requirement. Stabilization is usual practice for these problematic soils as replacing part or whole foundation material with good quality material is un-economical. In India, the soil stabilization began due to the shortage of petroleum and aggregates; it became necessary for the engineers to look at means to improve soil other than replacing the poor soil at the building site. Due to depletion of the sources of aggregates, cost of the road construction increases. Therefore, it is necessary to use alternative material for construction which would reduce the overall cost of pavement construction. Stabilization of the sub-grade soil can lead to reduce the thickness of the pavement layers and it works out to be economical. In India, the soil stabilization began due to the shortage of petroleum and aggregates; it became necessary for the engineers to look at means to improve soil other than replacing the poor soil at the building site. Nowadays, various additives are also being used for stabilization of various types of soils, which often accomplished by physical or chemical stabilization. In chemical stabilization process, it depends mainly on chemical reactions between stabilizer and soil minerals to achieve the desired effect. In case of pavements, even after traditional stabilization methods, it fails so fast, which increases the maintenance cost. So, researchers are now in search of materials which are environmental friendly, cost-effective and long lasting. Recently, chemical stabilization has become very popular and various researches are going on nano-chemicals.

The idea of nano-technology was suggested by Richard Feynman for the first time in 1959. After that, this technology is developed in all branch of sciences. Due to a variety of subjects, and macroscopic view of researchers and engineers to the soil, very little investigation has been performed in the field of nano-technology's applications in geotechnical engineering. The main strategy of nano-technology in geotechnical engineering is the improvement of soil parameters with application of nano-chemicals. The presence of only small amount of nano-material in the soil could influence significantly the physical and chemical behaviour of soil due to a very high specific surface area of nano-materials, surface charges and their morphologic properties [1]. A number of researchers have worked in developing different methods of soil stabilization based on nano-chemicals, which are practical and economical.

Patel et al. [4] utilized a concoction named Terrasil as stabilizer and it was utilized for altered measurement, i.e. 0.041% by dry aggregate weight of soil. Test outcome demonstrates that designing properties got modified and CBR on stabilized clayey samples increased considerably. Experimental programmes were carried out by Johnson and Rangaswamy [7] on both clay-treated and cement-treated clay treated with different dosages of Terrasil. The optimum dosage of Terrasil was obtained as 0.07% by weight of soil and the strength maximum for 4% cement content. The UCC strength of soil mixed with optimum dosage of Terrasil chemical is improved about 441%. The California bearing ratio (CBR) strength of soil mixed with optimum dosage of 0.07% Terrasil chemical is improved about 6 times. Mishra et al. [3] conducted study on black cotton soil and changes in various soil properties such as liquid limit, plastic limit, CBR were studied. Complete analysis of the improvement of soil properties and its stabilization using Terrasil is made. Roshni and Jeyapriya [8] investigated chemical stabilization of black cotton soil to improve the sub-grade. The maximum CBR value is obtained at a combination of 0.07% Terrasil and 3% cement. CBR percentage was increased by 3.6 times when compared to soil without stabilizer. SEM and EDAX analysis are also made to know the morphology and chemical composition in the un-stabilized and stabilized soil sample. Aderinola and Nnochiri [5] carried out preliminary tests on six natural soil samples. For soil sample 1, the unsoaked CBR values increased from 8.4% at 0% to optimum value of 30.3% at 12% Terrasil solution, while for sample 2, it increased from 6.2 to 32.0% at 12%. It was therefore concluded that the Terrasil solution serves as a cheap and effective stabilizing agent for poor soil. Giridhar et al. [2] conducted experiments to study the effect of Zycobond on liquid limit, plastic limit, CBR and shear strength of expansive soil at different percentages, such as 1.0, 2.0, 3.0, 4.0 and 5.0%. It is observed that the CBR value is decreased and shear strength is increased significantly with the addition of Zycobond. Rohith et al. [9] carried out economic analysis on both flexible and rigid pavements. Raghavendra et al. [10] used Terrasil and Zycobond for soil stabilization. They performed studies indicated that the application of nano-material produces more effective results, and this is significant in stabilization of weak soils.

The main objective of this work is to conduct laboratory experiments to understand use of organosilane-based nano-materials and implement the technology on rural road construction. In this paper, the performance of nano-chemical like Terrasil and Zycobond is investigated for stabilization of sub-grade soil. The study gives a comparison of compaction characteristics between cement, Terrasil, Zycobond and both Terrasil and Zycobond.

2 Materials

The following materials are used in the experimental program.

2.1 Clayey-Silt Soil

The soil is sample collected from the local agency of Agartala city in Tripura, India. It is having a solid specific gravity of 2.68 when the soil is tested as per IS-2720 (Part 3)-1980. Hydometer analysis is conducted as per IS: 2720 (Part 4)-1985 and concluded that the sample contained around 31% clay particles and 61% silt particles. The liquid limit as per IS: 2720 (Part 5)-1985 is 34% while the plastic limit of the soil observed 20%. The plasticity index of the soil sample is found to be 14%. As per USCS soil classification system, the soil is classified as clayey-silt soil of intermediate plasticity. Compaction test as per IS: 2720 (part 7 or 8) showed that the maximum dry density (MDD) is 15.8 kN/m^3 and the optimum moisture content (OMC) is 23%.

2.2 Cement

Ordinary portland cement (OPC) of Grade 43 is used for stabilization of the selected soil.

2.3 Terrasil

Terrasil is defined as an organosilane compound which reacts with soil particles and form hydrophobic (oily) layers on the surface of the soil and clay particles. It is a commercially available chemical stabilizer which is used in the present investigation. It is manufactured by Zydex Industries. Table 1 gives composition of Terrasil chemical. It is available in concentrated liquid form and is to be mixed with water in specified proportion before mixing with the soil. Table 2 gives properties of Terrasil chemical. Terrasil is nano-technology based 100% organosilane, water soluble, ultraviolet and heat stable, reactive soil modifier to waterproof soil sub-grade. It reacts with water loving silanol groups of sand, silt and clay, and aggregates to convert it to highly stable water repellent alkyl siloxane bonds and forms a breathable in situ membrane. It can be applied to almost all types of soil. Terrasil keeps the pores open to allow vapours to escape while preventing water to come in. Nano-chemicals can be identified as environmental friendly, since they conserve limiting resources, like

Table 1 Composition of Terrasil chemical

Chemical compound	Value in range (%)
Hydroxyalkyl-alkoxy-alkylsil	65.0–70.0
Benzyl alcohol	25.0–27.0
Ethylene glycol	3.0–5.0

Table 2 Properties of Terrasil chemical

Property	Description
Appearance	Pale yellow liquid
Density	1.01 g/ml
Solubility	Forms water clear solution
Flash point	>80 °C
Freezing point	5 °C

Table 3 Properties of Zycobond chemical

Parameter	Value
Colour	Milky white
Odour	No
Flash point	Above 100 JC
Explosion hazard	No
Ignition temperature	Above 200 JC
Solubility in water	Dispersible
pH value	5.0–6.0

aggregates and bitumen. They also allow the use of in situ soils minimizing use of fuel for transporting good soils over long distance [6].

2.4 Zycobond

Zycobond is chemical stabilizer which is available commercially. It is manufactured by Zydex Industries. It is acrylic co-polymer dispersion with long life purposed for bonding of soil particles and permanent erosion control and dust suppression on the road and environment. It offers water resistance, strong bonding and imparts flexibility to the soil surface. It is mixed with Terrasil solution and sprayed on compacted soils. Terrasil mixed with Zycobond gives excellent bonding of road construction materials. It enhances the quality of soil layer, controls soil disintegration, quick drying of soil layers/earth road after downpours and thus it helps in reducing maintenance cost. It imparts water proofing and resists water ingress through the unpaved areas, like shoulders and slopes. Characteristics of the chemical stabilizer used in this work are shown in Table 3.

3 Methodology

The soil is collected from local agency in Agartala. Preliminary tests such as basic tests including, Atterberg's limits standard Proctor density test, grain-size distribution are performed on the soil sample for the determination of the index properties and then dosages of different additives are decided. In these investigation compaction characteristics of soil for different compositions of cement, Terrasil, Zycobond, both Terrasil and Zycobond are compared.

The light compaction test is conducted for soil at different percentages of cement. The dosage of cement is varied between 3 and 11% with a difference of 2%. For testing, cement is mixed with soil and test is conducted as per standard procedure. For conducting compaction test in case of Terrasil, initially, Terrasil is mixed with water in required dosage to prepare solution. Dosage of Terrasil is varied between 0.2 and 1.4 kg/m^3 with a difference of 0.2 kg/m^3. Same procedure and dosage are followed for Zycobond also.

Similarly, for Terrasil and Zycobond solution, Terrasil is mixed first and then Zycobond is added. As these chemicals are in concentrated liquid form whose specific gravity is almost equal to water, no more calculations needed while mixing this solution to prepare sample.

Similarly, an effort to find out the effect of combination of the additives are also analysed in the study. From the tests, it is found out that 0.8 kg/m^3 is the optimum proportion for both Terrasil and Zycobond, respectively, for the selected soil. Combination percentages are selected on both sides of the optimum additive value. The selection methodology is explained in Table 4. The idea is to keep the total amount of additive a constant. Combination percentages are fixed in order to maintain a total additive amount of 1.60 kg/m^3 for the soil.

UCS test samples are prepared in 37 mm diameter and 100 mm height moulds. The mix is composed of soil, cement or nano-materials and is compacted with the same energy for light compaction to achieve maximum possible laboratory density. The sample is cured for 7 and 28 days, and then surface treatment is done with nano-material solution, and is further kept for drying for 3–4 days. Surface treatment is made on top of compacted soil in mould to simulate top layer water-proofing for creating impervious surface. Surface treatment will help to achieve impervious soil surface which will help in preventing the ingress of water. In case of UCS moulds,

Table 4 All mix program for various soil tests

Dosage cases		
Terrasil or Zycobond (kg/m^3)	Terrasil + Zycobond (kg/m^3)	Cement (%)
0.2	1.6T + 0Z	3
0.4	1.2T + 0.4Z	5
0.6	0.8T + 0.8Z	7
0.8	0.4T + 1.2Z	9
1.0	0T + 1.6Z	11
1.2		
1.4		

the water-proofing spray is applied on top, bottom and sides. Table 4 shows various combinations of cement, Terrasil, Zycobond, and Terrasil and Zycobond for the test.

4 Results and Discussions

This paper aimed to investigate performance of Terrasil and Zycobond. Various compaction tests are performed on the soil and the compaction results show that the MDD value increases with addition of any stabilizer but the behaviour of soil varies according to the type of soil and the amount of stabilizer used. It is also observed that after optimum moisture value the MDD value decreased.

Table 5 shows the maximum dry density (MDD) and optimum moisture content (OMC) values of maximum of different combinations using different stabilizers. The combination of the additives (i.e. Terrasil and Zycobond) are also analysed in the study. From the tests, it is found out that 0.8 kg/m^3 is the optimum proportion for both Terrasil and Zycobond, respectively. Combination percentages are selected on both sides of the optimum additive value. And it is found that maximum value is obtained for the case of combination of nano-chemicals at 1:1 mix ratio.

The MDD values for all samples generally increased and the OMC generally reduced. The maximum dry density and optimum moisture content are observed to be 17.4 kN/m^3 and 18.5% at dosage of 0.8 kg/m^3 for Terrasil. Similarly, maximum dry density and optimum moisture content for Zycobond are observed to be 17.3 kN/m^3 and 17.5% at dosage of 0.8 kg/m^3. Similarly, in case of cement, the MDD value is obtained is 17 kN/m^3 and OMC is 18% at 7% cement variation. For Terrasil and Zycobond combination, the MDD increased from 15.8 kN/m^3 for natural soil to 18 kN/m^3 at a mix proportion of 0.8 kg/m^{3}T + 0.8 kg/m^{3}Z (1:1 mix). The maximum value of MDD is obtained for 1:1 mix proportion of Terrasil and Zycobond of all the mixes. Increase in MDD is a good indication of improvement in soil property, whereas a reduction in OMC enhances the workability of a good soil.

For the soil used in the present study, it is observed from Fig. 1 that there is an increase of almost 10% in the MDD value when treated with Terrasil and 9.5% in

Table 5 Maximum OMC and MDD values of different combinations

	MDD (kN/m^3)	Percent increase in MDD (%)	OMC (%)	Percent decrease in OMC (%)
Natural soil	15.8	–	23.0	–
Terrasil (0.8 kg/m^3)	17.4	10.0	18.5	20.0
Zycobond (0.8 kg/m^3)	17.3	9.5	17.5	24.0
Terrasil and Zycobond (0.8T + 0.8Z)	18.0	14.0	15.0	35.0
Cement (7%)	17.0	8.0	18.0	22.0

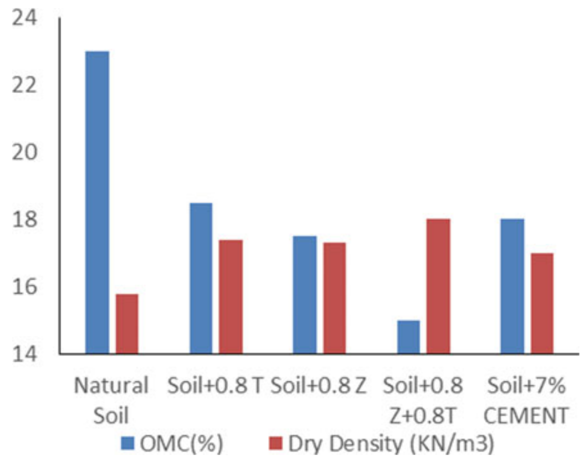

Fig. 1 Variation of OMC and MDD

case of Zycobond and also an increase of around 14% in the MDD value is observed when it is stabilized with both Terrasil and Zycobond. A decrease of around 35% in the OMC value is found when it is stabilized with both Terrasil and Zycobond and at the same time, a decrease of almost 20% in the OMC value when treated with Terrasil and a decrease of 24% in case of Zycobond. In case of cement, there is an increase of 8% in MDD and a decrease of 22% in case of OMC.

From Table 6, it is observed that in case of cement, the increase in compressive strength is considerable from parent soil to soil treated with 7% cement. The compressive strength is found to be 135.36 kN/m^2. For soil treated with 0.8 kg/m^3 of Terrasil, the compressive strength from test is found to maximum and it is 155.73 kN/m^2. As the percentage of chemical increases, compressive strength value increases up to a peak value and then decreases. The results are shown in Table 6. Similarly, in case of Zycobond, the compressive strength value increased up to a value of 150.99 kN/m^2 which maximum at a composition of 0.8 kg/m^3. And in case of combination of Terrasil and Zycobond, maximum compressive strength is obtained as 241.45 kN/m^2. Out of all combinations soil-Terrasil–Zycobond mix gives maximum unconfined compressive strength at a combination of 0.8 T + 0.8 Z (1:1 mix of both nano-chemicals). There is an increment in unconfined compressive strength of 7 days curing about

Table 6 UCS test results for 7 days of curing

	Cohesion (kN/m^2)	Unconfined compressive strength (kN/m^2)	% Increase in compressive strength (%)
Native soil	21.27	42.54	–
7% C	67.68	135.36	218.0
0.8T	77.86	155.73	266.0
0.8Z	75.49	150.99	254.0
0.8T +0.8Z	120.72	241.45	467.0

Table 7 UCS test results for 28 days of curing

	Cohesion (kN/m^2)	Unconfined compressive strength (kN/m^2)	% Increase in compressive strength (%)
Native soil	21.27	42.54	–
7% C	117.19	234.39	450.0
0.8T	126.67	253.35	495.0
0.8Z	123.69	247.38	481.0
0.8T + 0.8Z	170.25	340.5	700.0

218, 266, 254, and 467% for cement, Terrasil, Zycobond, and combination of both, respectively.

From Table 7, it is observed that in case of cement, the increase in compressive strength is considerable from parent soil to soil treated with 7% cement. The compressive strength is found to be 234.39 kN/m^2. For soil treated with 0.8 kg/m^3 of Terrasil, the compressive strength from test is found to maximum and it is 253.35 kN/m^2.

As the percentage of chemical increases, compressive strength value increases up to a peak value and then decreases. Similarly, in case of Zycobond, the compressive strength value increased up to a value of 247.38 kN/m^2 which maximum at a composition of 0.8 kg/m^3. And from combination of Terrasil and Zycobond, maximum compressive strength is obtained as 340.5 kN/m^2. There is an increment in unconfined compressive strength of 28 days curing about 450, 495, 481 and 700% for cement, Terrasil, Zycobond, and a combination of both, respectively.

5 Conclusions

This paper is aimed to investigate compaction characteristics of nano-chemical stabilizers. The overall conclusions about applicability of nano-chemical soil stabilizers based the results and observations in road construction are given below:

- The literature reviews and laboratory test results indicated that the soil type greatly influences the performance of these, i.e. Terrasil and Zycobond nano-chemical stabilizer.
- The behaviour of soil varies according to the amount of stabilizer used.
- It is observed that for both Terrasil and Zycobond, MDD value is highest at 0.8 kg/m^3 dosage.
- While mixing both Terrasil and Zycobond together, the maximum value of MDD is obtained at 1:1 mix ratio which is found to be the maximum of all mixes.
- There is an increase of 14% in the MDD value and a decrease of around 35% in the OMC when it is stabilized with both Terrasil and Zycobond together.
- An increase of 10 and 9.5% is observed in the MDD value when treated with Terrasil and Zycobond, respectively.

- A decrease of almost 20 and 24% is found in the OMC value is when treated with Terrasil and Zycobond, respectively. There is a considerable increase in compressive strength of soil when treated with nano-chemical.
- It is also observed that unconfined compressive strength increases with increase in dosage of different stabilizers and curing period. But, after an optimum dosage, it is decreasing.
- With increasing the composition of cement, the maximum unconfined compressive strength value of treated soil for 7 days curing is obtained as 135.36 kN/m^3 at a dosage of 7% and the percentage increase in strength of treated soil is 218% and for 28 days curing is observed as 234.39 kN/m^3 at a dosage of 0.8 kg/m^3 and the percentage increase is 450%.
- The maximum unconfined compressive strength value of treated soil with increasing the composition of Terrasil for 7 days curing is obtained as 155.73 kN/m^3 at a dosage of 0.8 kg/m^3 and the percentage increase in strength of treated soil is 266% and for 28 days curing is observed as 253.35 kN/m^3 at a dosage of 0.8 kg/m^3 and the percentage increase is 495%.
- In case of Zycobond, the maximum unconfined compressive strength value for 7 days curing is obtained as 150.99 kN/m^3 at a dosage of 0.8 kg/m^3 and the percentage increase in strength of treated soil is 254% and for 28 days curing is observed as 247.38 kN/m^3 at a dosage of 0.8 kg/m^3 and the percentage increase is 481%.
- When soil is treated with combination of Terrasil and Zycobond, the maximum unconfined compressive strength value for 7 days curing is obtained as 241.45 kN/m^3 at a combination of 0.8 T + 0.8 Z and the percentage increase in strength of treated soil is 467% and for 28 days curing is observed as 340.5 kN/m^3 at a dosage of 0.8 kg/m^3 and the percentage increase is 700%.

Interestingly, the stabilizers Terrasil and Zycobond found to be of high potential in improving soil behaviours. This study is likely to provide valuable inputs to the engineers.

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Effects of Delay Time on Compaction and Strength Properties of Stabilized Granular Soil

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Abstract Weak and marginal soils are conventionally stabilized with chemical stabilizers like lime and cement. During construction, sometimes inevitable delays occur between mixing of stabilizer with the soil and compaction, which have adverse effects on the geoengineering properties of the stabilized soil structures. The present study emphasizes the effects of delay time on compaction and strength properties of a granular soil stabilized with three different stabilizers, i.e. lime, cement and slag-based geopolymers. In this study, these three different stabilizers were added with soil in different proportions varying from 0 to 15% of the dry soil, and the effect of time lag was studied individually. The optimum moisture contents (OMC) and maximum dry densities (MDD) of these mixtures were determined after a time lag of 0, 3, 6, 12, 24, 48, 72 and 168 h. Further, cylindrical specimens of size 36 mm diameter and 72 mm length were prepared for all these mixes compacted to MDD at OMC taking into the effects of delay. Before conducting the UCS test, these specimens were cured at an average temperature of 30 °C for 0, 7 and 28 days in closed secure environment for assuring the prevention of moisture loss while curing. It was observed that the delay time significantly affects the OMC and MDD of mixes, and it is more noticeable in case of cement and geopolymer binders than the lime. Similarly, delay time affects the strength of cement and geopolymer stabilized mixes more adversely than lime stabilized mixes.

Keywords Soil stabilization · Delay time · Compaction characteristics · Strength properties

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1 Introduction

When cement is added to soil, the saturated sample compacted and cured results into a hard durable soil cement mixture. When the mixture of soil and cement is properly compacted at the time of construction, it becomes resistant to deterioration due to moisture and weather and also its deformation does not happen due to heavy traffic loads. Baghdadi et al. [1] found that cement kiln dust (CKD) can significantly decrease the optimum moisture content and increase the maximum dry density of pure kaolinite when the CKD content is less than 50%. Miller and Azad [2] observed an increase in the optimum moisture content and a decrease in the maximum dry density when CKD was added into three types of soil with different high, medium and low plasticity and concluded that the effect of CKD on optimum moisture content and maximum dry density is obviously a function of soil and CKD type as well as compaction method. Soil stabilization is a well-established discipline within geotechnical engineering. Cement is preferred for lowly cohesive (sandy) soils but it loses effectiveness for highly plastic soils. Cement is the most commonly used stabilizer, and its popularity is due to its ability to gain strength quickly and to obtain desirable mechanical properties with relatively low amounts of stabilizer.

Addition of lime to soils improves the workability and increases the strength of the mixtures, although strength gains are not as great as those due to addition of cement. For clayey soils, lime is generally used as a stabilizing agent because it flocculates the clay and increases the plasticity. Cementation ultimately results in slowing the Pozzolanic reaction. Clay is flocculated by cement due to free lime content. When both cement and lime are added to the soil, the lime causes ease mixing, and the cement gives strength and durability. Currently, chemical stabilization of soils is the most common method. Stabilizers like cement and lime are used. But due to more usage of cement, it has given rise to environmental issues like dust generation and CO_2 emission. Geopolymer is a developing material as an alternative to cement. Aside from the environmental problems, geopolymer stabilized soils have shown advanced properties to satisfy the microstructural and mechanical properties. When compared to compressive strengths of OPC specimen, lightweight GGBS-based geopolymer stabilized specimen has given 200–350% more strength.

Time elapses between mixing and compaction that vary depending upon the construction method employed. During construction, a time lag may elapse between soil–lime mixing and compaction due to hitches or technical breaks for logistic reasons. In reviewing literature, conflicting recommendations and opinions can be found concerning the influence of delayed compaction. Studies developed by the Louisiana Department of Transport in the early sixties pointed out that a delay longer than 48 h results in a lower strength of the soil–lime mixtures. Mitchell and Hooper [3] found that a 24 h delayed compaction reduces the dry unit weight and the long-term strength, whereas the swelling was found to increase.

Table 1 Physical properties of Portland slag cement

Physical property	Value
Specific gravity	3.0
Consistency of cement	33.5%
Initial setting time	2 h 38 min
Final setting time	9 h 45 min

Table 2 Physical properties of river sand

Physical property	Value
Specific gravity	2.64
Uniformity coefficient (Cu)	3.0
Coefficient of gradation (Cc)	0.925
Maximum dry unit weight (MDD)	17 kN/m^3
Optimum moisture content (OMC)	15.1%

2 Materials

Portland slag cement (PSC) of Konark brand is used as a stabilizing agent. The properties of PSC are given in Table 1. Lime (calcium hydroxide) is of Loba Chemie brand used as another chemical agent with 95% purity and placed in sealed container. River sand—river sand from Koel River is used as a base material. The properties of river sand are given in Table 2. Ground granulated blast furnace slag—slag used for the study is blast furnace slag obtained from the Rourkela steel plants. Slag was 10 mm in size initially which is grounded in a ball mill and sieved through 75 mm sieve before mixing to prepare samples. GGBS used is milky white in colour and has specific gravity of 2.82. Sodium hydroxide—laboratory grade sodium hydroxide pellets with 98% purity were used to prepare solution. Figure below shows the image of NaOH pellets.

3 Methodology

Using different percentages of cement and lime (i.e. 2.5, 5, 7.5, 10 and 15%) mixed with sand, and 5, 10 and 15% of slag-based geopolymer using sodium hydroxide (NaOH) as an activator by dry weight of the soil. The maximum dry density and optimum moisture content are determined by using light and heavy compaction test, as per IS-2720(7, 8). The delayed compaction (3, 6, 12, 24 and 48 h, 3 days and 7 days) for the samples with different percentages of cement, lime and slag-based geopolymer is studied. Using the mix, samples are prepared and cured for curing periods of 0, 7 and 14 days at a constant temperature of 30 °C, and UCS test is conducted as per IS-2720 (10).

4 Result and Discussion

4.1 Compaction Characteristics

With increasing lime content, the maximum dry density (MDD) is increased and the corresponding optimum moisture content (OMC) is decreased for both light compaction and heavy compaction test. In case of sand–lime mixture, the MDD increased from 18 to 19.2 kN/m^3 and OMC decreased from 17.2 to 13.1% for light compaction as shown in Figs. 1 and 2. For heavy compaction, the MDD increased from 19.1 to 20.3 kN/m^3 and OMC decreased from 16 to 11.7% as shown in Figs. 3 and 4.

It has been observed that the MDD increased by 6.29% and the OMC decreased by 26.9% when lime content increased from 2.5 to 15%. In the same way, for sand–cement mix in light compaction, from Fig. 5 it is observed that the MDD increased by 10.49% and OMC decreased by 19.35% when the cement content is increased from 2.5 to 15%. Similarly, for heavy compaction, MDD increased by 8.8% and OMC decreased by 20% with the increase of cement content from 2.5 to 15% as shown in Fig. 6. In the same way, slag-based geopolymer MDD increased from 19.92 to 21.26 kN/m^3 and OMC decreased from 10.81 to 8.85% for heavy compaction, and in case of light compaction MDD increased from 19.27 to 20.69 kN/m^3 and OMC decreased from 12.45 to 9.86% as shown in Figs. 7 and 8.

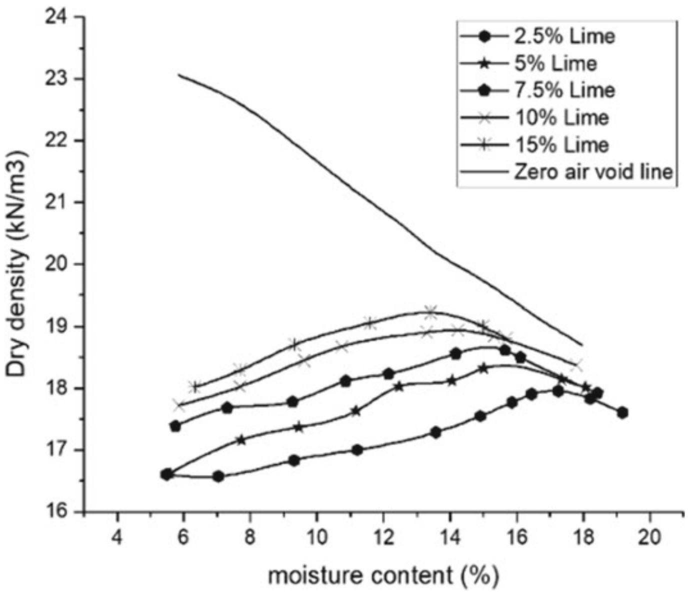

Fig. 1 Light compaction curves of sand–lime mixes

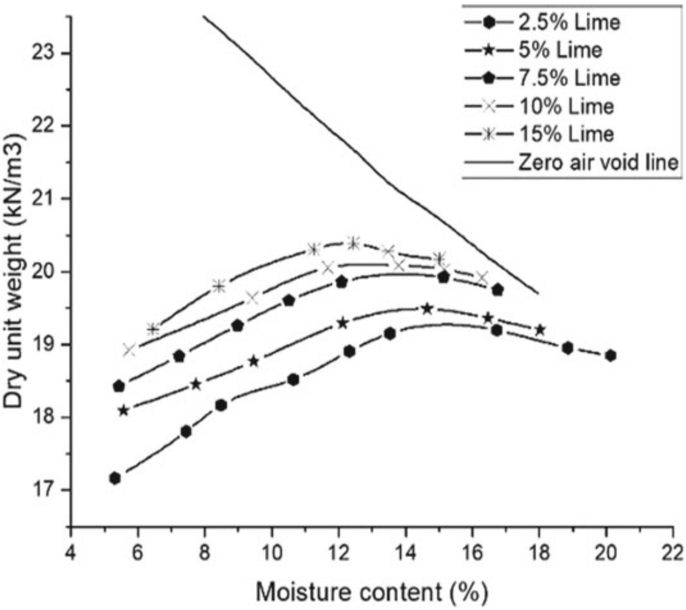

Fig. 2 Heavy compaction curves of sand–lime mixes

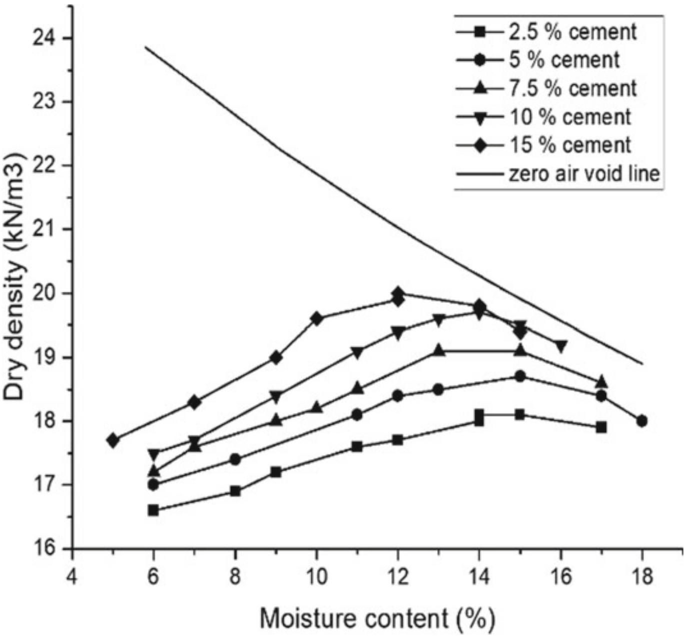

Fig. 3 Light compaction curves of sand–cement mixes

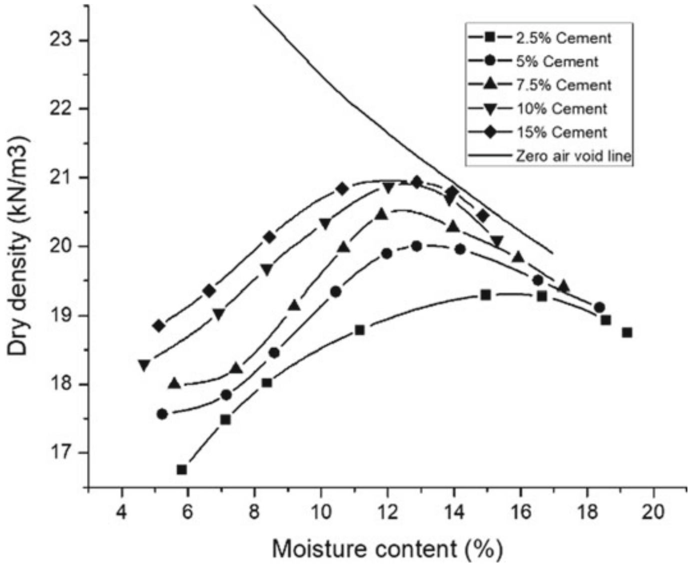

Fig. 4 Heavy compaction curves of sand–cement mixes

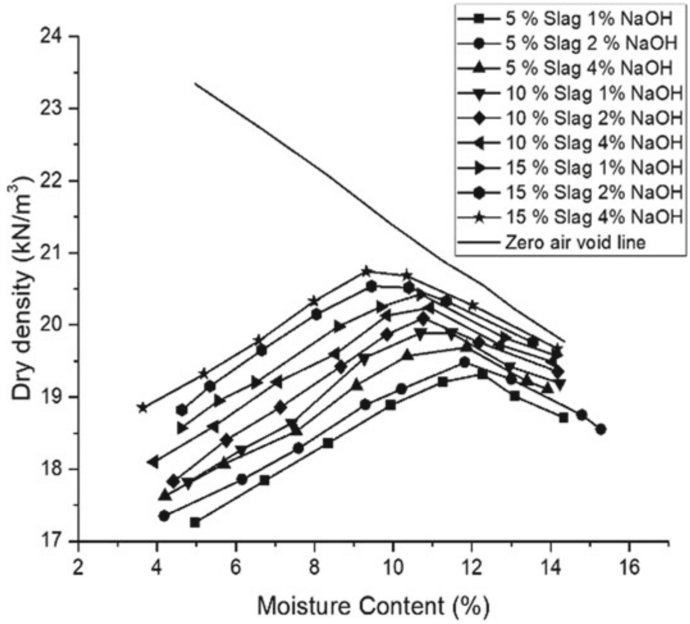

Fig. 5 Light compaction curves of sand and slag-based geopolymer mixes

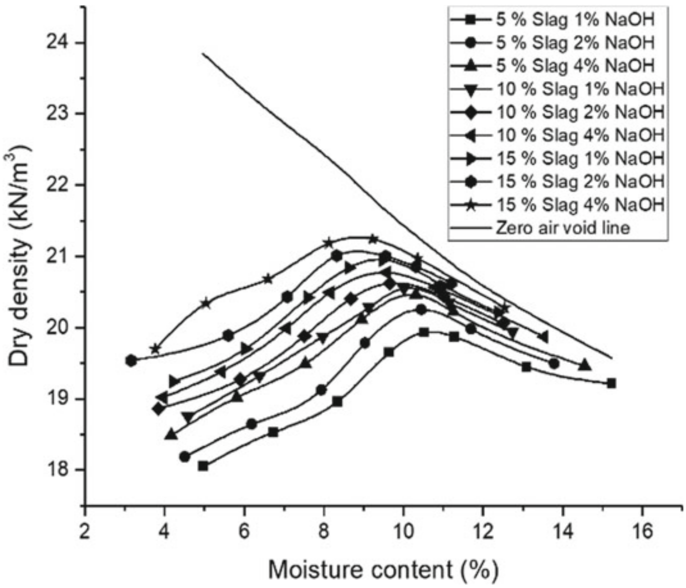

Fig. 6 Heavy compaction curves of sand and slag-based geopolymer mixes

4.2 *Delayed Compaction*

Sand–cement mixture, sand–lime mixture and slag-based geopolymer samples are mixed at obtained optimum moisture content (OMC) from the compaction characteristics graphs. The samples are compacted by light compaction and heavy compaction test after a delay period of 0, 3, 6, 12, 24, 48, 72 and 168 h. For light compaction of sand–lime mix with increasing delay time, the dry unit weight is reducing. From Fig. 7, it is observed that for 48 h delay period and 2.5% lime content the MDD is 14.1 kN/m^3 and for 48 h delay period and 15% lime content the same is 17.3 kN/m^3, i.e. increase in MDD by 22.69%.

For heavy compaction of sand–lime mix from Fig. 8, it can be observed that for 168 h delay period at 5% lime content MDD is 17 kN/m^3 and 15% lime content has 18.2 kN/m^3. It is also observed that for the higher percentage of lime or cement content, the decrease in MDD is rapid initially with increasing delay time and then it stabilizes, but for the lower percentage of lime or cement content the slope of the curve is lesser from the beginning. Compared to lime for sand–cement mix 2.5% cement content, the MDD is 15 kN/m^3 and for 48 h delay period and 15% cement content the same is 18.2 kN/m^3, i.e. increase in MDD by 21.33% it can be seen in Fig. 9. For heavy compaction of sand–cement mixes from Fig. 10, it can be concluded that for 168 h delay period at 5% cement content MDD is 16.4 kN/m^3, and at the same period of time 15% cement has 17.8 kN/m^3.

In case of slag-based geopolymer mixes for 5% slag and 1% NaOH for a delay period of 72 h, MDD is 14.5 kN/m^3, and for 15% slag 1% NaOH in case of same

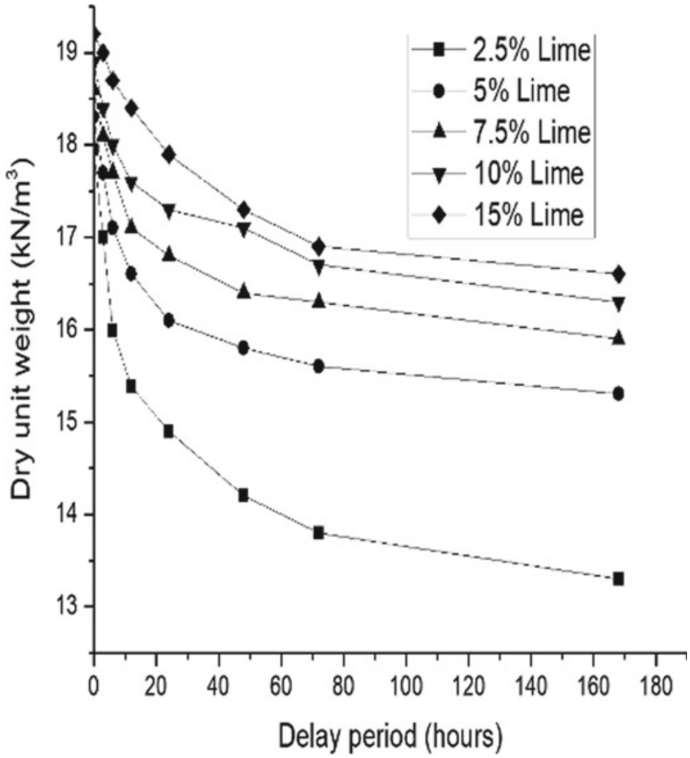

Fig. 7 Variation of dry unit weight of sand–lime mixes with delay time (light compaction)

delay period MDD is 16.4 kN/m^3 as shown in Fig. 11. For heavy compaction as shown in Fig. 12, 1% NaOH for a delay period of 24 h MDD is 17.8 kN/m^3 and for 15% slag 1% NaOH of the same delay period MDD has the value 19.7 kN/m^3, i.e. increase in MDD by 10.67%.

4.3 Unconfined Compression Strength

The unconfined compressive strength for sand–lime mixture is increased with increase in percentage and decrease in the delay period. 10% lime for 7 day curing period 0 h delay, the unconfined compressive strength is 234.5 kPa and for 7 days delay period 102.4 kPa, i.e. the compressive strength is decreased by 56.33% as shown in Fig. 13.

The unconfined compressive strength for sand–cement mixture is increased with increase in percentage of slag and decreases with the delay period same like sand–lime mixture. 10% cement for 7 day curing period 0 h delay the unconfined compressive strength is 369.6 kPa and for 7 days delay period 214.2 kPa, i.e. the compressive

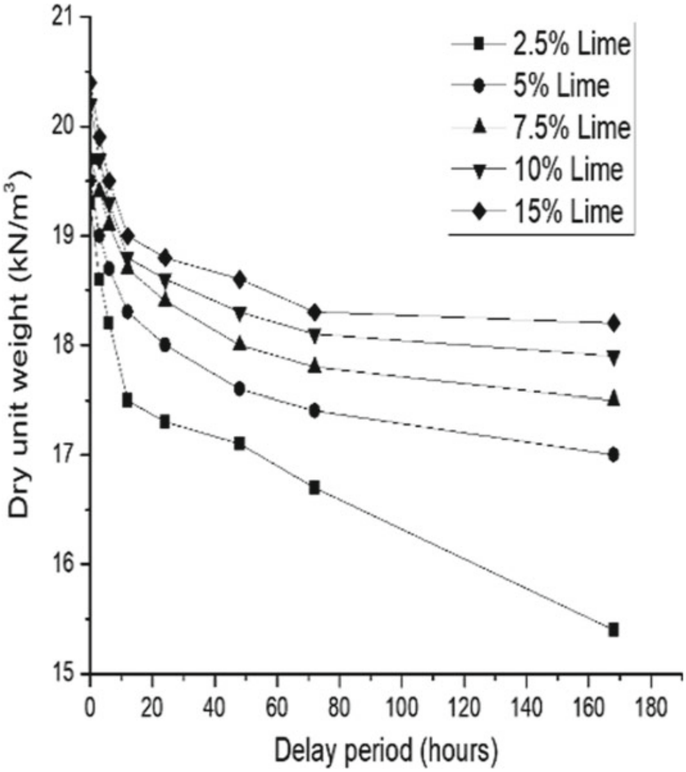

Fig. 8 Variation of dry unit weight of sand–lime mixes with delay time (heavy compaction)

strength is decreased by 42.04% as shown in Fig. 14. The unconfined compressive strength for slag-based geopolymer mixture is increased with increase in percentage of slag and NaOH activator and decrease in the delay period. 10% slag 1% NaOH activator 7 day curing period with 0 h delay the unconfined compressive strength value is 261 kPa and 48 h delay period 152 kPa and for 10% slag 4% NaOH solution the 7 day curing period with 3 h delay 336 kPa and 48 h delay 265 kPa as shown in Fig. 15.

5 Conclusions

The effects of delay time on compaction as well as strength properties of a granular soil stabilized with three different stabilizers such as lime, cement and slag-based geopolymers are studied. Stabilizers are added with soil in different proportions. The compaction characteristics (i.e. MDD and OMC) of these mixtures were determined at different time lags. UCS tests are conducted for specimens cured

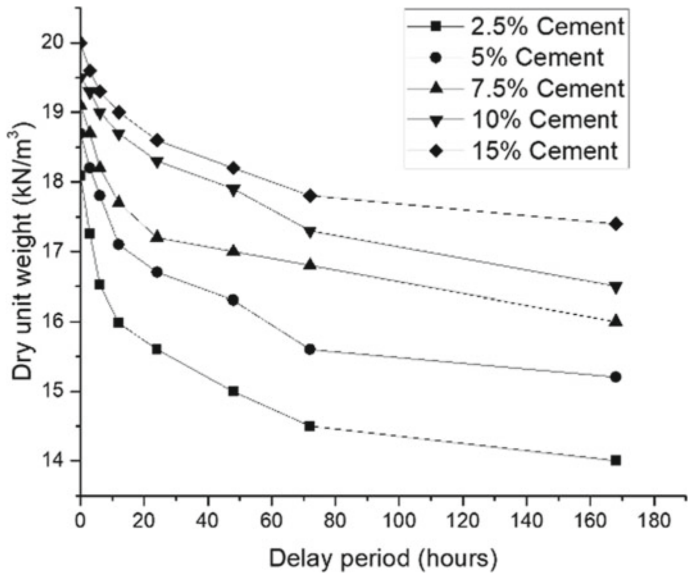

Fig. 9 Variation of dry unit weight of sand–cement mixes with delay time (light compaction)

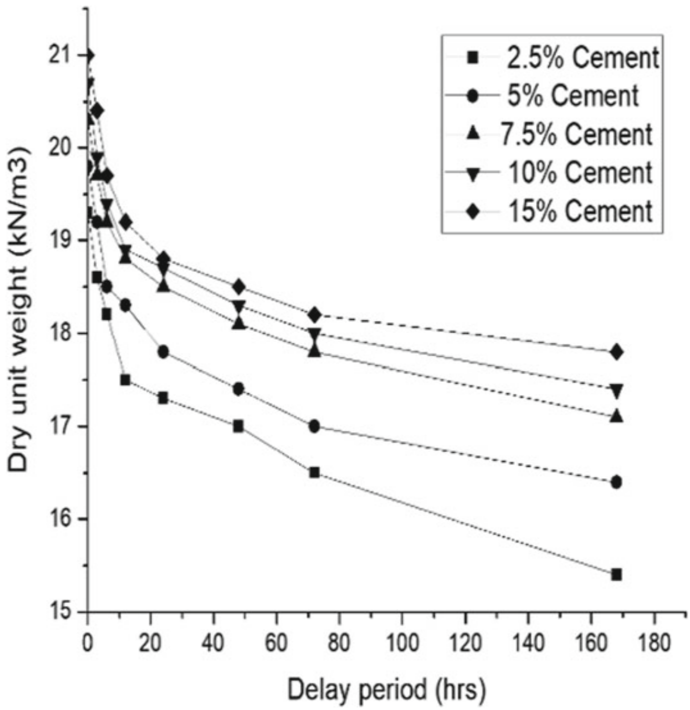

Fig. 10 Variation of dry unit weight of sand–cement mixes with delay time (heavy compaction)

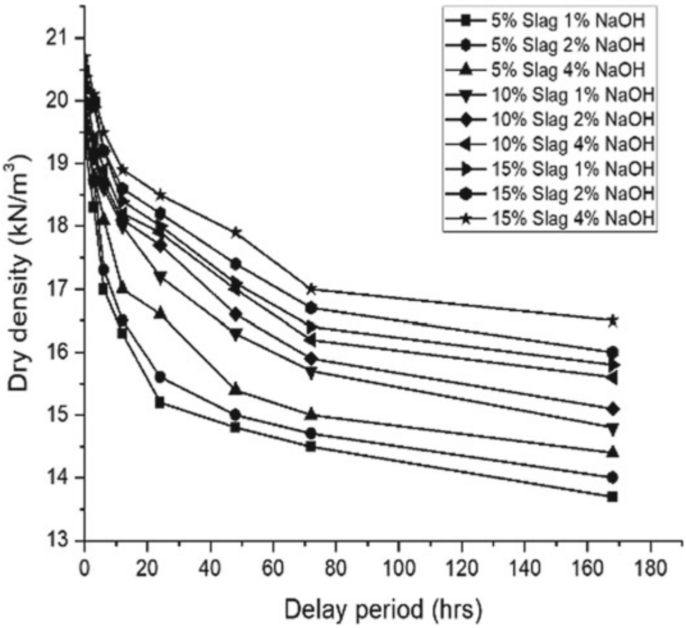

Fig. 11 Variation of dry unit weight of slag-based geopolymer with delay time (light compaction)

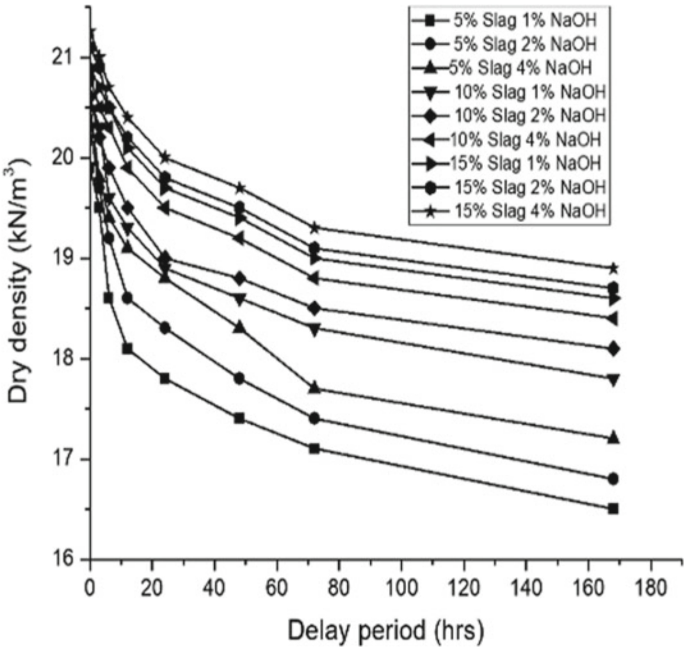

Fig. 12 Variation of dry unit weight of slag-based geopolymer with delay time (heavy compaction)

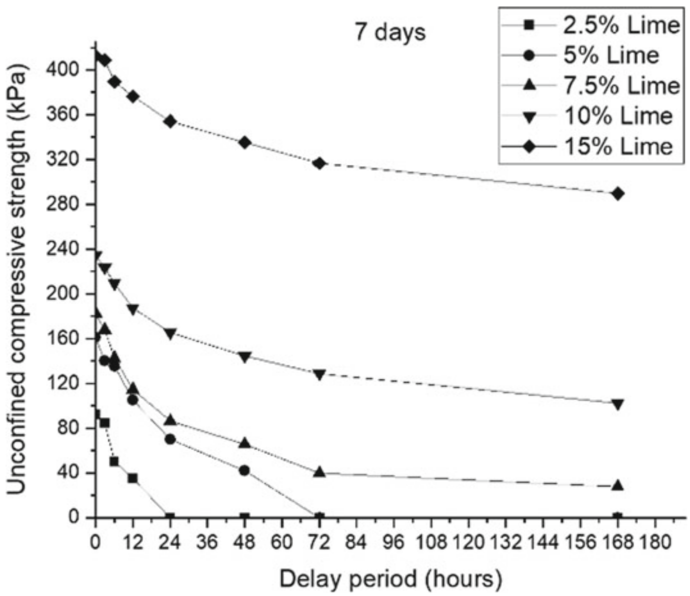

Fig. 13 Variation of unconfined compressive strength for different percentages of lime (7 days)

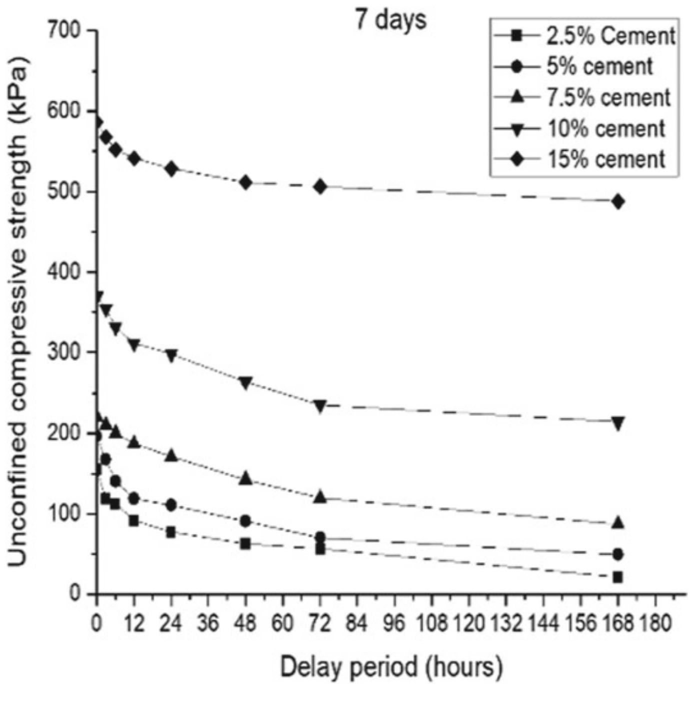

Fig. 14 Variation of unconfined compressive strength for different percentages of cement (7 days)

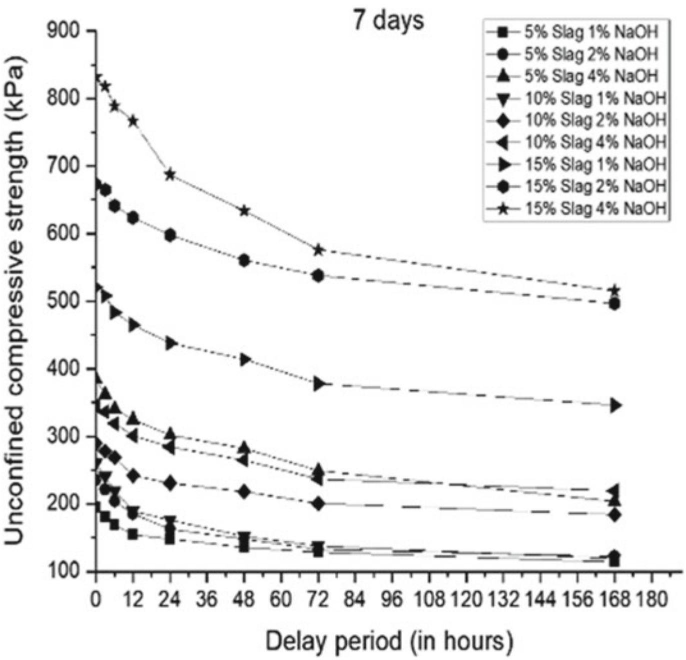

Fig. 15 Variation of unconfined compressive strength for different percentages of slag-based geopolymer (7 days)

for different curing periods at an average temperature of 30 °C. Based on the test results, the followings are concluded. For specimens prepared at both light and heavy compaction, optimum moisture content reduced while maximum dry density increased by increasing of cement, lime contents and slag-based geopolymer content. In delayed compaction, dry unit weight reduced with the time. This reduction of dry unit weight is due to hydration. For 7.5% cement, dry density is decreased by 18% for 24 h when compared to 12 h and 12% in case of lime. It indicates that reduction in dry unit weight is faster in case of cement. For 15% slag 4% NaOH alkaline activator, the dry density is decreased by 16% for 24 h when compared to 48 h. It is also observed that unconfined compression strength decreased with delay time and increased with the increment of cement, lime and slag-based geopolymer. It is observed that the unconfined compressive strength value for 15% cement is 29% more when compared to that of 15% lime after 7 days curing period. In unconfined compressive strength test when compared to cement and lime, slag-based geopolymer specimens have given good results.

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Strength and Deformation Characteristics of Lime-Admixed Black Cotton Soil Reinforced with Sisal Fibres

Jairaj, M. T. Prathap Kumar, M. Aashish, R. H. Basava, Y. Neeraj, and F. M. Sabira

Abstract Sustainable ground improvement techniques have led to the use of natural fibres to improve the shear strength properties of soil. Lime was admixed to black cotton soil starting from 1, 2, 3, and 4%, and optimum lime content was arrived at based on maximum value of dry unit weight and corresponding water content from compaction test. Using the optimum lime content thus obtained, black cotton soil was reinforced with randomly distributed sisal fibres of average length 10–20 mm and analysed for shear strength in terms of unconfined compression test. Sisal fibres were added in varying percentages starting from 0, 0.5, 1, 2, and 3%, and compaction characteristics were assessed. Remoulded specimens were prepared to have corresponding maximum dry unit weight obtained from compaction test and were tested for unconfined compressive strength to study the effect of curing with and without lime at different periods of 0, 10, 20, and 30 days. The result indicated sisal fibre content of 0.5% and indicated maximum shear strength with and without optimum lime. The surface characteristics of sisal fibre-admixed soil along with lime using X-ray refraction and scanning electron microscopic study revealed better bonding strength between soil and sisal fibre.

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Keywords BC soil · Sisal fibre · Unconfined compression test · Compaction

1 Introduction

Black cotton soils originate from basaltic rocks formed over millions of years due to volcanic action and cooling of lava, are referred to as “expansive soils”, and contribute to almost 20% of the terrains covering central India [1, 2]. Owing to the presence of high amounts of montmorillonite, these soils exhibit a peculiar swelling and shrinkage behaviour when subjected to changes in moisture content causing detrimental effects on the structures constructed over it [3–5]. Many years of research is contributed to finding ways to stabilize these soils in order to control and alter their properties as per the construction requirements [6–10]. Sisal fibre is very long, with an average length of 0.6–1.2 m, and it is creamy white to yellowish in colour. It is coarse, strong, and durable and can stretch. It also has good insulation properties, and it is highly resistant to bacterial damage and to deterioration in saltwater. Sisal fibre, modified soy protein resins, and composites were characterized for their mechanical and thermal properties. Sisal fibre is exceptionally durable and a low maintenance with minimal wear and tear. It contains 65% cellulose, 12% hemicelluloses, 9.9% lignin, and 2% waxes. Sisal fibre is known to increase the tensile property, CBR value, and shear strength of the soil [11–15]. Lime stabilization generates a long-term pozzolanic strength gain due to reaction between lime and the silica and alumina minerals present in clay, forming calcium silicates and calcium aluminates [16–19]. In the present study, BC soil admixed with optimum lime obtained from compaction test and reinforced with different percentages of sisal fibres was subjected to unconfined compression test (UCCS) after curing the specimens for different periods of 0, 10, 20, and 30 days. Similarly, BC soil admixed with different percentages of sisal fibres, 0.5, 1, 2, and 3%, is added to find the optimum percentage with and without lime. The test results were analysed for the efficacy of use of sisal fibres in BC soil admixed with and without lime [20, 21].

2 Materials and Methods

Black cotton soil (BC soil) was collected from Bidar district, Karnataka, India, form a depth of 0.9 m below the ground level. BC soil was pulverized, air-dried, and sieved through sieve no. BIS-40. Properties of BC soil are shown in Table 1. Class F hydrated lime was purchased form local hardware shop. The sisal fibres were procured from Kovai Green Fibres, Coimbatore, Tamil Nadu, and fibres that were cut to have average length of 10–20 mm [16, 22, 23] were used in the present study. All the basic properties were carried out as per IS codes.

Strength of sisal fibre-reinforced soil depends on dry unit weight and corresponding moisture content, which in turn depends on percentage of sisal fibres

Table 1 Properties of BC soil used

Description	Values
Liquid limit (LL)	64
Plastic limit (PL)	25
Shrinkage limit (SL)	12
Specific gravity (G)	2.61
Sand (%)	38
Silt and clay (%)	62

admixed in the BC soil. Further, sisal being a low-density fibre causes reduction in dry unit weight when admixed in BC soil. In this context, the compaction characteristics of sisal-fibre-reinforced BC soil assume importance which needs to be verified. To analyse the lime reaction on admixed sisal fibre BC soil, the sisal fibres mixed in lime-treated BC soil were also studied. Sisal fibre was mixed to BC soil at optimum lime content which was determined using provisions for "light compaction test as per relevant BIS standards" [24]. The compaction test was followed in the present study was similar light compaction test provisions by adding sisal fibre by dry weight percentage of BC soil. The average length of sisal fibres used was in the range of 10–20 mm [16].

To achieve uniform distribution of fibre, based on several trials and methods, the most suitable method of mixing the dry fibres with BC soil was adopted and water is then added to this dry mixture which produced less fibre lump formation. The lime percentage varied to get optimum lime content was 1, 2, 3, and 4% and to get optimum sisal fibre content was 0.5, 1, 2, and 3% with different curing periods in UCCS tests 0, 10, 20, and 30 days. The effect of curing period on unconfined compression strength was assessed with different curing periods of 0, 10, 20, and 30 days.

XRD and SEM tests were conducted at different curing periods to assess microstructural changes. Table 2 shows variation of maximum dry unit (MDU) weight and optimum moisture content (OMC) at different percentages of lime added to BC soil, which indicates that maximum dry unit (MDU) weight was obtained with corresponding OMC at 3% lime content. Hence, for the BC soil used in the experimental investigation, 3% lime was considered as optimum lime content (OLC).

Table 2 Variation of MDU and OMC at different percentages of lime added to BC soil

Lime (L) in %	MDU (kN/m^3)	OMC (%)
BC soil alone	16.11	16.96
BC soil +1% L	15.79	17.59
BC soil + 2% L	16.26	18.43
BC soil + 3% L	16.97	19.41
BC soil + 4% L	15.87	20.72

3 Results and Discussion

Compaction characteristics of sisal-fibre-admixed BC soil without lime and with OLC were determined. Table 3 shows the variation of MDU and OMC with different percentages of sisal fibre. It clearly indicates that increase in percentage of low-density sisal fibres in BC soil decreases density of BC soil. However, addition of 0.5% sisal fibre increases marginally the dry unit weight as compared to BC soil. It is attributed to possibility of uniform distribution of fibres at lower dosage leading to increase in bonding between soil particles.

3.1 Unconfined Compression Strength

To ascertain the strength characteristics of BC soil, unconfined compression tests were carried out with different percentages of lime and cured at periods of 0, 10, 20, and 30 days. Figure 1 shows the variation of unconfined compression strength

Table 3 Variation of maximum dry unit (MDU) weight and optimum moisture content (OMC) for sisal fibre-admixed BC soil

Sisal fibre (SF) in %	MDU (kN/m^3)	OMC (%)
BC soil alone	16.11	16.96
BC soil +0.5% SF	16.50	16.98
BC soil + 1% SF	16.00	20.37
BC soil + 2% SF	15.81	21.12
BC soil + 3% SF	15.37	24.22

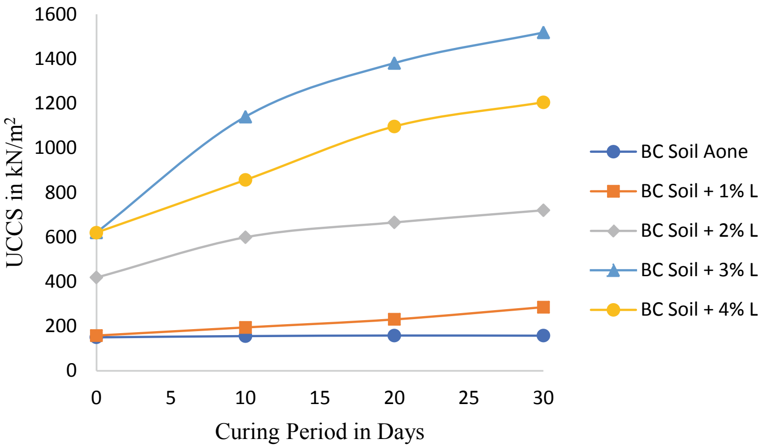

Fig. 1 Variation of UCCS of BC soil-admixed with different lime contents 0, 1, 2, 3, and 4% with different curing periods

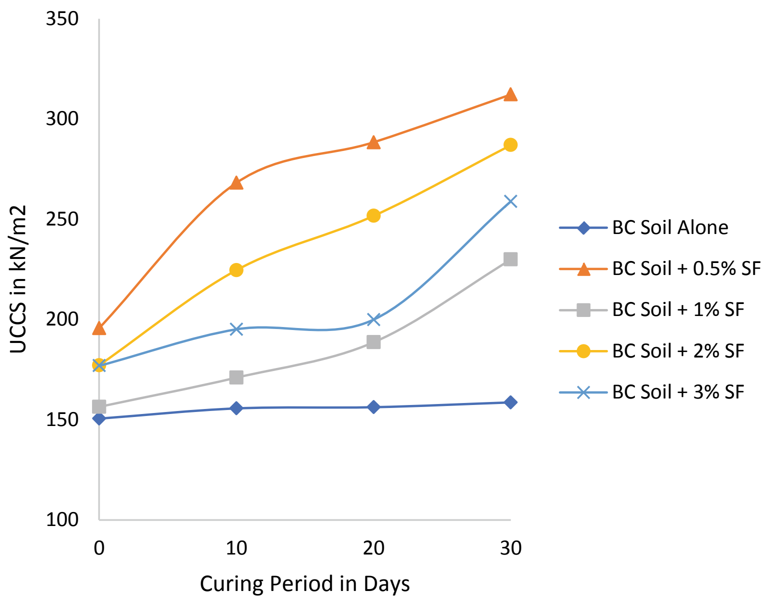

Fig. 2 Variation of UCCS with different sisal fibre contents of 0, 0.5, 1, 2, and 3% admixed BC soil

(UCCS) of BC soil admixed with different lime contents 0, 1, 2, 3, and 4% with different curing periods tested, and it concludes that at 3% lime optimum combination is higher dry unit weight for all the curing periods. Hence, both compaction test and UCCS test have indicated that 3% lime content is optimum for the BC soil used in the present study.

Figure 2 shows the variation of UCCS with BC soil admixed with sisal fibre content starting from 0, 0.5, 1, 2, and 3%. The test result indicates that 0.5% sisal fibre is optimum combination in terms of higher strength at all curing periods and UCCS is maximum at 30 day curing period. Based on the test results of BC soil admixed with lime alone and sisal fibre alone, it can be concluded that 3% lime and 0.5% sisal fibre are the optimum combination to achieve higher shear strength at 30 day curing period.

To assess the effect of admixing sisal fibre in lime-admixed BC soil, UCCS was conducted by admixing sisal fibre in BC soil + 3% optimum lime content by varying different percentages of sisal fibre. Figure 3 shows UCCS of BC soil + 3% OLC with different percentages 0.5, 1, 2, and 3% of SF with curing periods. It can be concluded from these results that, higher UCCS of at around 2000 kPa at 30 days curing period for BC Soil + 3% OLC with 0.5% SF indicates that sisal fibres are effective in increasing the shear strength of lime admixed BC soil to a large extent.

Increase in strength with increase in curing period indicates that the sisal fibres can sustain the exothermic reactions associated with lime addition to BC soil and hence be effective in increasing shear strength of lime-admixed BC soil. It can be attributed

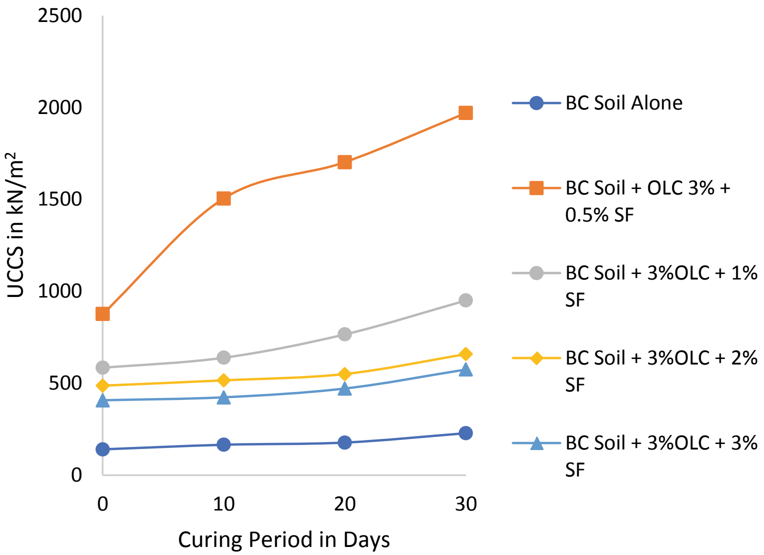

Fig. 3 Variation of UCCS with different sisal fibre contents of 0, 0.5, 1, 2, and 3% admixed BC soil

to the fact that addition of 0.5% sisal fibre ensures better bonding and addition of 3% OLC along with sisal fibre causes substantial jump in strength compared to all other combinations tested in the present study.

3.2 XRD and SEM Studies

To corroborate the trend in results of compaction test and UCCS test, XRD studies of BC soil admixed with 3% OLC with 0.5% sisal fibre for 0 and 30 day curing period were conducted. Figures 4 and 5 show variation of XRD pattern obtained for BC soil admixed with 3% OLC with 0.5% sisal fibre for 0 and 30 day curing period, respectively. BC soil admixed 3% OLC with 0.5% SF at zero-day curing period indicates that the soil predominantly has of montmorillonite clay mineral showing peak basalt spacing at 5.75 Å. Quartz/montmorillonite mineral strongly appeared at spacing of 27.5 Å, and BC soil admixed 3% OLC with 0.5% sisal fibre at 30 day curing period indicated associated lime reaction and presence of quartz mineral at spacing of 28 and 68.

Further, SEM studies of BC soil admixed with 3% OLC along with 0.5% sisal fibre at 0 and 30 days were also performed. Comparative analysis of Figs. 6 and 7 shows that at 30 day curing, BC soil admixed 3% OLC with 0.5% sisal fibre indicated the presence of white patches around soil particles. This is attributed to the formation of calcium carbonate due to pozzolanic reaction of lime-treated BC soil. Figure 7 clearly

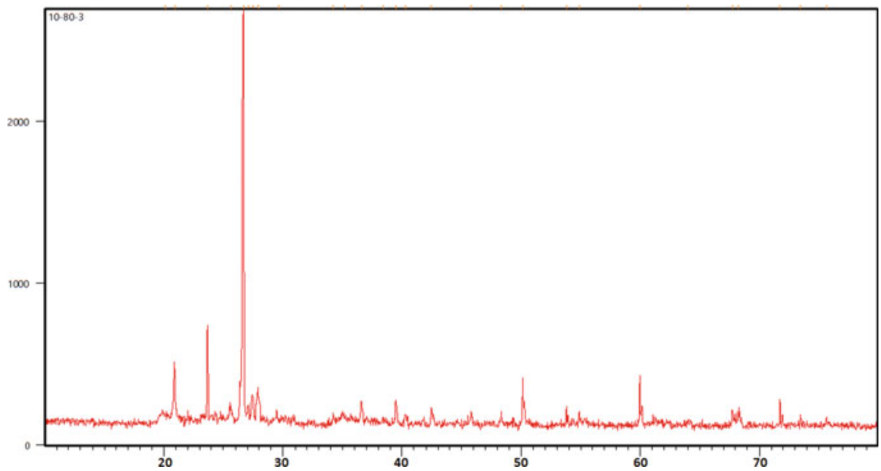

Fig. 4 BC soil admixed 3% OLC with 0.5% SF for zero-day curing period

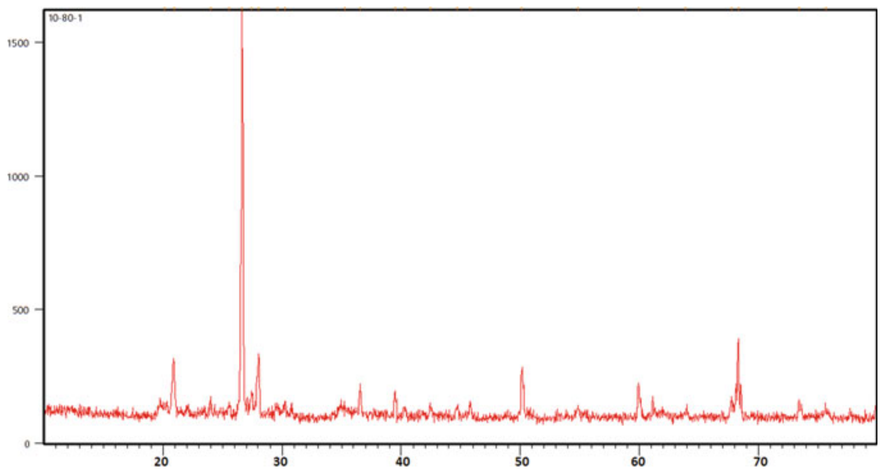

Fig. 5 BC soil admixed 3% OLC with 0.5% SF for 30 day curing period

brings out the associated bonding between sisal fibres and surrounding particles into BC soil admixed with 3% OLC at 30 day curing period. It is clear that 0.5% sisal fibre-admixed BC soil at 30 day curing period develops better bonding and hence better interface with surrounding particles in comparison with BC soil with 3% OLC admixed with 0.5% SF at 0 day curing period.

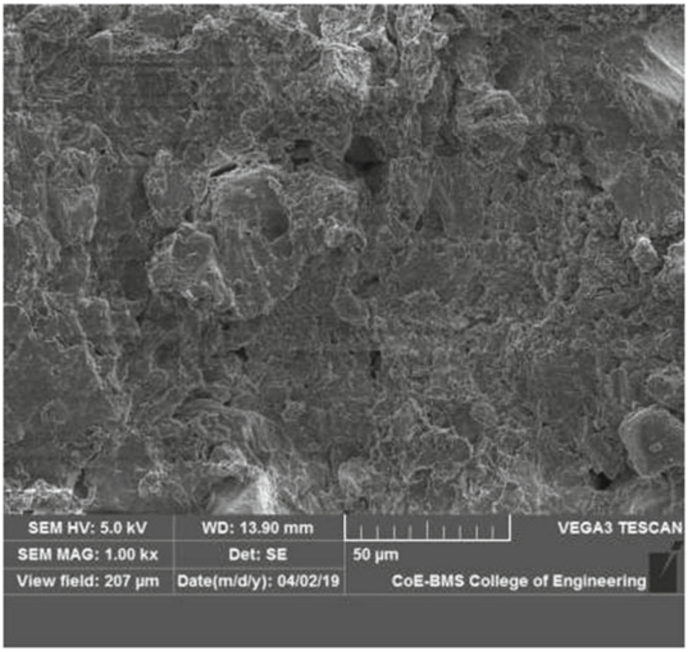

Fig. 6 SEM image of BC soil + 3% OLC + 0.5% sisal fibre at 0 day curing period

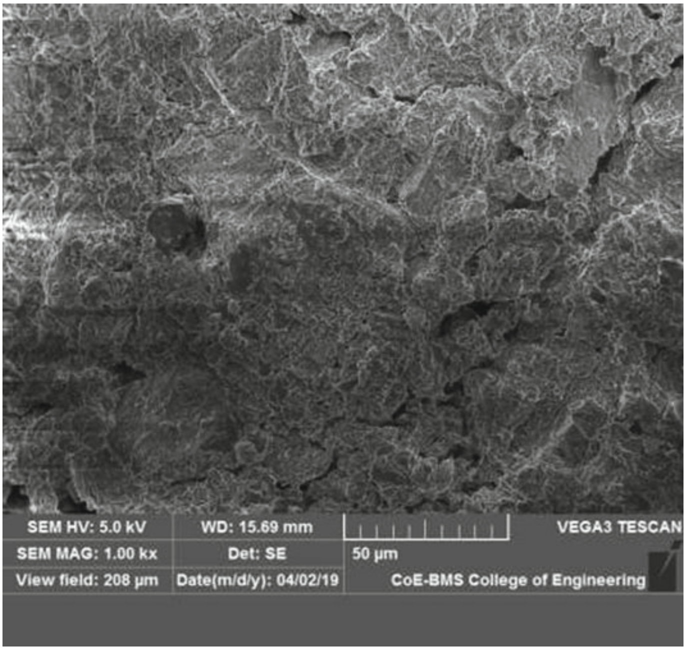

Fig. 7 SEM image of BC soil + 3% OLC + 0.5% sisal fibre at 30 day curing period

4 Conclusions

Based on the experimental observations, the following are the major conclusions:

1. The use of low-density sisal fibre decreases MDU with increase in percentage, and 0.5% can be considered to be optimum in terms of improvement in compaction characteristics of sisal fibre-admixed BC soil.
2. Increase in sisal fibre content causes a reduction in MDU along with the increase in the OMC for the case of both lime and sisal fibres-admixed BC soil. This is attributed to the fact that increase in percentage of fibre increases volume fraction of low-density sisal fibres that replaces soil particles.
3. Addition of sisal fibres along with optimum lime content causes an increase in UCCS at all curing periods. This is attributed to addition of lime causing an increase in shear strength due to associated pozzolanic reaction. The presence of sisal fibres along with lime induces better bonding, as envisaged in SEM studies, causing a significant jump in UCCS.
4. Increase in percentage of sisal fibre at all curing period indicates reduction in UCCS. This is because, higher the fibre content causes greater rebounding effect and consequent loss of contact from the surrounding particles due to increase in pore pressure as a consequence of compression. Based on the test results, it can be concluded that 0.50% sisal fibre can be considered optimum in BC soil admixed with and without lime.
5. BC soil admixed with 3% OLC and 0.5% sisal fibre has indicated greater shear strength at both 0 and 30 day curing period in comparison with other combination used in the present study. It is attributed to the fact that better interference prevails for this combination leading to higher shear strength even at zero-day curing as evidenced in SEM studies (30 day curing period).

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Compressibility Studies on Cochin Marine Clay Stabilized with Fly Ash and Lime Columnar Inclusions

Aswathy Rajendran, G. Sanoop, Sobha Cyrus, and Benny Mathews Abraham

Abstract The study focuses on the use of lime, fly ash and combination of these materials as columnar inclusion for improving the strength and compressibility characteristics of soft Cochin marine clay. Large-scale consolidation tank was used to study compressibility characteristics. A group of five columns of different materials (lime, fly ash and a combination of both) were installed in the tank filled with marine clay in its natural moisture content. The consolidation test was continued till the settlement rate reached a value less than 1 mm/day for the different applied pressures. It was observed that clay alone took 91 days to reach target settlement, whereas the lime column took 29 days, fly ash column took 51 days and 1:1 lime–fly ash column took 39 days. The shear strength of the clay increased with the installation of the columns of different materials.

Keywords Large-scale consolidation, lime column · Fly ash column · Lime–fly ash column

1 Introduction

Soft clays of low strength and high compressibility are located along the coastal and offshore areas, and they cause several foundation problems for the structures founded in these deposits. Construction in such soils requires ground improvement techniques. Use of fly ash and lime for ground improvement is a widely researched topic. Addition of fly ash to soil improves strength and decreases volume change behavior of expansive soils. These values are further decreased by addition of small percentage of lime [10]. Further, they reduce plasticity and linear shrinkage of expansive soils [4, 5] with nearly 6% lime added to soil giving optimum results in terms of strength, density and plasticity [9]. Fly ash, when compacted in the form of columns, decreases heave on clay beds [16]. Compacted lime–soil columns exhibit a stiffer and stronger response compared to conventional stone columns installed in soft soils,

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and its performance is remarkably enhanced by increasing the area ratio [13]. Also, their group efficiency decreases with increase in the number of columns for both lime and stone columns [1]. On addition of fly ash and lime, the montmorillonite structure of clays was found to be broken and pozzolanic action dominated over cation exchange capacity [17]. Lime–fly ash column effectively improves the physical and engineering characteristics of Cochin marine clay, and preloading technique offers better improvement in properties compared to compacted lime–fly ash columns [11]. In Kuttanad clays, shear strength, plastic limit and shrinkage limit are higher near lime columns and increase on curing [18], whereas moisture content and liquid limit decrease, permeability increase initially and then decrease.

Behavior of stone columns is affected by spacing, shear strength of clay, moisture content, diameter, etc., based on experimental and theoretical evaluation [3], and load settlement behavior may be taken as linear and maximum bulging occurs at 0.5 to 1 times column diameter from top. Geosynthetic encasement of stone columns increases their load carrying capacity further [11] by increasing its lateral confinement. Stone columns installed in soft clays provide moderate increase in load carrying capacity, accelerate consolidation settlement and thus reduce post-construction settlement [7]. In soft or loose layered soils, load carrying capacity of stone column-improved soils increases with diameter of columns [6]. However, subsoil investigation from boreholes should be supplemented by in situ test results before designing stone columns [8].

Numerical modeling techniques using finite element method (FEM) and finite difference method (FDM) are highly effective in understanding the long-term field performance of soils improved with columns. Studies using FLAC, an FDM package and a simple rectangular grid have been used to represent the foundation soil, modeled in plane strain [20], and interactions between the individual stone columns, the loaded area and the surrounding soil can be understood as the behavior of ‘piles’ with nonlinear, sand-like axial stiffness properties. In FEM, Mohr–Coulomb criterion is employed for drained analysis of clay, sand and stone and analysis is being carried out using a unit cell concept; i.e., deformations in the clay are restrained within unit cell [2]. From FEM analysis of geogrid encased stone columns [12], stone column derives its resistance by its bulging over a length of 4d to 6d under the load with maximum bulging at the depth around 2d and the column material offers passive resistance against bulging. From FEM analysis of fully drained stone columns in soft clays [19], the major foundation parameters affecting their group response were identified as area ratio, normalized column length, Young’s modulus of column, over-consolidation ratio, initial geostatic stresses and clayey soil parameters. From numerical analysis [14], assumptions, procedures and results of behavior of non-encased versus geogrid encased stone column in soft clay with the aid of finite element software, PLAXIS V8, a reasonable agreement obtained between the experimental investigation and the finite element method. Compacted lime-well-graded soil (CL-WS) columns increase the load carrying capacity of soft soils and reduce the settlement [13], but FEM results indicate influence of model size on the stiffness of the specimens. However, for specimens containing columns with diameter greater than 100 mm, the variations

Table 1 Properties of sand used in the study

Properties	Values
Specific gravity	2.57
Effective size, D10 (mm)	0.52
D30 (mm)	0.72
D60 (mm)	1.1
Uniformity coefficient, Cu	2.11
Coefficient of curvature, Cc	0.906
IS classification	SP

of stiffness become negligible and hence the results can be used to extrapolate and predict the full size behavior of these columns.

The paper concentrates in investigating consolidation behavior of lime columns and fly ash columns in soft marine clays. Large laboratory consolidation tests and numerical analysis were carried out to study the ground improvement achieved.

2 Materials

The materials used for the study are marine clay, lime, class F fly ash and uniformly graded fine sand. Marine clay used in the study was collected from Vallarpadam, in Cochin, on the western coast of India. For uniformity, samples were pooled together and mixed thoroughly into a uniform mass and preserved in polythene bags to maintain the water content. The properties of marine clay are presented in Table 2. The sand used in the study was obtained from the Periyar River Basin. This sand was sieved using IS sieve 2 mm and 425 μ, and the medium size fraction of the river sand thus obtained was taken for the study. The properties of the sand used in the study are given in Table 1. The commercially available superior grade quick lime was used to prepare lime column. The specific gravity of lime used in the study is 2.36. ASTM C618 specified two categories of fly ash, class C and class F. In the present study, class F fly ash is collected from the Hindustan Newsprint Ltd, Vellore, Kottayam. The fly ash had a natural water content of 14%, and its specific gravity is 2.26. Figure 1 shows the XRD pattern of class F fly ash used in the study.

3 Experimental Program

Consolidation study of columnar inclusions on clay bed cannot be conducted in laboratory oedometer test, so large-scale tanks were fabricated. The following section gives in detail the experimental setup. The setup consists of a model tank of height 700 mm and diameter 400 mm. The model tank boundary was determined on the

Table 2 Properties of marine clay used in the study

Properties	Values
Natural moisture content (%)	127
Specific gravity	2.65
Liquid limit (%)	135
Plastic limit (%)	58
Shrinkage limit (%)	16
Plasticity index (%)	77
Particle size distribution Clay size (<0.002 mm) (%) Silt size (0.002–0.075 mm) (%) Sand size (>0.075 mm) (%)	60 35 5
IS classification	MH

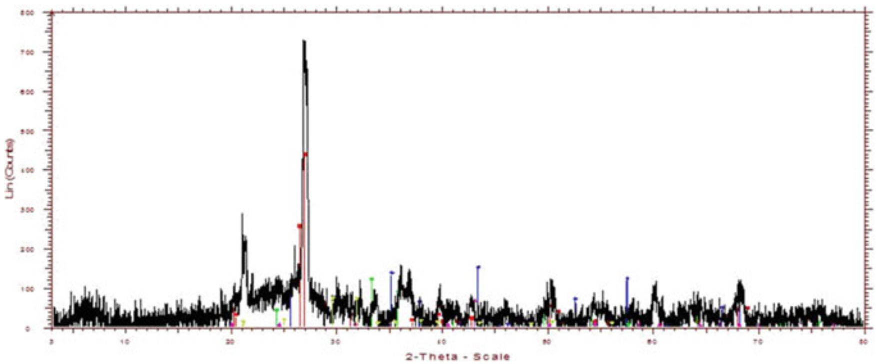


Fig. 1 XRD pattern of class F fly ash used in the study

basis of criterion that induced stresses should be insignificant at the tank boundaries. The load is applied to the clay bed by means of a loading frame using the principal of lever arm. The bottom portion of the cylinder has holes, and the upper portion of the clay bed consists of a porous plate to allow two-way drainage and thus simulate the field condition. The apparatus has a loading capacity of 35 kPa. The loading frame is considered as a cantilever beam and the load transferred to the cylinder calculated as: $X = 2.4$ W/05. Figure 2 shows the schematic diagram of experimental test arrangement.

A group of 5 columns with diameter 40 mm, length 580 mm and spacing 120 mm were installed in the model tank. The bottom of the tank was provided with a 50 mm-thick sand filter layer with drainage outlets. Thin open-ended PVC pipe of diameter 4 cm was placed inside the tank with the help of guards, and the clay was filled in the tank in between the PVC pipes with uniform compaction to achieve a density of 15 kN/m^3. Then, the column materials were filled into the PVC pipes in layers of 50 mm each giving uniform compaction to each layer using metallic rods. Casing

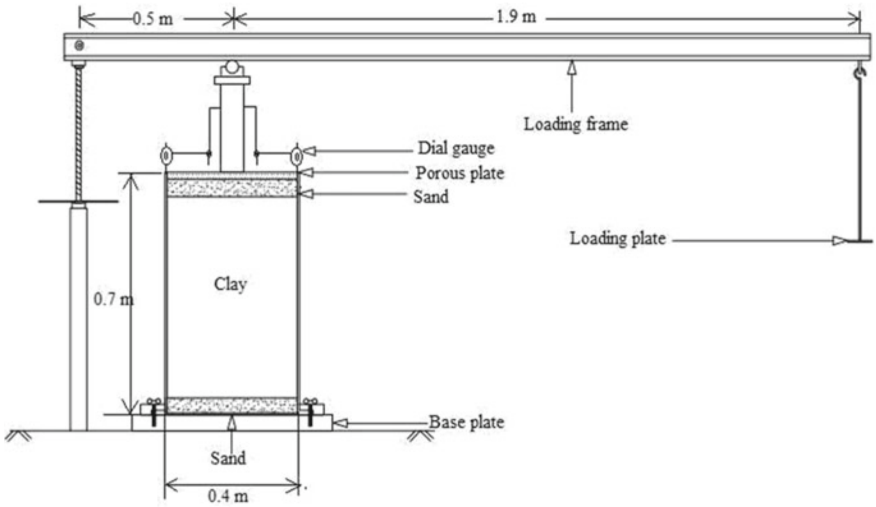
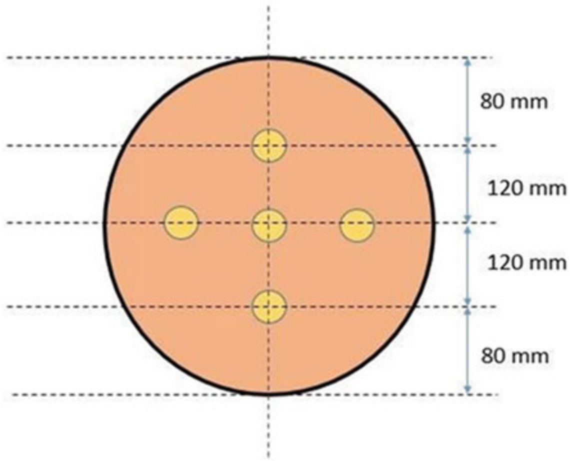


Fig. 2 Arrangement of column in the model tank

pipe is raised in stages ensuring minimum 5 mm penetration below the top level of the placed column material. Drainage was permitted at the top of the clay bed by placing 50 mm-thick sand layer and above that a porous metal plate. After preparing the column, the applied pressure v/s deformation behavior of column/treated soil will be studied by applying vertical load in a loading frame. The experiment was continued till the settlement rate reaches a value less than 1 mm/day for different applied pressures (12.5, 16.8, 21.04, 29.63, 46.8 kN/m^2). After consolidation, the clay surrounding the column was collected to study the strength characteristics. Figure 3 shows the arrangement of column in the model tank.

Fig. 3 Arrangement of column in the model tank



Lime columns and fly ash columns were installed in the test tank as explained above. Lime powder and fly ash passing through 425μ were filled and compacted to achieve a density of 10.4 kN/m^3.

4 Numerical Modeling

An axisymmetric analysis was carried out using Mohr–Coulomb's criterion for clay and other materials. The geometry model of tank with group of column is shown in Fig. 4. After drawing the geometry of the model, properties were assigned according to the study. Next step was to give the boundary condition and the load to the column. Then, the meshing was done. The mesh was refined at all the column portion. After the mesh generation, next step was staged construction and in that step the calculation has been carried out in 5 phases, i.e., initial phase, phase 1 and phase 2, phase 3 and phase 4. In the initial phase, the whole cylindrical model has the soft clay properties and other particulars like load and the column cluster were deactivated. Then, a preconsolidation pressure of 10 kN/m^2 was given as two stages in phase 1 and phase 2 with a time interval of 25 days. In phase 3, the properties of column material were assigned to the column cluster. In the 4th phase, the load system was activated. Points or the curve was selected and proceeded to the calculation stage.

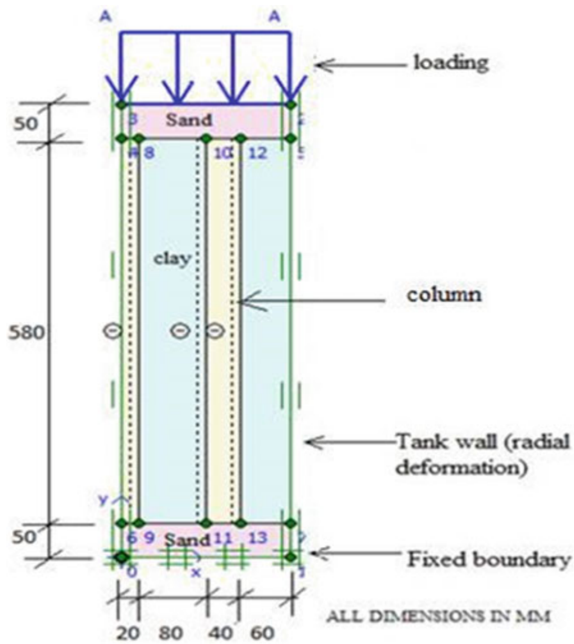


Fig. 4 Geometry model of tank with column

5 Results and Discussion

Consolidation tests were performed on models with only clay, group of lime column, fly ash column and 1:1 lime–fly ash column in the model tank. In order to have a settlement rate less than 1 mm/day for five different applied pressures (12.5, 16.8, 21.04 , 29.63 and 46.8 kN/m^2), the clay alone model in experimental setup took 91 days whereas the lime column installed in the tank took approximately 29 days, fly ash column took 51 days and lime–fly ash column took 39 days. From the obtained results, for the same applied pressure, the variation in time taken to have the same amount of settlement for only clay, group of lime column, fly ash column and lime–fly ash column is shown in Table 3. Table 3 show the percentage reduction in time for lime, fly ash and 1: 1 lime–fly ash column.

From the data given above, it is clearly seen that the rate of consolidation is more when lime column installed in the clay model followed by 1:1 lime–fly ash column and fly ash column.

For the same applied pressure of 25 kPa, the value of coefficient of consolidation of clay in large-scale consolidation test tank is 3.65×10^{-4} cm^2/s. The value of coefficient of consolidation increased to 6.74×10^{-4} cm^2/s when lime column installed in the model test tank and 6.01×10^{-4} cm^2/s for 1:1 lime–fly ash column and 5.38×10^{-4} cm^2/s for fly ash columns. The value of compression index (Cc) for clay alone is 0.857, and this value decreased to 0.687 when lime column installed in the model test tank and 0.744 for 1:1 lime–fly ash column and 0.755 for fly ash columns.

Table 3 Variation in time taken to reach same settlement under different applied pressures

Applied pressure (kPa)	Time taken in days						
	Clay alone	Lime column	Fly ash column	1:1 lime–fly ash	Lime column—% increase	Fly ash column—% increase	1:1 lime—fly ash—% increase
12.5	10	5	8	6	30	20	40
16.8	22	13	16	13	41	28	41
21.04	44	16	27	17	64	39	62
29.63	67	21	39	28	69	42	59
46.81	86	29	51	39	67	41	66

Table 4 Consolidation characteristics of clay after installation of columns

Combination	Compression index (Cc)	Coefficient of consolidation (Cv—cm^2/sec)
Clay alone	0.859	3.65×10^{-4}
Lime column	0.687	6.74×10^{-4}
Fly ash column	0.7×55	5.38×10^{-4}
1:1 lime–fly ash column	0.744	6.01×10^{-4}

Thus, it can be inferred that installation of columns in the clay bed accelerates the rate of consolidation. Also, it can be observed that of all the columnar materials used in the study, lime gave better compressibility characteristics (Table 4).

Laboratory vane shear test was conducted on the test bed of the different combinations of materials before and after consolidation. Shear strength of the marine clay used in the study is 6.639 kPa. This value increased to 14.32 kPa after the consolidation for 91 days without any columnar inclusions. When columns were installed in the model test tank, the undrained shear strength value increased to 18.31 kPa in case of lime column, 32.01 kPa for fly ash column and 23.47 kPa for 1:1 lime–fly ash column. Thus, it can be inferred that installation of columns in the clay bed increased the undrained shear strength of soil.

Experimental results were used for the validation of finite element software PLAXIS 2D, for the purpose of cross-checking the set of results of only clay in the model tank and group of 5 columns of lime, fly ash and their combination. The dimension of the finite element model has been kept as the same as that of the experimental model. The time–settlement plot of the only clay obtained from the experimental observations matches well with that obtained from PLAXIS analysis, whereas the time–settlement curve for group of column obtained from PLAXIS results shows a variation from that of the experimental results in higher applied pressure. For the same settlement, for an applied pressure of 46.8 kN/m^2 the results from PLAXIS show 10–12 days more time as that obtained from the experimental results for lime column, 20–25 days more in case of fly ash column and 10–15 days more in the case of 1:1 lime–fly ash column.

6 Conclusions

For the same applied pressure of 25 kPa, the value of coefficient of consolidation (Cv) of clay in large-scale consolidation test tank is 3.65×10^{-4} cm^2/s. The value of Cv increased to 6.74×10^{-4} cm^2/s when lime column installed, 6.01×10^{-4} cm^2/s for 1:1 lime–fly ash column and 5.38×10^{-4} cm^2/s for fly ash columns. The Cc value for clay alone is 0.857, and this value decreased to 0.687 when lime column installed and 0.744 for 1:1 lime–fly ash column and 0.755 for fly ash columns.

Installation of columns in the clay bed accelerates the rate of consolidation, and of all the columns used in the study, installation of lime column gave better compressibility characteristics. Lime column gave better drainage path compared to fly ash columns based on experimental results. However, all the combinations of columns provided better drainage path compared to clay only condition.

Undrained shear strength of the marine clay used in the study is 6.639 kPa. This value increases to 14.32 kPa after the consolidation without any columnar inclusions. When column was installed in the model test tank, the shear strength value increases and it becomes 18.31 kPa in case of lime column, 32.01 kPa for fly ash column and 23.47 kPa for 1:1 lime–fly ash column.

The rate of increase in undrained shear strength of the clay bed with fly ash columns showed maximum increase of 79%. Considering the strength criteria, fly ash columns perform better than others.

In addition to the environmental benefits achieved through the use of fly ash, it has the lowest embodied CO_2 content of 4 Kg CO_2/ton compared with cement (930 Kg CO_2/ton) and limestone 32 (Kg CO_2/ton) which makes them suitable for stabilizing purposes.

Experimental results were used to compare the results from the finite element software PLAXIS 2D. The time–settlement plot of the only clay obtained from the experimental observations matches well with that obtained from PLAXIS analysis.

The time–settlement curve for group of column obtained from PLAXIS results showed a variation from that of the experimental results in higher applied pressure. For the same settlement, for an applied pressure of 46.8 kN/m^2 the results from PLAXIS show 10–12 days more time as that obtained from the experimental results for lime column, 20–25 days more in case of fly ash column and 10–15 days more in the case of 1:1 lime–fly ash column.

Numerical study shows that installation of single column in the clay bed takes 20–35 days more than that of group of column for all materials to achieve a particular settlement of 20 mm.

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Load Settlement Behaviour of Soft Soil with 3D-Reinforced Sand Piles

O. Megha, M. N. Sandeep, and K. S. Beena

Abstract Nowadays due to scarcity of land, structures are built in sites with the available soil conditions including soft soil layers. In site conditions where deep foundations are not a viable solution due to economic considerations, the existing foundation soil has to be modified for better bearing capacity and reduced settlement. In the present scenario, a wide variety of ground improvement techniques is available including modification by inclusions and confinement. The use of sand piles is a proven technique to improve the desirable properties of soft soil. In this study, the effectiveness of sand piles is further improved by introducing 3D elements into the sand piles. A series of laboratory plate load tests were conducted on a model footing resting on soft clay reinforced with sand pile with and without 3D elements in it. The tests were conducted using two different shapes of 3D elements, namely tetrapod and pentapod. Also, tests were conducted for different volume ratios of 3D elements as 1.82, 2.27 and 2.73% at a constant relative density. Between the two shapes, pentapod elements were more effective due to better confinement of soil between the legs owing to the geometry. Also, the bearing capacity is improved by increasing the volume content of 3D elements from 1.82 to 2.73%. From the results, it can be found that for clay only, the bearing capacity at a settlement ratio of 4% is 94.86 kN/m^2. The bearing capacity of the clay is improved to 122.76 kN/m^2 when reinforced with sand pile only at 50% relative density. For same relative density of sand pile, the bearing capacity of clay is improved to 167.40 and 212.04 kN/m^2 for sand piles-reinforced tetrapod and pentapod elements, respectively, at volume content of 2.73%. This shows that 3D elements can be effectively used in sand piles for improving the properties of soft clay.

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Keywords Sand pile · 3D elements · Bearing capacity

1 Introduction

Ground improvement is a prerequisite for shallow foundation structures constructed in soft soil which has low bearing capacity and susceptibility to excessive settlement on loading. There are different methods to improve soft soil as per the site requirement such as mechanical modification, hydraulic modification, physical and chemical modification and modification by inclusions and confinement. Modification by inclusions and confinement denotes stabilizing the soil using fibres, strips, bars, geotextile, etc. The use of sand piles can also be included in this category to improve the soil properties. In this study, the effectiveness of 3D reinforcement, namely tetrapod and pentapod, along with sand pile to improve the bearing capacity of soft soil was studied by conducting a series of plate loads.

Lawton et al. [4] introduced the concept of three-dimensional reinforcement, by conducting field experiments performed to evaluate the practical aspects of reinforcing cohesionless soils with multi-oriented geosynthetic inclusions. Tests on California bearing ratio, resistance to compaction, rutting, etc., were conducted. Substantial increases in stiffness and strength were obtained by reinforcing cohesionless soils with multi-oriented inclusions. Multi-oriented elements also proved useful in reducing rutting in soft and loose soils. Merry et al. [5] carried out a study based on the performance of cohesionless soil-reinforced multi-oriented geosynthetic inclusions or geojacks placed over a biaxial geogrid. The results indicate that the combined reinforcement of biaxial geogrids and geojacks improves the ultimate bearing capacity in comparison with geogrid alone.

Harikumar et al. [2] carried out a study to investigate the effect of hexapods on the shear strength characteristics of sand using triaxial testing. The variables are volume content, hexapod concentration and number of hexapods per layer and the spacing of layers. From this study, it was concluded that the hexapods are effective for improving the shear strength characteristics of sand, and the layered arrangement is more effective. Harikumar et al. [2] carried out another study on the behaviour of model footing resting on sand bed reinforced with multidirectional reinforcing elements. The main objective of this study is to investigate the feasibility of using multidirectional reinforcement to improve the bearing capacity of soil. The reinforcement used was hexapods. The variables are depth to first layer of reinforcement, spacing between reinforcing elements, number of layers and spacing between layers. From this study, it was found out that the optimum depth to first layer of reinforcement is 0.5 B, optimum spacing between reinforcement 1b, optimum number of layers 2–3 and optimum spacing between layers 0.5 B.

Load settlement behaviour of soft soil reinforced with sand piles was studied by Devendra et al. [1]. The main objective of the study was to investigate the pressure settlement behaviour of a footing resting on a sand pile for a different S/D ratio and L/D ratio in soft black cotton soil. The materials used are black cotton soil and poorly

Table 1 Properties of test soil

Property	Value
Specific gravity	2.36
Liquid limit	53%
Plastic limit	27%
Shrinkage limit	16%
OMC	18%
Maximum dry density	1.54 g/cc
Unconfined compressive strength	27.2 kN/m^2
IS classification	CH
Organic content	8.3%
Clay content	61.82%
Sand content	20.17%
Silt content	18%

graded sand. The variables are length of pile, centre-to-centre distance of pile and diameter of footing. From this, it was found out that the optimum spacing of sand pile is 2.5 D and the optimum length of pile is 5 D where D is the diameter of the pile.

2 Test Materials

2.1 Clay

Locally available clayey soil, collected from Alappad region, Thrissur, was used in the investigation. The properties of clay are given in Table 1.

2.2 Sand

Locally available clean river sand obtained from Pattambi region, Palakkad, was used for the study. The engineering and index properties of sand are given in Table 2.

Table 2 Properties of sand

Property	Value
Specific gravity	2.65
Uniformity coefficient	3.36
Coefficient of curvature	0.94
IS soil classification	SP
Maximum dry density	1.83 g/cc
Minimum dry density	1.48 g/cc
Effective size D_{10}	0.22
D_{30}	0.39
D_{60}	0.74
Angle of internal friction at relative density 50%	39°

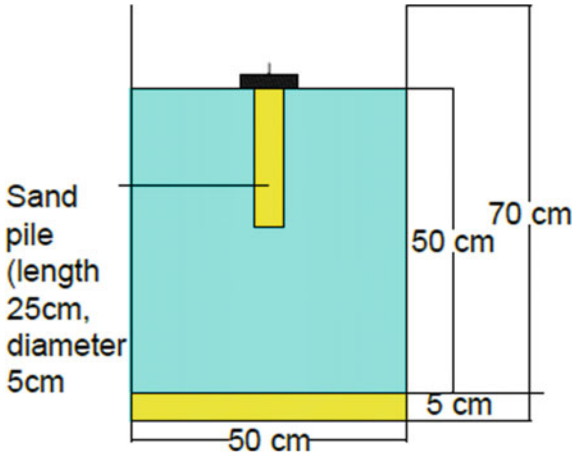
2.3 3D reinforcement

Two types of 3D jacks are used in this study, namely tetrapods and pentapods. Reinforcing elements were manufactured by 3D printing using polypropylene. The physical properties of the material are: density—0.995 g/cm^3 and melting point—175 °C (as provided by manufacturer). The width of tetrapod is 2 cm and that of pentapod 2.84 cm. Both tetrapod and pentapod are having a surface area of 15.53 cm^2 and weight of 3 g. The reinforcing elements are shown in Figs. 1 and 2.

Fig. 1 Tetrapod

Fig. 2 Pentapod

Fig. 3 Schematic test set-up



3 Methodology

3.1 Test Set-up

The test set-up for laboratory plate load test consists of the following.

(a) Test tank of size 500 mm × 500 mm × 700 mm, made of mild steel sheets.
(b) MS plate of size 100 mm × 100 mm × 20 mm.
(c) Hydraulic jack, 100 kN capacity.

The test set-up and loading arrangement are shown in Figs. 3 and 4.

3.2 Test Procedure

Series of plate load test were conducted as IS 1883-1962. A pre-calculated weight of clay was mixed with water at a moisture content of 24%, which is greater than OMC (wet of optimum) and filled in five layers to reach the required height. At the bottom of the tank, there is layer of sand in 5-cm thickness, which is provided for drainage. After filling the clay up to the mark made on the side wall of the tank, each layer was compacted and levelled using a rammer. Tank was filled with clay up to the level where footing has to be placed in unreinforced case, and then it is subjected to loading.

The inner faces of the test tank were made smooth to reduce the boundary effects. Holes are made in the bottom of the tank for drainage. A pre-calculated weight of clay was mixed with water at a moisture content of 24%, which is greater than OMC (wet of optimum). A PVC pipe of 25 cm depth and 5 cm diameter is used for providing sand pile. Initially, sand is placed in the bottom of the tank at thickness of 5 cm for

Fig. 4 Loading arrangement

drainage. Then, clay is placed in layers of equal thickness, up to the required height. Then, the PVC pipe is placed in the centre of the tank, and again clay is filled up to the required level. Sand is placed inside the pipe at the required relative density.

The sand is compacted using a small rammer. Then, the pipe is slowly lifted. Then, test is also conducted using 3D jack-reinforced sand pile. Tetrapods and pentapods are used to reinforce sand. The volume content of jacks is varied as 1.82, 2.27 and 2.73%.

Then, the load is applied by means of hydraulic jack, and the load and settlement are measured. The maximum bearing capacity was compared at a settlement ratio of 4%.

4 Results and discussion

4.1 Effect of unreinforced sand pile

Figure 5 shows the load settlement behaviour of unreinforced soil and soil reinforced

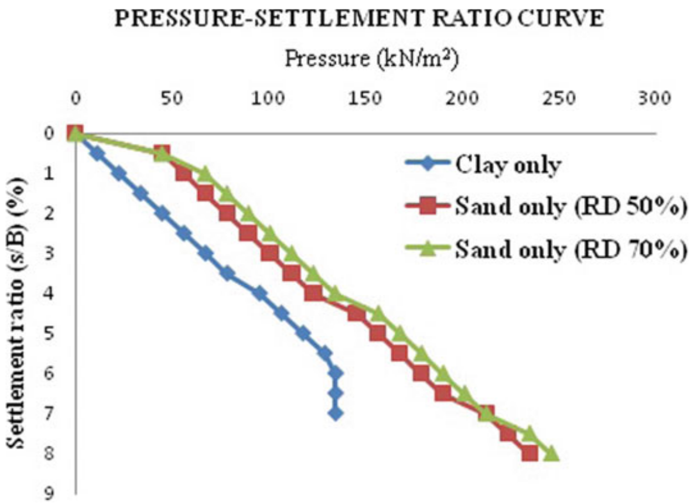


Fig. 5 Pressure settlement ratio curves for soil with and without sand pile reinforcement

Table 3 Effect of sand pile on bearing capacity of soil

Test condition	Bearing capacity at s/B of 4% (kN/m^2)
Clay only	94.86
Clay + sand pile (RD 50%)	122.76
Clay + sand pile (RD 70%)	133.92

with sand piles at different relative densities. The sand piles improve the bearing capacity of footing resting on soft clay from 94.86 to 122.76 kN/m^2 with a percentage improvement of 29% at relative density of 50% at settlement ratio of 4% (Table 3).

From the pressure–settlement ratio curves, it was found that when sand pile is provided there is a large increase in bearing capacity. This is because of the increased stiffness of the reinforced soil medium due to the installation of the sand pile.

4.2 Effect of Volume content of Tetrapod (RD 50%)

Figure 6 and Table 4 show the effect of volume content of tetrapod on the bearing capacity of footing resting on tetrapod-reinforced sand pile at a relative density of 50%. Volume content (V_r) is varied as 1.82, 2.27 and 2.73%. From the pressure–settlement ratio graphs, it was found that as the volume content increases the bearing capacity also increases. The optimum volume content is 2.73%. The increase in bearing capacity is due to the fact that as the volume content increases the number of tetrapods increases. So, the surface area of reinforcement that comes in contact with

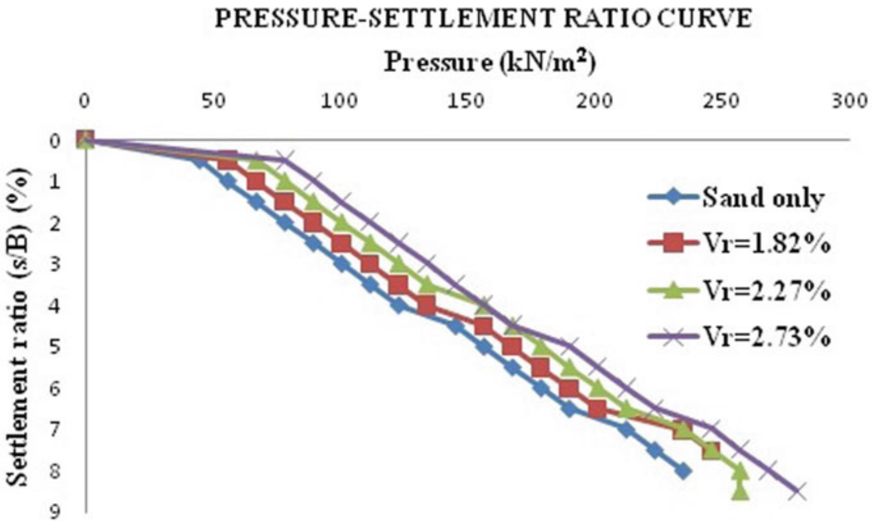


Fig. 6 Pressure settlement ratio curves for sand pile reinforced with tetrapod for different volume contents

Table 4 Effect of volume content of tetrapods on bearing capacity (RD 50%)

Test condition	Bearing capacity at s/B of 4% kN/m^2)
Sand only	122.76
Tetrapod ($V_r = 1.82\%$)	133.92
Tetrapod ($V_r = 2.27\%$)	156.24
Tetrapod ($V_r = 2.73\%$)	167.40

the soil will be more, which in turn increases the friction, confinement and stiffness of reinforced soil mass. Comparing the increment in volume content from 1.82 to 2.27%, the percentage improvement in bearing capacity is about 16% at a settlement ratio of 4%. Similarly for the increment of volume content from 2.27 to 2.73%, the percentage improvement is 7 %. There is a marginal increase only in the second case. This may be due to transfer of load from sand particles to 3D jacks and less coherent soil structure to increased number of 3D elements.

4.3 Effect of Volume content of Pentapod (RD 50%)

Figure 7 and Table 5 show the effect of volume content of pentapod on the bearing capacity of footing resting on pentapod-reinforced sand pile at a relative density of 50%. Volume content (Vr) is varied as 1.82%, 2.27 and 2.73%. From the pressure–settlement graphs, it was found that as in the case of tetrapod, for pentapod

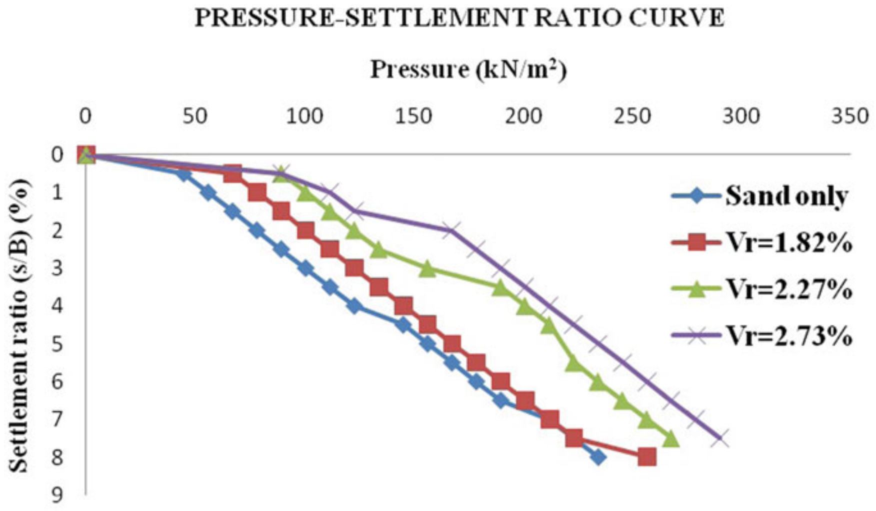


Fig. 7 Pressure settlement ratio curves for sand pile reinforced with pentapod for different volume contents

Table 5 Effect of volume content of pentapods on bearing capacity (RD 50%)

Test condition	Bearing capacity at s/B of 4% (kN/m^2)
Sand only	122.76
Pentapod ($V_r = 1.82\%$)	145.08
Pentapod ($V_r = 2.27\%$)	200.88
Pentapod ($V_r = 2.73\%$)	212.04

reinforcement also as the volume content increases the bearing capacity of reinforced soil also increases. The optimum volume content is 2.73%. Comparing the increment in volume content from 1.82 to 2.27%, the percentage improvement in bearing capacity is 38% at a settlement ratio of 4%. Similarly for the increment of volume content from 2.27 to 2.73%, the percentage improvement is 5%. There is a marginal increase only in the second case.

4.4 Effect of Shape of Reinforcement (RD 50%)

From the pressure–settlement ratio curves, it was found that when comparing each volume content of pentapod and tetrapod, pentapods are more effective, and the maximum value for bearing capacity is obtained for pentapod at a volume content 2.73%. This is because of the fact that when comparing tetrapods and pentapods, the quantity of soil confined between the jacks will be more in case of pentapods owing to its geometry.

5 Conclusions

- The bearing capacity of soft soil is improved by 29% from unreinforced condition to reinforced condition with sand pile of relative density 50%. Similarly, there was an improvement in bearing capacity of soil by 41% when comparing unreinforced condition to reinforced condition with sand pile of relative density of 70%.
- The bearing capacity of soil reinforced with sand piles and 3D elements increased as the volume content of elements was increased. But the percentage increment in improvement decreased as volume content of 3D elements was increased. This volume content of 2.27% is considered as the optimum percentage of 3D reinforcements for both tetrapods and pentapods.
- The soft soil having tetrapod-reinforced sand pile at optimum volume content showed an improvement in bearing capacity of about 76% in comparison with the unreinforced condition. Also, the improvement in bearing capacity was about 36% when soil reinforced with sand pile and 3D elements was compared with soil reinforced with sand pile only.
- Similarly for soft soil reinforced with pentapod and sand pile, the improvement in bearing capacity is 123% in comparison with unreinforced soil and 72% when compared to that of soil and sand pile only.
- Comparing the shapes, pentapods are more effective than tetrapods as reinforcing elements .There is around 26% additional improvement in bearing capacity of soil when pentapods are used instead of tetrapods as reinforcing elements in sand piles at optimum content.

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Geotechnical Properties of Lime and CKD Admixed Biosolids

M. T. Prathap Kumar, N. Karthik, Basavalinga, M. N. Annappa Hemanth, and D. R. Nagesh Kumar

Abstract The amount of waste generated due to impact of urbanization has increased worldwide. To evolve sustainable use of materials, it is imperative to use treated waste materials as a good step in waste management. The objective of present investigation is to verify the improvement in geotechnical properties of biosolids which is one of the waste materials generated from wastewater treatment plant. In the present study, the biosolid was admixed with different percentage of lime. The optimum lime content (OLC) was determined based on compaction characteristics. To further improve its geotechnical properties, cement kiln dust (CKD) was added along with OLC, and various geotechnical properties were determined. It was found that the blended biosolids with 20% OLC along with 10% CKD showed significant improvement reduction in liquid limit, increase in shear strength and CBR, indicating its potential application as a subbase material in road construction.

Keywords Biosolids · Cement kiln dust · Optimum lime content

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1 Introduction

Sludge formed in the sewage treatment plants is the unavoidable results of treating the domestic sewage and industrial effluent. Failure to regularly remove the sludge from the sewage treatment results in the work failing, having an adverse effect on receiving watercourse. Management of the sludge requires a secure outlet. Increasing legislation and environmental pressures on conventional sludge disposal has led to the high-tech sludge processing and treatment solutions throughout in the world. The production of large number of biosolids generated during the treatment of wastewater presents an enormous challenge. Historically, biosolids have been disposed using practices such as ocean disposal, landfill and incineration. These options are becoming less viable due to stricter environmental regulations and the economic cost associated with these disposal options. More recently, options for the use of biosolids for beneficial purpose have been explored. Biosolids are rich in organic matter and nutrients, and therefore land application has been advocated as a sustainable option for resource recovery. In many advanced countries, biosolids have been used for land application in agriculture (vine, cereal, pasture, olive), co-generation/power production/energy recovery and road base. Some of the recent studies have explored use of stabilized biosolids for many infrastructure applications. Biosolids are rich in both organic matter and essential plant nutrients and can be utilized in a variety of ways, directly as a soil amendment and fertilizer and failure to regularly remove the sludges from the sewage treatment works has an adverse effect on the receiving watercourse [1]. It has been investigated that the addition of biosolid and lime sludge undermines the strength of landfill and has been found that landfilling biosolids offer competitive cost and, in fact, is cheaper than some alternative disposal option [2]. The effects of untreated biosolids if used for agricultural activities transmit many pollutants to environment and create many hazards for public health. However, addition of lime to stabilize biosolids reduces the fecal coliforms more than 99.99% and hence can be used for reconditioning the poor soil and are for covering of solid waste landfill site [3, 4]. Several field and laboratory studies have been reported in literature to assess the geotechnical properties of biosolids and viability of using biosolids as stabilized fill for road embankments [5–7]. Review of several literatures has indicated beneficial uses of cement kiln dust (CKD), which is a by-product waste material of the cement manufacturing process. Because of high lime and potassium concentration, CKD can be used as a soil amendment or fertilizer. Use of CKD in stabilizing biosolids results in reduction of concentrations of heavy metals within acceptable international limits [8–10]. Tests conducted on poor soil conditions using CKD as stabilizer results in inadequate strength and bearing capacity of soil. Further, native soil properties like compaction characteristics, plasticity, CBR and permeability can be improved using CKD. However, use of CKD as a stabilizer for assessing geotechnical properties of biosolids is scarce in literature. The objective of the present study is to assess geotechnical properties of biosolids stabilized with optimum lime content as well as cement kiln dust (CKD). Lime is a well-known stabilizer for many expansive soils and hence suitable for stabilizing biosolids in terms of reduction in volume change capability of biosolids. Hence, along with lime, CKD has capability to induce better cementation and hence increase strength of stabilized biosolids making it suitable for pavement subbase applications. A laboratory investigation of stabilized biosolids using optimum lime and CKD content was conducted, geotechnical properties of biosolids were assessed in terms of plasticity, and shear strength in terms of curing period and its possible applications as a subbase material has been assessed in terms of CBR.

2 Materials and Methods

2.1 Materials Used

Biosolids Used: For the present study, the biosolids were procured from wastewater treatment plant located at Mylsandra, Kengeri, Bengaluru which has a capacity of treating 75 ml per day (MLD). Chemical analysis of biosolids indicated presence of arsenic, cadmium, copper, iron, magnesium, lead and other heavy metals within permissible hazardous limits. Table 1 summarizes index properties of biosolids determined in the laboratory. Figure 1 shows XRD analysis of biosolids alone that indicates peak occurring at 2θ in the range of 25–27° indicating the presence of bentonite and low quartz.

Table 1 Index properties of biosolids

S. No.	Particulars	Value
1	Grain size analysis	
	% of gravel	0
	%of sand	82
	% of silt	17
	% of clay	1
2	Specific gravity	1.62
3	Plasticity properties	
	Liquid Limit (LL) percentage	104
	Plastic Limit (PL) percentage: Not possible to roll	
4	Compaction characteristics (heavy compaction test)	
	Maximum dry density kN/m^3	8.25
	Optimum moisture content percentage	57
5	P^H value	6.85
6	Organic content in % (LOI Test)	43

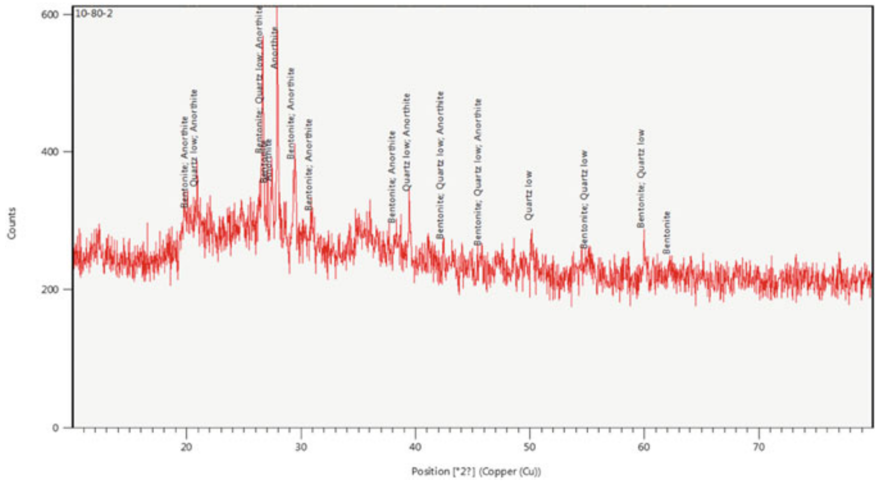


Fig. 1 XRD analysis of biosolids alone

Hydrated Lime: Class F hydrated lime ($CaOH_2$) is used in the present study which was procured from *SD Fine-Chemicals Limited; Mumbai*, India. The chemical analysis and constituent of hydrated lime ($CaOH_2$) used in the present investigation indicated the presence of CaO at around 85% by mass of lime used.

Cement Kiln Dust: The cement kiln dust is brought from the ACC cement production plant, which is placed at Wadi of Gulbarga district, Karnataka, India. Chemical analysis of CKD indicated greater absorption of alkalies and chloride which is useful for biosolid stabilization. The sulfate content lies within the acceptable limits with CaO at around 49.3% by weight.

2.2 Methodology

Optimum lime content and optimum CKD along with lime were determined based on heavy compaction test by mixing lime alone to biosolids in percentages varying from 5, 10, 15, 20 and 25% and CKD along with optimum lime content in percentage varying from 5, 10 and 20%. The optimum percentage was thus determined based on maximum dry density (MDD) with minimum optimum moisture content (OMC).

Shear strength of biosolids alone, stabilized biosolids using optimum lime as well as optimum lime and CKD were conducted using direct shear test on remolded specimen. The remolded specimen was prepared using corresponding MDD and OMC obtained from compaction test for respective configuration used in the present study. The effect of curing was studied by keeping the prepared remolded specimen in plastic cover and kept in desiccator for 0, 7, 14 and 28 days. CBR test on biosolids alone and stabilized biosolids for optimum configuration was obtained based on shear strength test in 4 days soaked condition after curing the specimen for a period of 24 h with optimum lime and CKD content to assess various geotechnical properties and arrive at conclusions on its suitability as a subbase material.

3 Results and Discussions

3.1 Determination of Optimum Lime Content (OLC) and CKD

Maximum dry density (MDD) and corresponding optimum lime content (OMC) were obtained using heavy compaction test on biosolids alone and by adding lime varying from 5, 10, 15, 20 and 25%. The OLC was arrived at based on the maximum value of MDD and corresponding minimum value of OMC as shown in Fig. 2. It was found that biosolids mixed with 20% lime content are found to be optimum. Using the OLC thus obtained, biosolids mixed with 20% OLC were further mixed with different percentage of CKD varying from 5, 10, 15 and 20%.

As shown in Fig. 3, it was found that 10% CKD was optimum when it is used in conjunction with biosolids plus 20% OLC. Table 2 summarizes the values of MDD and OMC obtained at OLC and optimum CKD obtained for biosolids admixed with OLC.

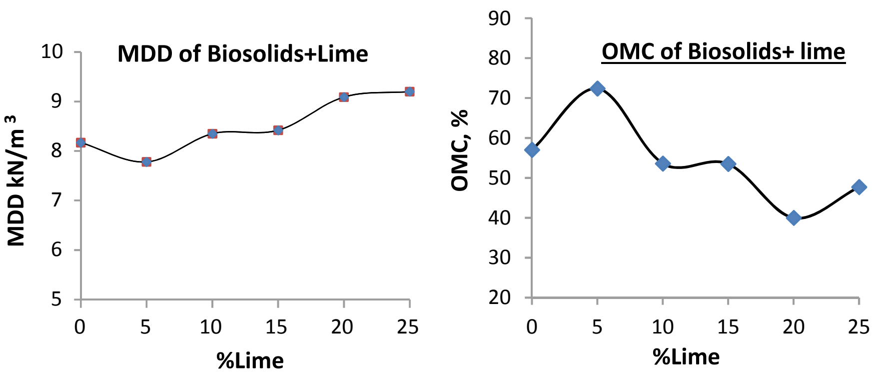


Fig. 2 Variation of MDD and OMC for biosolids +lime

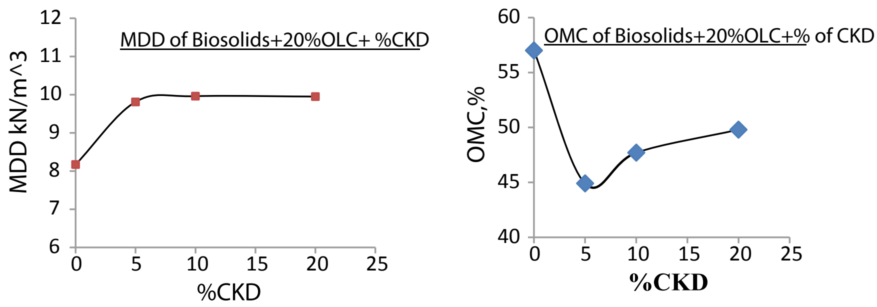


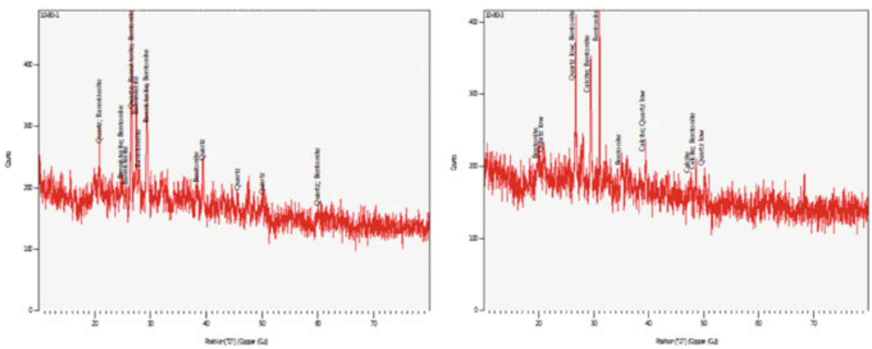
Fig. 3 Variation of MDD and OMC for biosolids +20% OLC +5% CKD

Table 2 Summarized values of maximum MDD and minimum OMC of biosolids admixed with lime and CKD

Particulars	MDD kN/m^3	OMC (%)
Biosolids alone	8.17	57
Biosolids +20% Lime	9.09	40
Biosolids +20% Lime +10% CKD	9.96	47.7

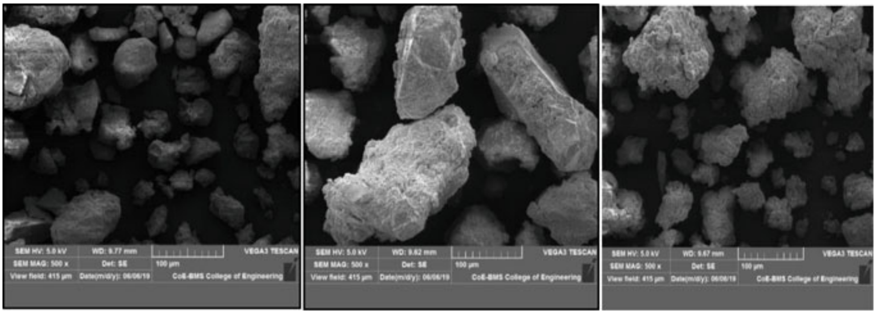
3.2 *Effect of OLC and CKD on Shear Strength Characteristics of Biosolids*

Figure 4a shows XRD of biosolids admixed with 20%OLC which indicated peak in the range of 2θ equal to 27–28° degree with presence of calcium oxide. Presence of CaO assists pozzolanic reaction and also causes agglomeration of particles. Figure 4b shows XRD of biosolids +20% OLC +10% CKD which indicated several peaks in the range of 2θ equal to 26–32°. The peaks indicate the formation of calcite with low



(a) Biosolids+20%OLC (b) Biosolids+20%OLC+10%CKD

Fig. 4 XRD analysis of biosolids stabilized with OLC and CKD



(a)Biosolids alone (b) Biosolids+20%Lime (c) Biosolids+ 20%OLC+10%CKD

Fig. 5 SEM images of biosolids alone and stabilized biosolids with OLC and CKD

quartz and hence better cementation, due to addition of CKD in combination with lime.

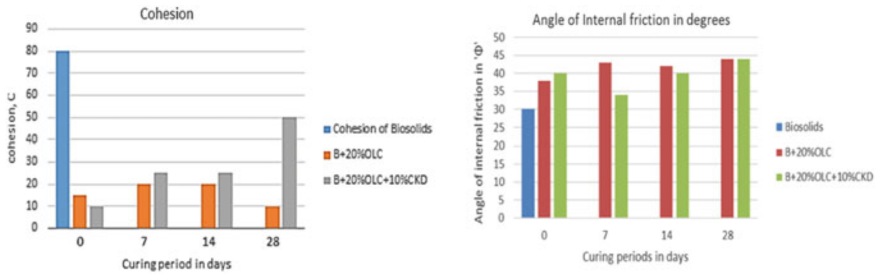
Figure 5 shows microstructural changes observed in particles of biosolids in comparison with biosolids admixed with OLC as well as biosolids admixed with 20% OLC +10% CKD at magnification of 500x particles of biosolids are rounded and small, whereas particles of biosolids +20% OLC indicate larger particles that are rough textured, which assist denser packing leading to higher compaction obtained in the experimental results, white patches indicate that the particles are coated with calcium ions. Addition of both optimum lime and CKD makes particles of biosolids coarser and more angular, and white patches around the particles indicate presence of calcite. These structural changes, hence, lead to better compaction that leads to increase in shear strength and reduction in liquid limit, since stabilization with lime and CKD has resulted into stable particulate matter suitable for many civil engineering applications such as road works and embankments.

Table 3 summarizes the values of liquid limit with the addition of optimum lime and CKD. Since the particle size analysis of biosolids indicated at around 82% sand sized particles, it was not possible to roll it to threads smaller than 5 mm and hence lacks plasticity. Further, addition of OLC reduces liquid limit and addition of CKD along with OLC causes further reduction in liquid limit which is desirable in many civil engineering applications.

Since the compacted density of remolded specimens of biosolids alone and stabilized biosolids with OLC and CKD falls in the range of 8.17–9.96 kN/m^3, direct shear

Table 3 Variation of LL of stabilized biosolids

Particulars	LL (%)	PL (%)
Biosolids alone	104	Not possible to roll
Biosolids +20% OLC	81.5	Not possible to roll
Biosolids +20% OLC +10% CKD	68	Not possible to roll



(a) Variation of Cohesion **(b) Variation of Angle of Internal Friction**

Fig. 6 Variation of shear strength parameters of stabilized biosolids

test was performed by remolding the specimens directly in the shear box, and shear strength parameters were evaluated in terms of effect of curing period by conducting undrained shear strength test. The specimens were mixed with OLC and OLC+CKD and cured in a desiccator by keeping the specimens in a plastic cover for a period of 0, 7, 14 and 28 days. The specimens were then remolded in the shear box corresponding to the MDD and OMC obtained from compaction test.

Figure 6a, b shows variation of shear strength parameters 'cohesion, c in kN/m^2 and angle of internal friction, Φ in degrees' in terms of curing period obtained from direct shear test on stabilized biosolids in comparison with biosolids alone. Figure 6a shows that the addition of lime decreases significantly the cohesion of biosolids. However, with increase in curing period and addition of CKD, cohesion increases marginally at 7 and 14 days and significantly at 28 days curing. This is because, addition of CKD increases availability of calcium cations causing significant base exchange and hence induces better cementation than lime alone, as evidenced in XRD and SEM analysis presented earlier. Figure 6b shows similar such variation of Φ with curing period. In the initial stages of addition of lime as well, Φ value increases with in curing period. Further there is consequent reduction in Φ values. It can be attributed to the fact that introduction of finer CKD particle increases surface area and with increase in curing period better cementation and hence reduction in Φ. However, at 28 days curing, it seems to attain equilibrium indicating almost same value of Φ due to completion of pozzolanic reaction. XRD analysis and microstructural changes presented in SEM images have substantial evidence in corroborating the trend in variation of shear strength. The overall trend thus indicates increase in shear strength due to addition of lime and well as addition of CKD which is necessary for biosolids to be used as a construction material in many applications in the field of civil engineering.

Table 4 Summarized CBR values of stabilized biosolids

Particulars	CBR (Soaked)	
	CBR at 2.5 mm	CBR at 5 mm
Biosolids alone	1.52	1.82
Biosolids + 20% Lime	5.83	5.23
Biosolids + 20% Lime + 10% CKD	4.11	4.36

3.3 CBR of Stabilized Biosolids

Road work constructions demand use of large quantities of construction materials such as aggregate and filled materials. Use of stabilized biosolids in roadwork applications can significantly affect the economy of road construction. The result on stabilized biosolids has clearly indicated increase in density of compaction, decrease in liquid limit as well as increase in shear strength. In order to verify the applicability of biosolids in roadwork application, soaked CBR test was conducted by compacting the biosolids along with OLC and OLC+CKD at respective MDD and OMC. The biosolids mixed with lime as well as CKD was cured for 7 days and then soaked for four days before the CBR test was done. The CBR value was comparatively assessed with respect to CBR value obtained for biosolids alone. The CBR at 2.5 and 5 mm penetration was used for comparison purpose. Table 4 shows comparative variation of CBR of stabilized biosolids admixed with lime as well as lime+CKD. CBR of biosolids stabilized with OLC have indicated at around 280% increase in CBR. Further biosolids admixed with 20% OLC +10% CKD have indicated an increase of around 170% in the measured values of CBR. Thus, the results indicate that the stabilized biosolids have soaked CBR value in excess of 4% which is desirable and enough to recommend its use as a subbase material with a potential to be used in roadwork based on material performance measures.

4 Conclusions

On the basis of the present experimental study on stabilized biosolids with OLC and CKD, the following major conclusions have been drawn:

- Addition of lime increases dry density and decreases optimum moisture content. However, the MDD of biosolids alone was found to be 8.17 kN/m^3 at OMC of 57%. The values of both bulk and dry density obtained in the compaction test were low due to low particle density of the biosolids.
- SEM studies indicated the presence of calcium ions in both lime as well as CKD that induces cementation and makes particles to agglomerate resulting in the reduction of available surface area, contributing to denser packing of the admixture.

- The structural changes due to the addition of both lime and CKD making particles of biosolids more coarser and angular assist compaction, increase shear strength, reduce liquid limit with a consequent increase in CBR making biosolids more stable to be used for many civil engineering applications such as roadworks and embankments.
- A reduction of 21.6% in LL occurs due to addition of optimum lime and around 35% reduction in LL occurs due to addition of optimum lime along with optimum CKD.
- Addition of both lime as well as lime+CKD reduces cohesion with consequent increase in angle of internal friction due to the transformation of particles into angular and large sized, due to addition of lime as well as CKD.
- Increase in curing period with the addition of CKD increases cohesion at 7 and 14 days marginally and significantly at 28 days curing. Additional availability of calcium cations from CKD causes significant base exchange CKD, inducing better cementation than lime alone. At 28 days curing, Φ value seems to attain equilibrium indicating completion of pozzolanic reaction.
- Comparative variation of CBR of stabilized biosolids admixed with lime as well as lime + CKD indicated at around 280% increase in CBR due to addition of lime alone. Further biosolids admixed with 20% OLC +10% CKD have indicated an increase of around 170% in the measured values of CBR. The results suggest that stabilized biosolids have the potential to be used in roadwork based on material performance measures.

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Effect of Soil Slope on Failure Mechanism of Soil-Nailed Structures by Aluminum Nails and Bamboo Nails

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Abstract In the present day scenario, improvement of ground is necessary in various occasions due to wide range of construction requirements. Various ground improvement techniques have been developed over the past few years. Increasing the load carrying capacity by inserting steel bars generally termed as soil nails is one of the effective techniques. These are mostly used in improvement of soil slopes. Wide range of materials can be used as soil nails. In the present study, hollow aluminum tubes and bamboos were used as soil nails for improving the ground characteristics. Model tests were performed for soil slope with different conditions of nail inclination. Further, these test results are compared with unreinforced soil. Parameters considered for the study are nail inclination and soil slope. Three nail inclinations are considered for the present study; they are 0°, 15°, and 30° with horizontal axis and two soil slopes they are 45° and 60°. Constant parameters considered for the study are soil, height, nail length, and nail pattern. The results obtained are compared with the conventional unreinforced soil slope for each case and curves for load versus settlement were developed for the same. From experimental results, soil slope with 0° nail inclination with horizontal axis gives the maximum load carrying capacity in all the cases, followed by 15° nail inclination with horizontal axis and then 30° nail inclination with horizontal axis.

Keywords Backfill · Reinforced soil · Soil nail · Unreinforced soil

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1 Introduction

Soil nail has gained popularity in recent years in the construction industry due to its ease of construction, technical stability, and relatively free of maintenance. This technique is used several construction activities such as slope stability by reinforcing the soil with vertical and inclined elements [1, 2] for protection and preservation of historical monuments [3], etc. The shear strength of clayey soil has been investigated worldwide by soil nailing technique using a variety of reinforcing elements in the form of closely spaced steel bars called nails as mentioned by several researchers (e.g., Casagrande (2006); Dermatas and Mang (2003); Freilich et al. (2010); Indraratna (1996); Maher and Ho(1994); Naeini et al. (2012) [4–9]). This technique is not suitable for soft clays due to low cohesion of soft clay which leads to small friction between the soil and soil nails soft soils properties can be improved by combined method of fracture grouting and soil nailing techniques [10, 11]. The shear strength of cohesive soils can be increased and the settlements are reduced by the utilization of soil nailing technique [12].

Soil nailing is a realistic and confirmed technique used in constructing excavations, reinforcing slopes, and solving geotechnical foundation problems by reinforcing the ground with relatively small, completely bonded inclusions, typically steel bars [13]. The behavior of slopes and excavations using the field or experimental tests [14–19]. It is noticed from the results of various studies that initialization of soil nails provides considerable changes in soil in the vicinity of nails and improves the shear strength with in the soil mass. Dai et al. [20] mentioned an alternative method using Moso bamboo. The bamboo elements were employed as soil nails and piles using laboratory and field studies. The tests showed that the load capacity of bamboo nails is significantly increased by 250% compared with steel pipe nails. Garg et al. [21] introduced a soft computing method called multi-gene genetic programming, which is used to predict the factor of safety for different soil properties of three-dimensional (3D) soil nailed slopes.

It has been reported by many researchers that the increase in shear strength and decrease in settlements of cohesive soils can be achieved by soil nailing techniques. In the present work, an attempt is made to study the load settlement behavior of cohesive soils with and without reinforcement in the form of nails. Two types of nails (aluminum and bamboo) were used to study the load settlement behavior of cohesive soils.

2 Materials

Details of various materials used during the experimentation are reported below.

2.1 Soil

The soil used in the present study was collected from Godavari Institute of Engineering Technology (A) Campus Rajahmundry. The soil properties obtained from laboratory tests are specific gravity 2.68, grain size distribution (Gravel 52%, Sand 18%, silt and clay 30%), maximum dry density 19.5 kN/m^3, OMC 12.5%, liquid limit 33%, plastic limit 19%, plasticity index 14%, cohesion 48 kN/m^2, and angle of friction (Ø) 6.0°.

2.2 Aluminum Tubes

Hollow tubes of cross-sectional area 34.57 mm^2, 150 mm length, modulus of elasticity E 6.9×10^4 N/mm^2 were used as nails.

2.3 Bamboo Sticks

Bamboo sticks of same cross-sectional area, same length, and modulus of elasticity 1.68×10^4 N/mm^2 were also used as nails.

3 Experimental Study

A model box of dimensions 50 cm × 22 cm × 35 cm is fabricated by using 6 mm thick glass.

3.1 Model Tests

Tests were carried out by preparing two soil slopes such as 45° and 60° in model box in the laboratory. A fine layer of red dye is used between the layers for identification of failure pattern of the slopes, a crest of 150 mm and base width of 500 mm is maintained for all the slope angles (Fig. 1).

Fig. 1 Model box with 45° soils and with nails for test

3.2 Unreinforced Soil Model: Model-1

The prepared soil model is mounted on the testing machine; a bearing plate of size 15 cm by 15 cm × 0.6 cm is placed on the crest slope for uniform load distribution. Load is applied gradually at a rate of 10 N/s and the corresponding settlements were recorded by attaching two dial-gauges at the top of the bearing plate.

Reinforced with aluminum nails: Model-2

The soil model was reinforced with aluminum nails at 0°,15°, and 30° inclination with horizontal in a square (10 cm × 10 cm) nail pattern with 3 rows and 2 columns and is tested similarly as model-1.

Reinforced with bamboo nails: Model-3

The soil model was reinforced with bamboo nails at 0°, 15°, and 30° inclination with horizontal in a square (10 cm × 10 cm) nail pattern with 3 rows and 2 columns and is tested similarly as model-1.

Load tests were conducted on unreinforced soil model and reinforced soil model in the laboratory till the failure occurs (Fig. 2).

4 Results and Discussions

The results obtained from laboratory experimentation were tabulated and are discussed Tables 1 and 2.

Figures 3, 4, 5 and 6 depict the load settlement curves for different soil models constructed with 45° soil slope and 60° soil slope with aluminum and bamboo nails at different nail inclinations.

For 45° soils slope, the load carrying capacity is increased by 40.7% for 0° nail inclination, 29.6% for 15° nail inclination and 11.11% for 30° nail inclinations and

Fig. 2 Model box under load test

Table 1 Failure load versus settlement curves of 45° soil slope for different aluminum and bamboo nail inclinations

Model	Failure load (N)	Settlement (mm)	Nail inclination
Model-1	2025	8.05	No reinforcement
Model-2	2850	7.00	0°
	2625	7.42	15°
	2250	7.805	30°
Model-3	3000	6.685	0°
	2775	7.00	15°
	2475	7.70	30°

settlement was decreased by 13.0%, 7.8%, and 3.0%, respectively, for aluminum nails with respect to unreinforced soil model.

For 60° soils slope, the load carrying capacity is increased by 39.0% for 0° nail inclination, 27.0% for 15° nail inclination and 15.1% for 30° nail inclinations and settlement was decreased by 12.5%, 7.1%, and 3.8%, respectively, for aluminum nails with respect to unreinforced soil model.

For 45° soils slope, the load carrying capacity is increased by 48.1% for 0° nail inclination, 37.0% for 15° nail inclination, and 22.2% for 30° nail inclinations and

Table 2 Failure load versus settlement curves of 60° soil slope for different aluminum and bamboo nail inclinations

Model	Failure load (N)	Settlement (mm)	Nail inclination
Model-1	1890	9.6	No reinforcement
Model-2	2625	8.4	0°
	2400	8.92	15°
	2175	9.24	30°
Model-3	2925	7.92	0°
	2625	8.28	15°
	2325	8.96	30°

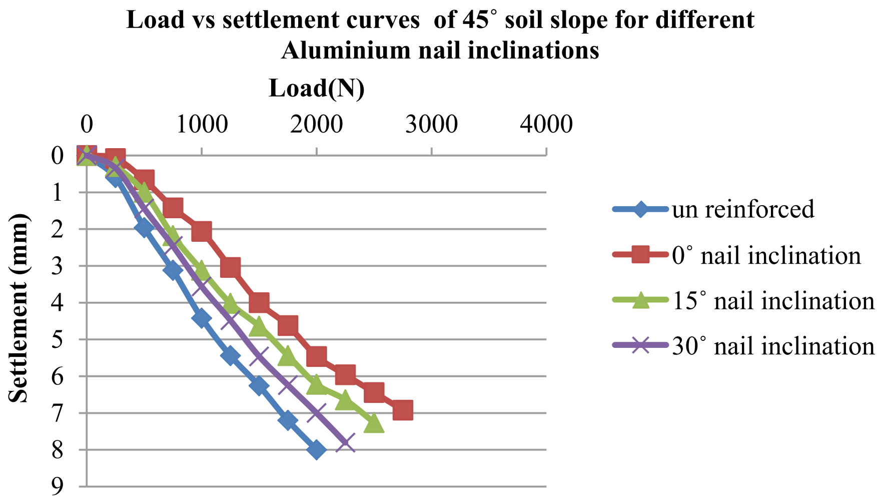

Fig. 3 Load versus settlement curves of 45° soil slope for different aluminum nail inclinations

settlement was decreased by 17%, 13%, and 4.3%, respectively, for bamboo nails with respect to unreinforced soil model.

For 60° soils slope, the failure load carrying capacity is increased by 54.8% for 0° nail inclination, 38.9% for 15° nail inclination and 23.0% for 30° nail inclinations and settlement was decreased by 17.5%, 13.75%, and 6.7%, respectively, for bamboo nails with respect to unreinforced soil model.

It can observed form the above figures, that the load carrying capacity has substantially increased for 0° nail inclination for both aluminum and bamboo nails compared to 15° and 30° nail inclinations. The improvement in load carrying capacity and the decrease in settlement could be attributed to the insertion of the reinforced elements into the soil mass.

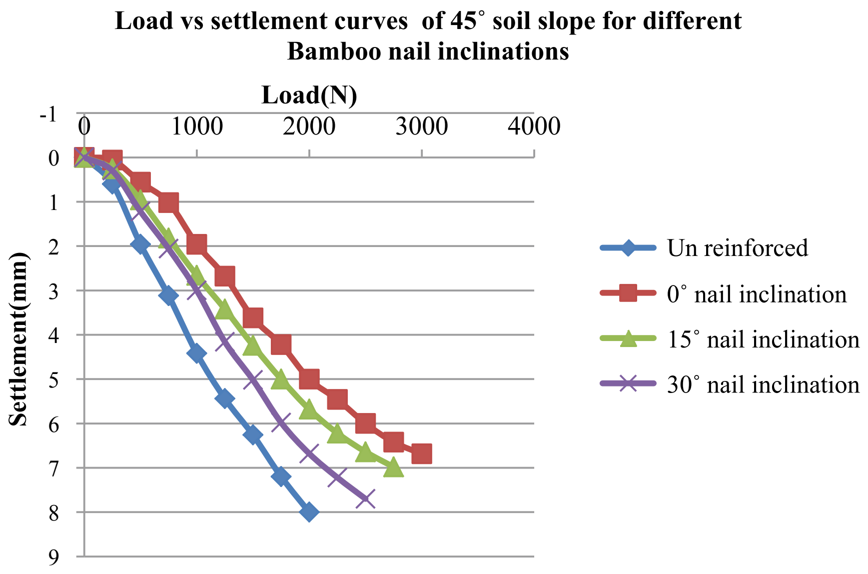

Fig. 4 Load versus settlement curves of 45° soil slope for different bamboo nail inclinations

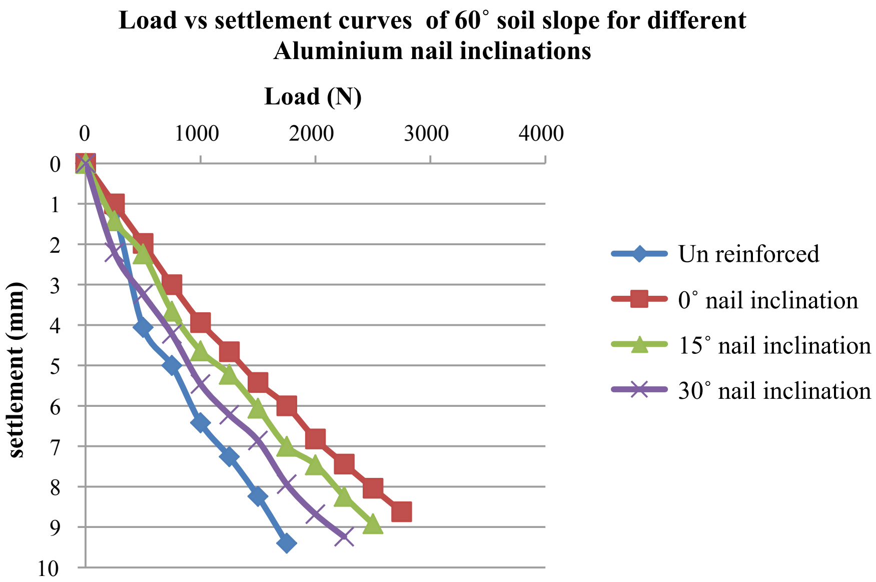

Fig. 5 Load versus settlement curves of 60° soil slope for different aluminum nail inclinations

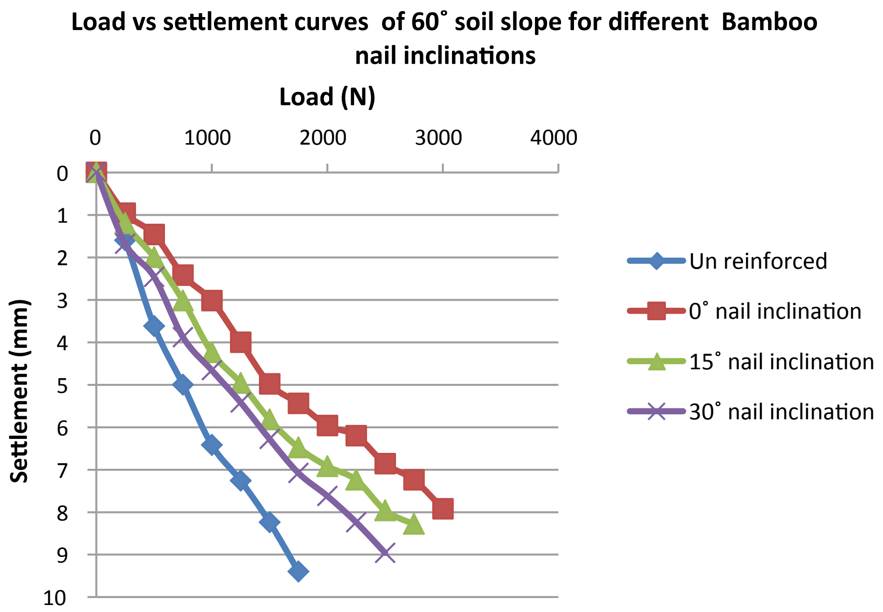

Fig. 6 Load versus settlement curves of 60° soil slope for different bamboo nail inclinations

5 Conclusions

1. It is observed that the 0° nail inclination is proved to be more efficient as it gives fewer settlements at a particular load compared to other nail inclinations.
2. For 45° and 60° soil slope, the load carrying capacity of 0° aluminum and bamboo nail inclinations has increased by 40.7%, 48.1% and 39%, 54.8% with respect to unreinforced soil model.
3. For 45° and 60° soil slope, the settlements of 0° aluminum and bamboo nail inclinations has decreased by 13%, 17% and 12.5%, 17.5% with respect to unreinforced soil model.
4. For 45° and 60° soil slope, the load carrying capacity of 0° bamboo nail inclinations has increased by 5%, 10.3% with respect to aluminum reinforced soil model at same inclinations, respectively.
5. For 45° and 60° soil slope, the settlements of 0° bamboo nail inclinations has decreased by 4.5% and 5.7% with respect to aluminum reinforced soil model at same inclinations, respectively.
6. Out of all nail inclinations, 0° nail inclination shows better performance in both load carrying capacity and in settlement for both soil slopes.

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Stabilization of Clays and Clayey Soils Using Polycom-A Polyacrylamide Additive

M. Padmavathi and V. Padmavathi

Abstract The aim of this study is to assess the benefits of using Polycom a polymer-based additive to improve various performance-related properties of clays and clayey soils. Polycom is a commercially available polymer, which has a wide range of applications related to the improvement of various properties of soils. Generally, the substance is used in road construction as a means of stabilizing the soil movement due to changes in moisture content. Studies by the University of Adelaide have indicated that the addition of Polycom to the clay had a significant influence on decreasing the permeability of the respective compacted clay liners (CCL). Three types of soils namely clay of high compressibility and two clayey sand samples are tested in the laboratory by adding Polycom-mixed water. Wet application method is used to stabilize the soils in which the dry Polycom powder is mixed with water that is to be applied to the soil to obtain the optimum moisture content. The Polycom reacts with the water such that a highly viscous solution forms. The effect of Polycom on engineering properties of soils is investigated by dry density vs moisture relationship, unconfined compressive strength (UCS), and direct shear tests. Results show that clay of high compressibility (CH) showed appreciable improvement in strength in terms of UCS with the stabilizer for different curing periods. The improvement in UCS is about 160% with respect to strength of untreated soil. Similarly for clayey sand (SC), the increase is about 150% and 40% in cohesion and angle of internal friction, respectively, for 5 days of curing with respect to untreated soil strength.

Keywords Soil stabilization · Polycom · Polymer based stabilizer · Clayey sand · Clay of high compressibility

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1 Introduction

Traditional additives such as lime and cement have been used all over the world to stabilize sealed and unsealed roads. Traditional additives have shown to improve soil workability, strength, durability and often lead to a reduction in the design thickness of the structural base layers. They are required in large quantities to be effective and require relatively long curing times. The compaction process also needs to be completed within a limited time period, which leads to significant construction costs. Another disadvantage is that shrinkage cracks associated with cement-stabilized layers which reflect rapidly through asphaltic surfaces. These disadvantages of traditional additives lead to the development of polymeric-based additives. Polymeric-based additives demonstrate many added advantages such as their ability to reduce permeability, increase durability, allow non-time dependence during the mixing stage and provide increased flexibility. The additive used in this study is a synthetic poly-acrylamide (PAM) additive called Polycom. Polycom has been used in Australia with positive field results [1]. This paper is focused on laboratory investigation on stabilization of clays and clayey soils by adding Polycom.

1.1 Polycom-Polymer-Based Soil Stabilizer

Various polymers have been proposed for polymer soil stabilization, including cationic, anionic and non-ionic polymers. Most chemical stabilizers react with soil in one of two ways: specific chemical reactions occur between soil particles and stabilizer, or the stabilizer provides physical stabilization through the use of binding agents. Polymers fall into the second category. Polymers are essentially binding agents that bind soil particles into a cohesive mass and subsequently improve the physical properties of soil. The interaction between soil and polymer is highly dependent on the properties of the polymer, i.e. type and amount of surface charge, polymer configuration, chain length, molecular weight and size as well as the soil properties. Soil properties that influence polymer stabilization are type and percentage of clay content, ionic strength of the soil solution, type of ion in solution and PH value [2, 3].

Polycom is a commercially available Australian-made soil polymer-based soil stabilizer, which has a wide range of applicability relating to the improvement of various properties of soils. Polycom stabilizing aid is used to strengthen almost any material commonly found in road construction and earthworks projects. The polymeric additive used in this study is a synthetic soluble anionic PAM [4]. It is produced in a granulated form and was developed in Adelaide, Australia, by Bio-Central Laboratories Ltd. The PAM has a moderate charge density (approx. 18%) and a high-molecular weight between 12 and 15 Mg per mole, which is equivalent to approx. 150,000 monomer units per molecule. Apparent viscosity increases by mixing Polycom with water. The product is a non-toxic water soluble material with a specific gravity of 0.8 and a PH value of 6.9 at 25 °C. Polycom delivers a stronger, more resilient pavement, improving the flexibility and workability of the materials to create a tighter, water resistant surface. Polycom has a pronounced effect on the behaviour of the pavement such that maintenance is dramatically reduced which of course reduces life cycle costs.

1.2 Mechanism of Stabilization by Polycom

After wet mixing Polycom into an area and then compacted, its action assists the compactive effort improving the natural mechanical interlock between particles which in turn produces tighter compaction of the subject material. This improved compaction means higher density and higher CBR results. Polycom does not 'cure' but after drying Polycom leaves behind a polymer bond between the material particles, it is this polymer bond which delivers flexibility to the stabilized pavement.

2 Review of Literature

Experimental study was carried out to assess the benefits of using a synthetic Polyacrylamide (PAM) additive to improve various performance-related properties for three types of pavement materials commonly used in the construction of unsealed roads in Australia [4]. The three materials selected for testing were a silty gravel, a clayey sand and a clayey gravel. The addition of PAM was found to increase the unconfined compressive strength and dry density for all soil types. Furthermore, the addition of PAM also changed the failure mode from brittle to ductile, which will naturally increase the fatigue life of the pavement. Improvement in tensile strength of the stabilized samples was also noted. Scanning electron microscopy analysis results revealed a decrease in the quantity of loose particles and pore volume, and an increase in contact points between particles for the treated samples, which further verified the mechanical and physical improvements gained by adding PAM to unbound pavement materials.

Soils treated with polymeric-based additives have shown to improve various physical characteristics [5, 6]. Results reported a significant improvement in strength for granular-type soils when treated with three different types of polymers after 28 days of curing. Furthermore, studies revealed that six polymer emulsions (acrylic vinyl acetate copolymer, polyethylene-vinyl acetate copolymer, acrylic copolymer, polymeric proprietary inorganic acrylic copolymer, acrylic vinyl acetate copolymer and acrylic polymer) all yielded an increase in strength and toughness values for a silty sand soil after 7 days of curing, which was similar in magnitude to a typical cement treatment after 28 days [6]. More importantly, the increase in toughness of a pavement layer translates to better flexibility at higher capacity and indicates that this layer would be less susceptible to sudden damage under repetitive axle loading.

Polymer binders effectiveness as stabilizers in terms of improving physical, chemical, mechanical and microstructural properties of a silty clayey gravel type soil is studied [7]. Recently, conducted studies by the University of Adelaide have indicated that the addition of Polycom to the clay had a significant influence on decreasing the permeability of the respective CCL [8]. Few case studies are presented on the use of Polycom as stabilizer both for sealed and unsealed roads [9, 10].

3 Methodology

Three different soils are used for experimental investigation by adding Polycom water, to find out the changes in the strength of the soil. Classification tests like sieve analysis, Atterberg limits are carried out for the three selected soil samples. Three selected soil samples are classified as clay of high compressibility (CH) and two soil samples of clayey sand (SC).

Polycom is available in dry powder form. Ploycom water is prepared by mixing 3 g of Polycom powder in 5 litres of water. Polycom water is used as substitute for water in compaction and CBR tests. OMC is found for each sample by conducting modified compaction test. Soil sample is now kept in incubator after mixing required amount Polycom water (equal to OMC) for curing. UCC tests are conducted for clay samples and direct shear tests are conducted for clayey sand (SC) samples. Tests are conducted for different curing periods of 1 day, 3 days and 5 days along with zero curing period or immediate test.

4 Results and Discussion

Results of classification tests carried out for three soil samples are given in Table 1.

The variation of UCS of clay for different curing periods with Polycom water is shown in Fig. 1. Untreated soil, i.e. without addition of Polycom is first tested for its compressive strength. Clay samples are compacted to maximum density after mixing with required amount of Polycom water (equal to OMC). UCS samples are then extracted and kept in desiccator for curing period. UCS increases with increase in curing period as mentioned in Table 2. The increase is about 116, 128 and 160% for zero, one day and 5 days curing periods, respectively, with respect to untreated soil. UCS is almost the same for 3 days and 5 days of curing periods.

Shear stress versus normal stress variation of Polycom-treated clayey sand is depicted in Fig. 2. Soil without addition of Polycom has a cohesion, c of 0.18 kg/cm^2 and an angle of internal friction of 37°. Cohesion of the soil increases with increase in curing period upon addition of Polycom. It increases by about 122% with respect to cohesion of untreated soil sample. The friction angle increases from 37° to about 50° which indicates a better improvement in overall shear strength of the soil. Table 2 presents the shear strength parameters obtained for Polycom-treated clayey sand (soil

Table 1 Index properties of three soil samples used for study

Sample No.	Sample identification	Modified proctor compaction		Atterberg llimits (%)			Particle size distribution (%)					
		OMC (%)	γ_{dmax} (t/m^3)	Liquid Limit (%)	Plastic Limit	Plasticity Index	Gravel (> 4.75 mm)	Sand: Coarse (2.0–4.75 mm)	Sand: Medium (0.425 – 2.0 mm)	Sand: Fine (0.075 – 0.425 mm)	Silt and lay (< 0.075 mm)	Classification
1.	Soil sample -1	23	1.6	73	31	42	0	0	0	9	91	CH
2.	Soil sample -2	11.5	1.95	31	18	13	0	20	42	16	22	SC
3.	Soil sample -3	8	2.13	34	16	18	10	8	33	15	34	SC

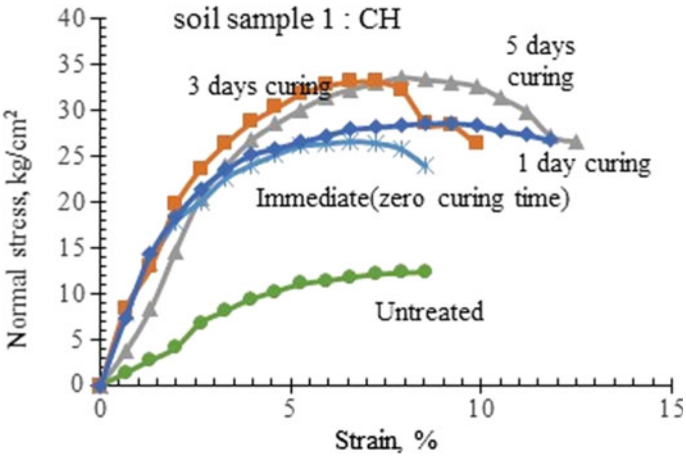

Fig. 1 Comparison of UCS of Polycom-treated clay of high compressibility (CH)—soil sample-1 for different curing periods

Table 2 UCS values for Polycom-treated clay soil (soil sample-1) for different curing periods

Soil treatment specification	Untreated	Immediate (without curing)	1 day curing	3 days curing	5 days curing
Unconfined compressive strength (UCS), kg/cm^2	6.25	13.5	14.25	16	16.25

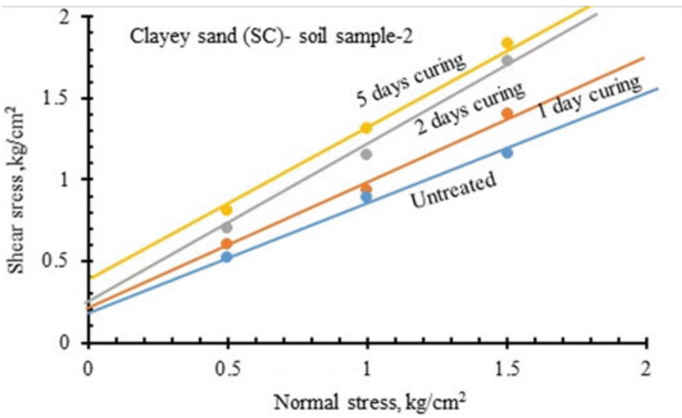

Fig. 2 Shear stress vs normal stress variation of Polycom-treated clayey sand (SC)—soil sample-2 for different curing periods

Table 3 c, ϕ values for Polycom-treated clayey sand (soil sample-2) for different curing periods

Soil treatment specification	Untreated	Immediate (without curing)	3 days curing	5 days curing
Cohesion (c), kg/cm^2	0.18	0.2	0.25	0.4
Angle of internal friction, (ϕ)	37°	43°	48°	50°

sample-2). Detailed investigations are required to find the optimum design parameters like curing period, compaction energy, percentage of polycom, etc., for maximum benefit (Table 3).

Unlike the untreated samples, the effective bonding in Polycom-treated samples was significant. It is believed that the polymeric stabilizer was able to bind the granular particles effectively due to the relatively small amount of fines and specific surface area. In other words, Polycom molecular encapsulates the soil particles and upon drying leaving an elastic membrane structure, which acts as a damper sheet that made the application of high compactive effort possible. Also, this bonding increased the contact points between the soil particles, resulting in further increase in frictional resistance force [4]. The Polycom molecules were able to coat most of the clay particles in the clayey sandy soil, and increase the cohesion and internal friction forces. This can be attributed to the nature of the soil gradation and fines contents. Clayey sandy soil consists of adequate proportions of particle sizes that increase inter-particle contact area among large and fine particles and hence, enhancing the bonding action of the polymer-treated soils [4].

Figure 3 depicts the variation direct shear test results of clayey sand (soil sample-3) for different curing periods. Similar variation as depicted in Fig. 2 is observed but

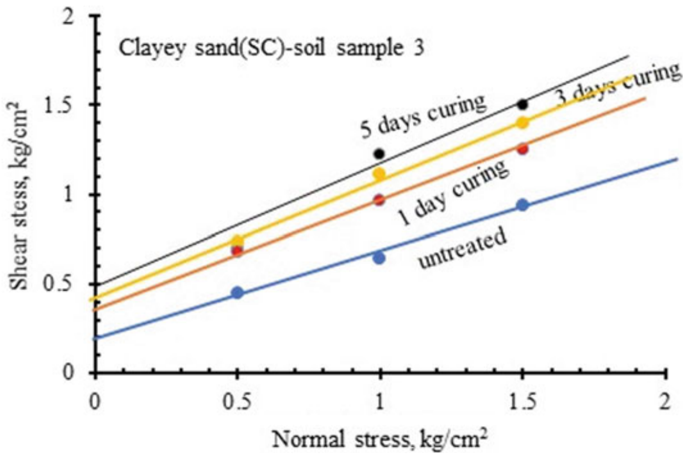

Fig. 3 Shear stress vs normal stress variation of Polycom-treated clayey sand (SC) sample—soil sample-3 for different curing periods

Table 4 c, ϕ values for Polycom treated clayey sand (soil sample-3) for different curing periods

Soil treatment specification	Untreated	1 day curing	3 days curing	5 days curing
Cohesion (c), kg/cm^2	0.2	0.35	0.4	0.5
Angle of internal friction, (ϕ)	32°	39°	43°	45°

with little variation in percentage of improvement in both cohesion, c and angle of internal friction, ϕ. Untreated soil possess cohesion of 0.2 kg/cm^2 which increases to 0.35 kg/cm^2, 0.4 kg/cm^2, 0.5 kg/cm^2 for 1 day, 3 days and 5 days of curing period respectively (Table 4).

5 Limitations of the Study

Paper presents the results of soil samples treated with Polycom, a polymer-based additive. Polymer-based additives are better substitution for traditional additives like lime, cement, etc. Tests are conducted on limited number of soil samples and the conclusions are drawn based on the results of three soil samples. Detailed investigations are required to confirm and conclude with benefits of using the polymers for soil stabilization.

6 Conclusions

1. Polycom increases the strength of clayey soil to almost double the in situ soil strength with no curing time. Curing period influences the gain in UCS for clayey soils. UCS increases with increase in curing period. Detailed investigation is required to find the optimum curing time to achieve maximum UCS.
2. Both cohesion and angle of internal friction have increased for Polycom-treated clayey sand samples. The increase is attributed to Polycom molecular encapsulation around each soil particle and forms a coat on most of the clay particles which facilitates the process of compaction. The increase in cohesion and angle of internal friction is about 150% and 40%, respectively.

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An Experimental Investigation of Properties of Black Cotton Soil Treated with Copper Slag and Groundnut Shell Powder

Pooja Pandya and Bhoomi Kamdar

Abstract Expansive soils have large volume changes due to the variation of water content. Therefore, expansive soils are also known as problematic soils due to its expansive nature. The black cotton soil has very low bearing capacity, high plasticity, high compressibility, low permeability and high swelling and shrinkage properties due to the presence of montmorillonite mineral. Due to these properties, black cotton soils are problematic soils. Therefore, it is necessary to improve its properties by using soil stabilization method. This study aims at conducting stabilization of black cotton soil of Bharuch region in Gujarat state by using combination of two soil stabilizing agents: copper slag and groundnut shell powder. In this research, copper slag is added 5% and 10% and groundnut shell powder is added 2% and 4%, respectively. The research gives comparison between the soil properties of non-treated and treated black cotton soil. Various laboratory tests are carried out such as specific gravity, liquid limit, plastic limit, shrinkage limit, free swell index, standard proctor compaction test, unconfined compressive strength test for both the cases by taking IS:2720 as reference. These laboratory tests results can be helpful to engineers to carry out soil stabilization on field during construction.

Keywords Black cotton soil · Copper slag · Groundnut shell powder

1 Introduction

Black cotton soil is a one type of expansive soil and is one of the major deposits of India. Black cotton soil is found in Gujarat, Maharashtra, Uttar Pradesh, Madhya Pradesh, Andhra Pradesh, Tamilnadu and Karnataka. Black cotton soil is produced

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geologically by the decomposition of volcanic rock and is very rich loamy earth of great productiveness and abnormal power of retaining moisture. Black cotton soil is a one type of clayey soil because of presence of montmorillonite mineral in this soil which expands when it comes in contact with water and it shrinks when the water is evaporated. Therefore, black cotton soil expands during monsoon and shrinks during summer season due to intake of water, respectively. Black cotton soil has large volume changes due to the variation of water content. The soil indicates high shrink-swell characteristics with surface cracks, opening during the dry seasons which are more than 50 mm or more wide and several mm deep. These cracks close during the wet season and an uneven soil surface is produced by irregular swelling and heaving [1]. The colour of this soil varies from deep black to light black due to presence of iron and aluminum compounds. The soil has low bearing capacity, low permeability, high plasticity and high swelling and shrinkage properties. In south Gujarat, black cotton is found in Bharuch, Valsad, Vadodara, Surat, and Junagadh as deep black cotton soil. The heavy loaded structures are most susceptible to damages as a result of volume changes. Because of the swelling and shrinkage characteristics of soil, therefore, treatment of the soil or special design needs to be adopted. To improve properties and increase the strength of soil, soil stabilization method is used. The main aim of soil stabilization is to improve the load bearing capacity, shear strength, foundation by using materials and chemicals or by using proportions and the addition of suitable admixture like stabilizers. The copper slag and groundnut shell powder are two soil stabilizer agents which are used to improve strength of soil. Waste and industrial byproducts of copper utilize in the soil stabilization of civil constructions. Copper slag can be used as a building material, formed into blocks. Copper slag is widely used as an abrasive media to remove rust, old coating and other impurities in dry abrasive blasting due to its high hardness (6–7 Mohs), high density (2.83.8 g/cm^3) and low free silica content [2]. Groundnut contains about 25% protein and 45–50% oil. Groundnut shells contain high cellulose (37%), hemi cellulose (18.7%), Lignin (28%) and carbohydrates (2.5%) content, which increase the efficiency of fermentation and provide better yield [3]. Lignin prevents the absorption of water which used as coatings, agricultural chemicals, micronutrients, natural binders, adhesives and resins [4]. In this research, to used copper slag (CS) and groundnut shell powder (GSP) to improve the engineering performance of black cotton soil which may be an economical solution of soil stabilization.

Copper slag and steel slag are used as stabilizing agents in the stabilization of black cotton soil. The specific objectives of this study to determine the index and engineering properties of the soil samples and determined the optimum % of copper slag and steel-slag content required. All tests were carried out in accordance with the procedures outlined in BS 1498 (1970) for the natural and treated soils, respectively [5].

The UCS of the treated black cotton soil with groundnut shell ash did not improved properties of soil [1]. Alkaline-treated groundnut shell powder has been shown to improve the mechanical properties of GSP-recycled polyethylene composites, with a treated GSP of 20% weight fraction recycled polyethylene composite having the highest mechanical properties. This treated sample also has a lower rate of water absorption and this shows that composites produced with alkaline-treated GSP and smaller particles are better materials for their intended applications [6].

1.1 Objectives of the Study

- To study the combined effects of copper slag and groundnut shell powder on properties of black cotton soil.
- To improve the engineering properties of black cotton soil.
- To reduce the plasticity of the black cotton soil.
- To determine the unconfined compressive strength of BC soil mixed with different percentages of copper slag and groundnut shell powder.

2 Materials and Methods

2.1 Materials

Black Cotton Soil: Black cotton soil is a one type of clayey soil because of presence of montmorillonite mineral in this soil which expands when it comes in contact with water and shrinks when the water is evaporated. The black cotton soil was collected at a depth of 2 m from Rahadpor village near Nandelav Part in Bharuch city. The Bharuch city located in South Gujarat area of India which having top layer of black cotton soil. Around 20 kg of black cotton soil was used. The properties of non-treated black cotton soil are as given (Table 1).

Copper slag: Copper slag is a by-product of copper release by smelting. During smelting, impurities become slag which floats on molten metal. Copper slag was collected from Nice Treading Company, Harni, Vadodara. 15 kg copper slag is used to conduct experiments (Fig. 1).

Copper slag is used passed through 600 microns IS sieve. The proportion of copper slag mixed with black cotton soil is in the range of 5% and 10% (Table 2).

Groundnut shell Powder.: The groundnut shell powder is produced by the best grade ground nut shell. It contains high level of nutrients like Ca, Mg, K, P, Na, S and the micro-nutrients Mn, Cu, Zn, Mo, B, Cl and Fe. [8]. The chemical compositions of groundnut shell powder are contains organic matter (92%), ash content (3.8%), crude protein (5.4%), crude fat (0.1%), lignin (36.1%), hemicellulose (5.6%) and cellulose (44.8%) [9]. Groundnut shell powder was collected from KD Oil Mill, Jambuva, Vadodara. 10 kg groundnut shell powder is used to conduct experiments (Fig. 2).

Groundnut shell powder is used passed through 600 μm IS sieve. The proportion of groundnut shell powder mixed with black cotton soil is in the range of 2% and 4%.

Table 1 Properties of non-treated black cotton soil

Properties	Black cotton soil
Colour	Grayish black
Water content (%)	8.45
Specific gravity	2.36
Liquid limit (%)	60.33
Plastic limit (%)	23.91
Plasticity index (%)	36.42
Shrinkage limit (%)	3.60
Soil classification	CH
Free swell index (%)	62.33
Compaction test: Standard proctor test: Maximum dry density (g/cc)	1.529
Optimum moisture content (%)	23.9
Unconfined compressive strength (Kg/cm^2)	2.51

Fig. 1 Copper slag

Table 2 Chemical composition of copper slags determined by X-ray fluorescence [7]

wt%	Min	Max
Al_2O_3	0.01	18.9
CaO	0.15	21.9
FeO total	0.67	62.0
K_2O	0.01	4.83
MgO	0.09	6.45
MnO	0.03	6.55
Na_2O	0.01	4.31
SiO_2	9.82	70.7
TiO_2	0.1	11.8

Fig. 2 Groundnut shell powder

2.2 Methods

Various laboratory tests are performed such as specific gravity, atterberg's limits, free swell index, standard proctor compaction test and unconfined compressive strength with IS 2720 references. These tests are performed with non-treated soil and compared with treated soil with different percentage of proportions as (Table 3).

Specific Gravity: To perform specific gravity, 10 g of oven dried soil samples are taken and transfer it carefully to the density bottle. Take the weight of empty weight of density bottle (M_1), weight of dry soil with density bottle (M_2), weight of density bottle with dry soil and water (M_3) and weight of density bottle with water (M_4). For 93% BC soil (9.3 g), 2% groundnut shell powder (0.2 g), 5% copper slag (0.5 g), test were performed as per IS 2720-3(1980) [10]. The specific gravity is calculated as below:

$$G = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)}$$

Atterberg's Limits: To perform liquid limit and plastic limit, 300 g sample required for conducting the experiment for each proportions. For 93% of soil by weight (279 g), 2% groundnut shell powder (6 g), 5% copper slag (15 g) and for 86% of soil by weight (258 g), 4% of groundnut shell powder (12 g) and 10% copper slag (30 g) were taken which are passed through 425 microns IS sieve and mixed as per IS 2720-5(1985) [11]. Shrinkage limit test was performed as guidelines require in IS 2720-6(1972) [12].

Table 3 List of various proportions

Soil (% by weight)	Copper slag (% by weight)	Groundnut shell powder (% by weight)
100	0	0
93	5	2
86	10	4

Free Swell Index To perform this test, 20 g of oven dried soil specimens passing through 425 microns IS sieve and 10 g of soil specimen should be poured two glass cylinders of 100 ml capacity. One cylinder shall be filled with kerosene and the other with distilled water up to 100 ml mark. After 24 h to take the reading and determine the volume of soil in both cylinder. For this paper, 93% BC soil (18.6 g), 2% groundnut shell powder (0.4 g), 5% copper slag (1 g) and 86% BC soil (17.2 g), 4% groundnut shell powder (0.8 g), 10% copper slag (2 g) proportions were used to make a soil specimens and test were performed as per IS 2720-40(1977) [13].

Standard Proctor Compaction Test: Tests were performed as per IS 2720-7(1980) for standard proctor test. The sample is mixed thoroughly with soil, copper slag and groundnut shell powder. Water content is added from 10, 13, 16, 19, 22, 25 and 28% by weight of the sample. Then mix is placed in the mould and compacted in three layers and each layer was compacted using 2.6 kg rammer under a free fall of 310 cm. This process is continuing till the weight of soil decreases with increment of water added and a graph is plotted between dry density and moisture content [14].

Unconfined Compressive Strength Test: The aim of this test is to determine a compressive strength of soils as per IS 2720-10(1991) specifications. The soil specimen is prepared with adding optimum water content and weight of soil required based on maximum dry density [15].

3 Observations and Results

3.1 Properties of Treated Black Cotton Soil and Compared with Properties of Non-treated Black Cotton Soil

Specific Gravity:
See Table 4.

Free swell Index:
See Fig. 3.

Atterberg's Limit:
See Fig. 4.

Table 4 Specific gravity

Proportions	Specific gravity
100% BC soil, 0% GSP, 0% Copper slag	2.36
93% BC soil, 2% GSP, 5% Copper slag	2.09
86% BC soil, 4% GSP, 10% Copper slag	2.01

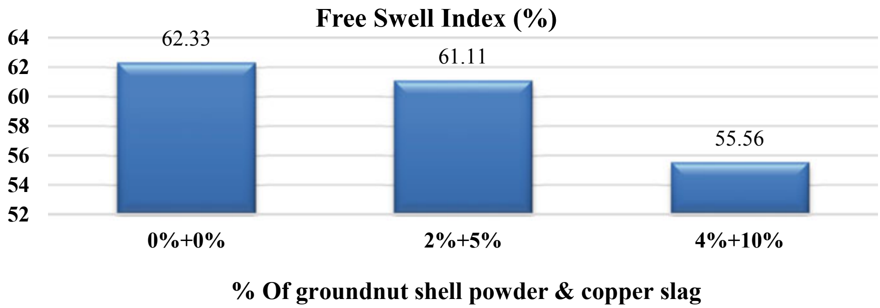

Fig. 3 Graph of free swell index

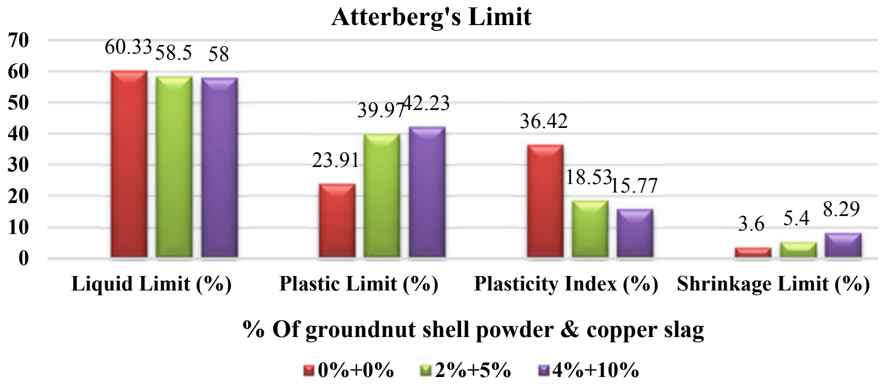

Fig. 4 Graph of Atterberg's limit

Standard Proctor Compaction Test:
See Fig. 5.

Unconfined Compressive Strength Test:
See Fig. 6.

4 Discussion

- The black cotton soil changes behaviour due to combination of groundnut shell powder and copper slag. The combination of groundnut shell powder and copper slag with black cotton soil gives low plasticity from black cotton soil inorganic clay of high plasticity. The plasticity of black cotton soil decreases with increasing in percentage of groundnut shell powder and copper slag.
- The behaviour of montmorillonite mineral may change due to breaking of water bond and removal of interlayer cations between clay particles, while it combines

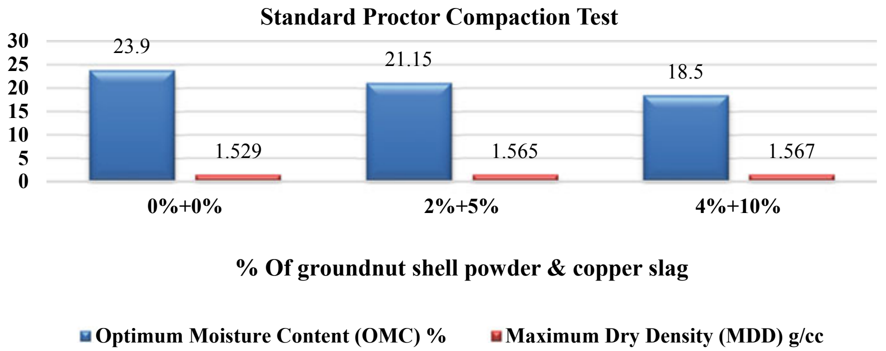

Fig. 5 Graph of standard proctor compaction test

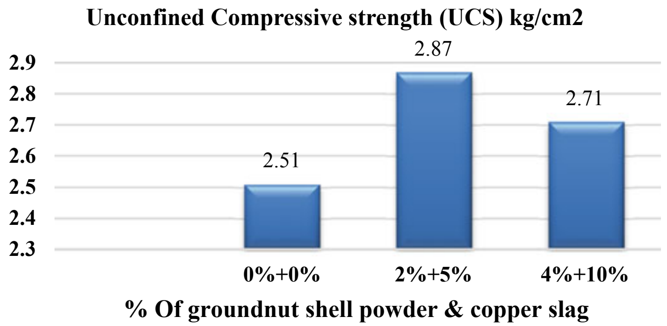

Fig. 6 Graph of unconfined compressive strength

with minerals present in copper slag and groundnut shell powder under hydration process.

- Lignin presents in groundnut shell powder which may give strength to bond particles of treated soil by act as a coating, natural binder or adhesive to the soil.

5 Conclusions

Based upon the results of experimental investigations and objectives of this research, following conclusions were obtained.

- The combined effect of copper slag and groundnut shell powder is to improve the engineering properties of black cotton soil.
- The plasticity of black cotton soil is decreases from 36.42 to 15.77% with increasing in percentage of copper slag and groundnut shell powder.

- The swelling index of black cotton soil is decreases from 62.33 to 55.56% with increasing in percentage of copper slag and groundnut shell powder.
- The shrinkage limit is varies from 3.60% to 8.29% with increase in percentage of groundnut shell powder and copper slag.
- The maximum dry density and optimum moisture content of non-treated black cotton soil is 1.529 g/cc and 23.9% determined but when 86% BC soil + 4% GSP + 10% copper slag is added in black cotton soil, the maximum dry density and optimum moisture content are increased up to 1.567 g/cc and 18.5% respectively. Use of groundnut shell powder and copper slag could be an economical and feasible solution to stabilize the black cotton soil due its economical cost.
- The unconfined compressive strength of black cotton soil is 2.87 kg/cm^2 at optimum percentage of 93%BC soil + 2%GSP + 5%copper slag.

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Experimental Investigation of Silty Soil Treated with Sodium Lignosulfonate

Abhijeet Gupta, Awdhesh K. Choudhary, and Anil K. Choudhary

Abstract Soil stabilization refers to the process of changing soil properties to improve strength and durability. There are many techniques for soil stabilization, including compaction, dewatering, and by adding chemicals to the soil. Out of these, chemical stabilization is one of the most effective and popular techniques, which has been practiced successfully in the field. There are several chemical additives such as lime, cement, fly ash, and rice husk. Most recently, lignin is an industrial by-product that has been identified as a chemical additive for stabilization of soil mass. Besides, lignin does not have any adverse effect on the environment. In view of this, the behavior of lignin-stabilized soil has been investigated in the present study. Results obtained from unconfined compressive strength tests indicate that the performance of lignin-stabilized soil increases with increase in percent of lignin content. However, it has been observed that the performance of stabilize soil reduces beyond 3% of lignin content. This is possibly because the soil particles completely get coated with lignin if it increased beyond 3%, thereby mobilizes strength at the surface of two lignin particles, which has lesser bonding strength than the strength mobilized at soil lignin interface. Therefore, it can be stated that the optimum percentage of lignosulfonate giving maximum performance of stabilized soil mass should be about 3% by weight.

Keywords Lignosulfonate · Soil · Unconfined compressive test

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1 Introduction

In India, the soil stabilization began in the early 1970s. There was a general lack of oil and aggregates. In view of this, engineers required to find the ways to improve soils instead of replacing poor soils on construction sites. Stabilization in the broadest sense involves various methods of modifying the properties of a soil to improve technical performance. Stabilization is used for various engineering works, most often in road construction, where the main objective is to increase the strength or stability of the soil and reduce construction costs by making the best use of available materials. Soil stabilization involves improving the technical properties of the soil and making it more stable. In the broadest sense, stabilization includes compaction, pre-consolidation, drainage, and many other processes. Soil stabilization is used to reduce the permeability and compressibility of soil mass in the soil structure and to increase its shear strength [1–5]. The main objective of the stabilization is, however, to improve the natural terrain for motorway construction and to make a territory practicable in a short time in the event of a military or other emergency. In view of this, the behavior of lignin-stabilized soil has been investigated in the present study through laboratory tests.

2 Materials Used

Locally available silty soil in Jamshedpur is found as highly erodible and dispersive in nature, which is a major problem regarding failure of earth surface such as embankments dam, rail or road embankments, canal banks, and foundation due to surface and internal erosion. In such situation, adopting a suitable ground improvement technique to control soil erosion is necessary to avoid damage and maintenance cost caused by such soils. To eliminate such major problems, chemical stabilization has been proven to be an appropriate and cost-effective method, however, the traditional soil stabilizers such as cement, lime, fly ash, slag, and gypsum have been identified to cause serious environmental problems like changing of pH of soil and groundwater, thus, negative impact on agriculture and aquaculture and these soil tend to exhibit excessive brittleness. In this context, lignin-based chemicals such as lignosulfonate have shown promising potential in stabilizing erodible and dispersive soil. The properties of silty soil and sodium lignosulfonate are presented in Tables 1 and 2, respectively.

Table 1 Properties of silty soil

Properties	Value
Specific gravity	2.28
Liquid limit	36.2%
Plastic limit	28.33%
Plasticity index	7.87
Soil classification	MI
OMC	19.55%
MDD (g/cc)	1.64
Free swell index	4.17%

Table 2 Properties of sodium lignosulfonate

Properties	Value
pH (10% solution)	4.9
Sodium	6.0%
Total sugar	3.0%
Color	Brown
Moisture	≤7.0%
Bulk density	635 kg/m^3

3 Methodology

3.1 Unconfined Compressive Strength Test

The unconfined compression test is a special type of unconsolidated-undrained (UU) test commonly used for clay samples. In this test, the compressive force applied on a cylindrical soil sample (with a height-to-diameter ratio of 2–2.5) in vertical direction. To encase the sample, no rubber membrane is needed. This test can be performed on undisturbed cohesive soils. It cannot perform on coarse grained soils such as sand and gravel, as they cannot stand without lateral support. In addition, the test is essentially fast because it is believed that no moisture loss occurs during the test, which is performed relatively quickly.

3.2 Fall Cone Test

The fall cone test, also called the cone penetration test, is an alternative method to the Casagrande method for measuring the liquid limit of the soil. In this test, a 55 mm diameter soil sample is placed in a metal dish with a depth of 40 mm. A stainless steel cone weighing 80 grams having apex angle of 30° placed in such a way that

Fig. 1 Experimental setup for fall cone test

its tip touches only the sample. The cone is released for 5 s to allow the soil to penetrate. According to the measurement of cone penetration depth, the undrained shear strength of soil sample can be expressed by Zhang et al. (2018) as follows.

$$S_u = K\frac{W}{h^2} \tag{1}$$

where S_u = undrained shear strength (kPa); W = weight of cone (80 g in this study); h = penetration depth (mm); and K = fall cone factor, which was set as 1.33 in this study as suggested by Koumoto and Houlsby (2001). Equation 1 suggests that the undrained shear strength of the soil depends on weight of the cone, depth of penetration, and fall cone factor. The fall cone test was carried out with the cone weighing 80 g having apex angle of 30°. Before each fall cone test, the surface was jelly coated to minimize the frictional effects. The experimental setup is shown in Fig. 1. The prepared sample was placed under the top of the cone and then the cone was slowly lowered until its tip just touched the surface of the sample. Thereafter, the cone was released and was allowed to get into the soil cup due to its own weight. The penetration time was set to 5 s to measure the penetration depth. The depth was measured with a graduated scale with an accuracy of 0.01 mm.

About 20 g of soil paste was taken from the cup to determine the moisture content of the test sample after penetration. For different lignin content, the fall cone test was performed on five samples with different moisture contents, including 0%, which was selected for comparison of treated and untreated soil.

3.3 California Bearing Ratio Test

This is a penetration test developed by the California division of highways, as a method for evaluating the stability of soil subgrade and other flexible pavement materials. The test results have been correlated with flexible pavement thickness requirement for highways and air fields. The CBR test may be conducted in the laboratory on a prepared specimen in a mold or in situ in the field. The ratio of the force per unit area required to penetrate a soil mass with standard penetration plunger at a uniform rate of 1.25 mm/min and to the corresponding penetration load of the standard material or crushed stone is called CBR.

Before initiating the test, the calibration of the CBR proving ring was done. 4.5–5 kg of soil was taken and mixed it well with the required amount of water (OMC) or moisture content in the field available. The separator was placed on the bottom of the mold on the base plate and a coarse filter sheet was placed on the spacer disk. Wet soil was compressed by light or heavy compaction in the mold. The collar was removed and the extra soil was removed, the clamps were removed, and the compressed soil mold was raised. The filter paper was placed on the base plate, the mold compacted, the bottom turned and placed on the plate. Base and clamps were fixed. The weight of 2.5–5 kg was placed on the top of the mold. The mold was installed on the base plate and the same pressure weights were applied to the test sample. A complete setup was placed under the loading machine. Penetration of the piston was applied to the soil surface by applying a 4 kg load. The dial gauge of the calibration ring and the penetration dial gauge were set to 0. The load was applied at a penetration rate of 1.25 mm/min. The CBR value can be obtained as follows

$$\mathrm{CBR}(\%) = \frac{\text{Test load} \times 100\%}{\text{Penetration load (standard material)}} \tag{2}$$

The load value and the corresponding intervention value were stored. On the x-axis, a graph was drawn against the penetration depth (mm) and on the y-axis against the load (kN). Finally, the CBR was calculated from the Eq. (2).

4 Result and Discussion

4.1 Unconfined Compressive Strength Test Result

Influence of sodium lignosulfonate in increasing the performance of soil was investigated using a series of unconfined compressive strength test. The samples were prepared at optimum moisture content (i.e., 19.5%) with 0, 1, 2, 3, 4 and 6% of lignin content and were cured for 14 days.

From test results, it is observe that the value of unconfined compressive strength of the silty soil was increasing up to 3% addition of lignosulfonate but as the content of lignin was increased beyond this value there was drastic downfall in unconfined compressive strength of soil (Fig. 2). It can be seen that the unconfined compressive strength of soil is found to be 143.09 kPa, and at 1% addition of lignin with 14 days curing, it increases up to 8.04% (154.59 kPa). With increase in percent of lignin additive up to 3%, the strength increases up to 59.32% (227.99 kPa) beyond this, and the unconfined compressive strength decreases to 197.6 and 105.85 kPa at lignin

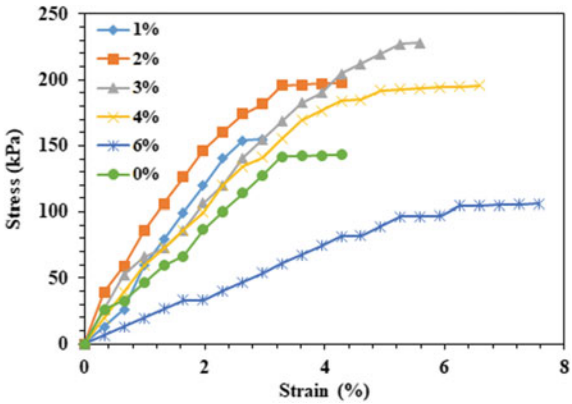

Fig. 2 Stress-strain behavior for different lignin contents

content of 4 and 6%, respectively. Therefore, the optimum percent of lignin content giving maximum performance is found to be 3% (weight/weight).

Failure of sample under axial load during unconfined compressive strength test is shown in Fig. 3. It is observed that the failure in the samples made from 0%, 1%, 2%, and 3% lignin content was sudden, and on further addition of additive, failure pattern shifted to progressive failure. It can be seen that the heavy bulging of sample was occurring. From Fig. 3a–c, it is clearly observed that the cracks are developed in 1%, 2%, and 3% additive samples, while at lignin content of 4%, sample undergo progressive failure and no clear crack is observed (Fig. 3d).

Further, the unconfined compressive strength test was carried out to investigate the influence of curing period on sample with optimum moisture content and optimum lignin content. Figure 4 presented the unconfined compressive strength variation for

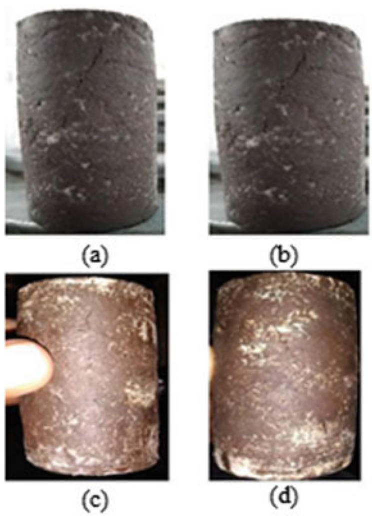

Fig. 3 **a** Failure at 1% lignin, **b** failure at 2% lignin, **c** failure at 3% lignin, **d** failure at 4% lignin

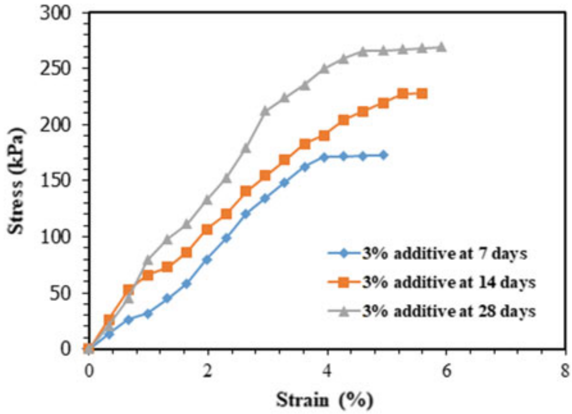

Fig. 4 Stress-strain behavior for a different curing time

curing period of 7, 14, and 28 days. It can be seen that the strength is increases with increase in curing period. This is because the cementing property of lignin increases with increase in curing time leading to increase the unconfined compressive strength of treated soil. Similar behavior has been reported by Zhang et al. [5]. The percent increase in unconfined compressive strength is found to nearly 88% with increase in curing time from 7 to 28 days. Hence, it can be concluded that the curing time is the most critical parameter for increasing the performance of treated soil mass.

4.2 Fall Cone Test Result

The undrained shear strength obtained from fall cone tests are presented in Figs. 5 and 6. From Fig. 5, it has been seen that the undrained shear strength (S_u) of soil mass at 0% lignin content was 8.33 kPa, whereas it increased to 12.3 kPa at 3% lignin content. Further, the undrained shear strength decreases with increase in the

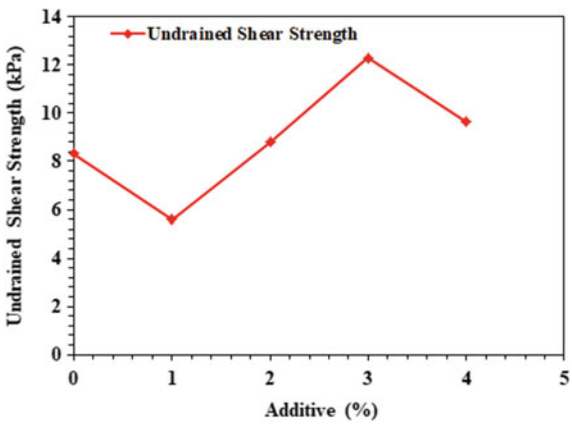

Fig. 5 Undrained shear strength using fall cone (28% water content) 14 days curing

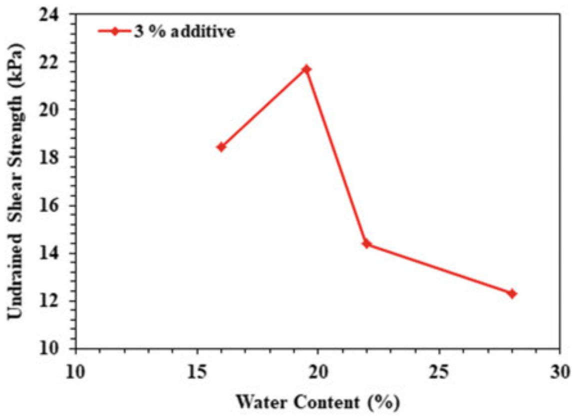

Fig. 6 Undrained shear strength using fall cone with 14 days curing at optimum lignin content

lignin content. The effect of water content on unconfined compressive strength with optimum content of lignin (3%) using fall cone tests is presented in Fig. 6. It can be seen that the undrained shear strength increases initially with increase in water content up to 19.5%, beyond which it decreases. Hence, it can be said the optimum water content giving maximum performance at lignin content of 3% can be considered 19.5%.

4.3 California Bearing Ratio Test Result

Unsoaked CBR tests were also conducted for silty soil with optimum percentage of sodium lignosulfonate (3%) with different curing time 7 days, 14 days, and 28 days. From Fig. 7, it can be seen that with increase in curing period the load carrying capacity increases. It is in general agreement with the results obtained from undrained shear test.

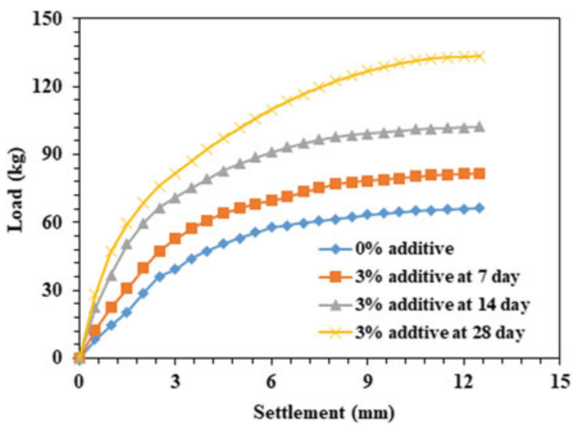

Fig. 7 Load-settlement behavior from unsoaked CBR at 3% additive of lignin with a different curing time

5 Conclusion

In this study, silty soil was stabilized with sodium lignosulfonate and its performance is evaluated using a series of laboratory experiments. The conclusions obtained from the results are summarized as follows.

1. The increase in curing time and additive content generally facilitates higher undrained shear strength (q_u), CBR, and whereas this property decrease slightly when lignin content exceeds 3%.
2. The optimum percentage of lignin for silty soil in this study is found to be 3%. Under the same curing time and degree of compaction, the 3% lignin-stabilized soil exhibits superior performances relative to the untreated soil.
3. Inclusion of lignin into silty soil results in lignin-based cementing materials that create bonding and fill the pores between detached soil particles. As a consequence, a stronger soil structure is formed, thereby increases the undrained shear strength (S_u) of soil.
4. The optimum combination is found out to be soil plus 3% lignin for unsoaked CBR. Value of CBR is found to be increased by 30.68%, 84.1%, and 109.8% for 3% lignin with 7 days, 14 days, and 28 days curing, respectively.

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Evaluation of Strength Characteristics on Black Cotton Soil–Stone Dust Mixtures Reinforced with Shredded Tyre Rubber

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Abstract Disposal of waste materials is a big problem with exceptionally growing up country like India. Rapid industrialization, population explosion, an extensive repletion of natural resources produces large quantities of waste materials which cause serious geo-environmental problems. Expansive soil swells and shrinks with regular wetness variation make structures founded on it unhinged and in practical cause huge economic loss in transportation division. In this, find out the waste material stone dust as a stabilizer and shredded tyre rubber which acts as a reinforcing material is selected. In the present study, stone dust blend to black cotton soil in changeable percentages of 5, 10, 15 and 20% by dry weight and shredded tyre rubber in varying of 1, 2, 3 and 4% added to black cotton soil stabilized with optimum percentage of stone dust. The treated and untreated samples were subjected to compaction, CBR (soaked and unsoaked) and unconfined compressive strength tests. The experimental values proved that there is a significantly increase in strength parameters for the black cotton soil added with stone dust and shredded tyre rubber combination.

Keywords Black cotton soil · Stone dust · Shredded tyre rubber · Compaction · CBR · UCS

1 Introduction

The enhancement of soil character is essential to build the soil secure in behaviour the load of structures. Wasteland can be improved by using ground improvement techniques which consist of stabilization or reinforcement of soil by means of mechanical and chemical methods or both. Soil stabilization by waste material is most popular

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due to low cost. Stone dust is very common material found abundantly which received from stone crushers. It helps to improve the properties of cohesive soil. Stabilization is a method of improving soil distinctiveness by use of assured additives. Use of shredded rubber for groundwork has acknowledged a great awareness in current period. Lime and cement are the most commonly used stabilizers. From the recent studies, it is observed that solid waste materials like stone dust, fly ash, blast furnace slag, rubber tyre, fibres, etc., for the stabilization of soils can be used as an alternative to conventional stabilizers. Stone dust is a solid waste produced from stone crushing units imposing environmental problems and respiratory problems for public. shredded tyre rubber, a solid waste which is produced from used tyres of vehicles and accumulating in large volumes posing serious geo-environmental problems. The disposal of stone dust and rubber tyres can be made in a proper and safe manner by using them in construction purposes for stabilization of soils in embankments for roadways, railways and foundations due to their binding and reinforcing properties. Effect of shredded rubber used as a stabilizer to develop the engineering properties of black cotton soil added with a varying range of 0, 2.5, 5, 7.5, 10% for experimental work of standard proctor tests and CBR tests were performed on both virgin soil and mix soils. From the tests carried out, MDD of soil sample increases up to 2.5% and reduces considerably with increase in rubber content. OMC reduces significantly as percentage of rubber increases. The CBR values show a minor increase up to 5% and thereafter increases significantly at 7.5% and then reduces drastically. The maximum enhancement of soaked CBR rate is found at 7.5% which is 2.62 and the percentage reduction in pavement thickness = 17.33% [1]. Adopted shredded rubber has been picked as the reinforcement material and marble dust as binding agent which was reinforced into the soil at different percentages with different combinations from 2, 4, 6, 8 and 10% by weight of soil. Results showed that the addition of rubber in the soil the maximum dry density decreases and the optimum moisture content does not show much of changes, whereas, with addition of marble dust in the soil, the maximum dry density starts to increases with a decrease in optimum moisture content. This could be due to specific gravity of marble dust and low plasticity, improvement in UCS value after the addition of marble dust into the soil and plasticity of the soil decreases as we add the marble dust in to the soil [2, 3]. Investigate done the use of waste stone dust and plastic glass strips in geotechnical applications by conducting compaction and CBR tests blending varying percentage of stone dust (5, 10 and 15%, etc.) and plastic glass strips (0.5, 1 and 1.5%, etc.). The maximum dry density is 1.94 gm/cc and optimum moisture content 18.91%, CBR percentage goes on increasing at 15% stone dust and 1.5% plastic strip and further addition of stone dust and plastic strip the maximum dry density decrease and optimum moisture content is increases. From the various study, it has been found that adding of 50% of stone dust to the soil is decrease the water requirement during field compaction, increase the MDD and 30% of stone dust help to gain effective specific gravity. The optimum value of stone dust is to improve the CBR and UCS 10–15% and concluded that stone dust can be used as cost-effective stabilized agent which improved the engineering properties of highly cohesive soil effectively [3, 4]. Studies for utilization of shredded tyre rubber and marble dust for the stabilization of expansive soils blended in rate of 2, 4, 6, 8 and

10% by dry weight of soil. Results found that the addition of rubber in the soil the maximum dry density decreases and the optimum moisture content does not show much of changes. Waste shredded rubber-soil mixture showed an improvement in UCS value after the addition of marble dust into the soil [5]. Investigations regarding the appropriateness of shredded tyre rubber for its use in geotechnical engineering in respect of stabilization of subgrade pavements were dwelt at full length. Added amount of rubber tyre had been varied in proportions of 4, 6, 8 and 10%. shredded tyre rubber mixed with soil showed enhancement in CBR value with adding up to 8% and there beyond decreased with additional increment in tyre content in unsoaked condition. Hence, the optimal value of shredded tyre rubber is 8% of size 25 mm × 50 mm in unsoaked conditions. The percentage enhancement in CBR value of stabilized soil is 66.28% in unsoaked condition, whereas an increase in CBR value can considerably trim down the total thickness of the pavement, and hence the total cost concerned in the project [6]. Adding stabilizers or binders either in dry or wet situation to develop the stiffness and strength of the weak soil and from the results of experiments, there is an increase in unconfined compressive strength due to increase in percentage of tyre scrap of various sizes, and the value of UCS is greater in comparison with that of parent soil and soil treated with 18% of tyre scrap, highest UCS value of 1.75 kg/cm^2 has been observed and no significant variation in the values of strain at failure [7]. In the present research work, an attempt has been made to calculate the index and engineering properties of black cotton soil blend with different proportions of stone dust and shredded tyre rubber by weight. From the test results, there is an improvement in geotechnical properties of stabilized black cotton soil blend with admixture and reinforcement material.

2 Study Design

The present study has planned in three stages. In the first stage, it is proposed to carry out individual geotechnical properties in laboratory of the materials used during the study. In the second phase, stabilization method tried in the laboratory carried out blending with different proportions of stone dust to calculate the optimum percentage as shown in Fig. 1. In the third phase, different percentages of shredded tyre rubber were added to the optimum values arrived at second phase combinations as shown in Fig. 2. Based on the assessment of results, the optimum percentage of shredded tyre rubber find out from the laboratory experimentation and comparison will be made with a view to know the improvement in geotechnical properties. The details of each of stages are explained in the following articles.

Fig. 1 Flowchart showing different percentages of stone dust blend with black cotton soil

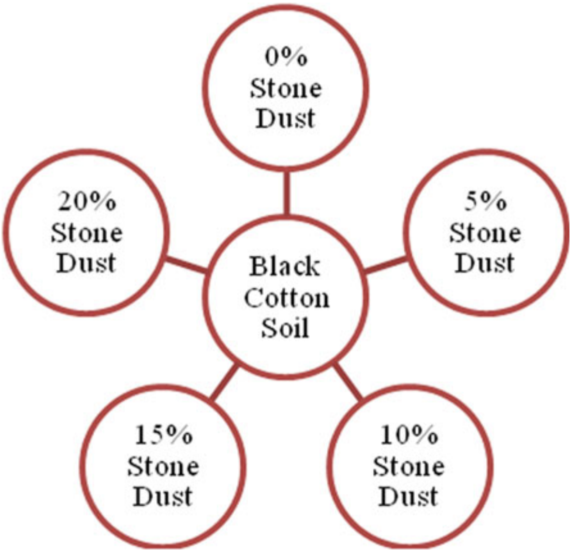
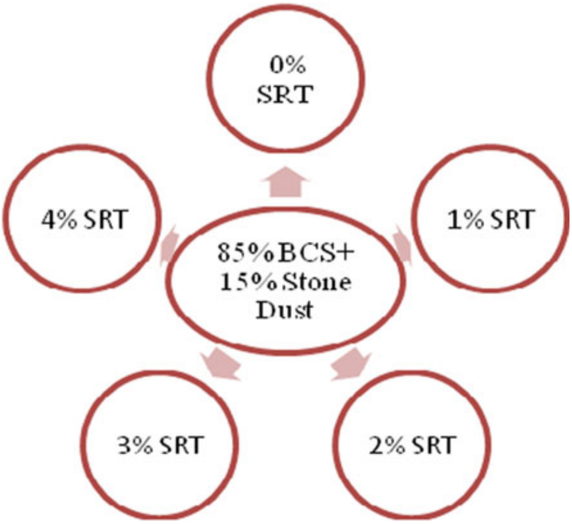


Fig. 2 Flowchart showing various mix proportions of black cotton soil with 15% stone dust and % of shredded tyre rubber



3 Materials Used

Details of assorted materials used throughout the laboratory testing are presented in the succeeding segment.

3.1 Black Cotton Soil (BCS)

The soil used was a typical black cotton soil procured from 'Amalapuram', East Godavari District, Andhra Pradesh State, India. All the tests carried on the soil are as per IS specifications.The geotechnical properties of soil are differential free swell = 140, specific gravity (G) = 2.67, liquid limit = 85%, plastic limit = 39%, coefficient of uniformity = 6.7, coefficient of curvature = 2.01, OMC = 23%, MDD = 14.38 kN/m^3. Soaked CBR = 1.35% and UCS = 40.74 kN/m^2.

3.2 Stone Dust (SD)

Stone dust can be considered as cost-effective stabilizer for ground improvement techniques of weak soils. Stone dust for this study was collected from Mahalakshmi Stone Crushing Unit, Nidadavole, Andhra Pradesh. The air-dried stone dust was passed through IS sieve 4.75 mm was used for this work and its properties are specific gravity (G) = 2.57, optimum moisture content (OMC) = 13.45% and maximum dry density = 1.59 gm/cc.

3.3 shredded tyre rubber (SRT)

shredded tyre rubber was collected from Tyre Shredded Unit, Palakol, West Godavari, Andhra Pradesh. Tyre cutting machine removes the steel belts during cutting and shreds of the tyre into pieces. shredded tyre rubber is in a length ranging between 10–25 mm and thickness 2–3 mm. The specific gravity of rubber tyre is in the range of 1.1–1.2.

4 Laboratory Experimentation

Various tests were carried out in the laboratory for finding the index and other important properties of the soils used during the study. Compaction, soaked CBR and unconfined compressive strength tests were conducted by using different percentages of stone dust (SD) and shredded tyre rubber (SRT), with a view to find the optimum percentages and its effect on strength properties of black cotton soil. The details of these test results presented in the following sections.

4.1 Index Properties

Regular procedures recommended in the respective IS codes of practice [IS:2720 (Part-5)-1985; IS:2720(Part-6)-1972] [8, 9] were followed while finding the index properties, viz. liquid limit and plastic limit of the samples tried in this cram.

4.2 Compaction Properties

Optimum moisture content and maximum dry density of black cotton soil blend with dissimilar percentages of stone dust and shredded tyre rubber mixes were indomitable according to IS compaction test IS: 2720 (Part VIII)-1983 [10].

4.3 California Bearing Ratio (CBR) Test

CBR test was carried out on prepared soil samples of untreated and treated black cotton soil with various percentages of stone dust and shredded tyre rubber under soaked conditions as per IS: 2720 Part XVI-1987 [11] recommendations as shown in Fig. 3.

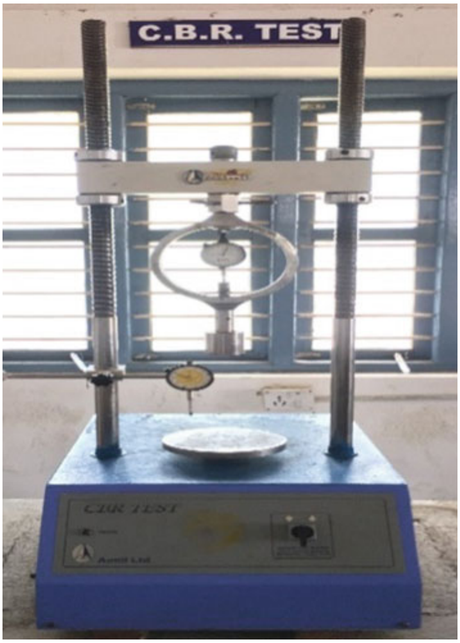


Fig. 3 CBR test apparatus

4.4 Unconfined Compressive Strength (UCS)

These tests are carried out in the laboratory under the IS code (IS: 2720, Part X (1991)) [12] from compaction parameters, i.e. maximum dry density and optimum moisture content at a displacement rate of 1.2 mm/min. Proving ring used 2 kN capacity for testing models as shown in Fig. 4.

5 Results and Discussion

Laboratory tests were conducted for finding the index and other important geotechnical properties of the materials used during the study. Compaction, soaked CBR and UCS tests were conducted by using different percentages of stone dust and shredded tyre rubber mixed with black cotton soil for finding optimum percentages and its effect on geotechnical properties.

Fig. 4 UCS test apparatus

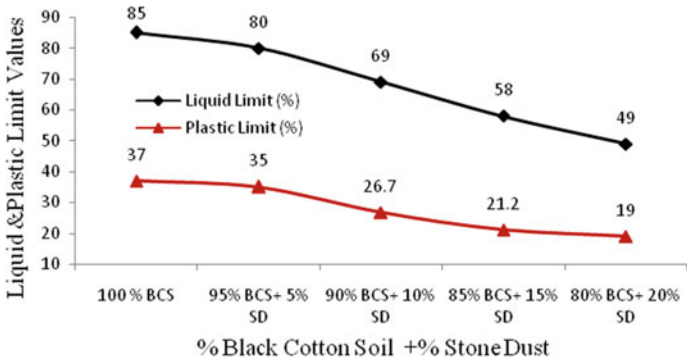


Fig. 5 Variation in liquid and plastic limit values of black cotton soil treated with different percentages of stone dust

5.1 Variation of Liquid and Plastic Limit Properties

The liquid limit and plastic limit values are decreasing as blending of stone dust percentage increasing as shown in Fig. 5. The liquid limit values are decreasing from 85% to 49% and the plastic limit values are also decreasing from 37% to 19% when the percentage of stone dust varies from 0% to 20%, respectively.

5.2 Effect on Compaction Parameters

The maximum dry density of black cotton soil was 14.38 kN/m^3. When optimum percentage of stone dust (15%) blend with black cotton soil, the maximum dry density increased to 16.86 kN/m^3. The maximum dry density of black cotton soil treated with optimum percentages of stone dust (15%) and shredded tyre rubber (2%) the density was decreased to 16.32 kN/m^3 where as the optimum moisture content decreases, respectively, as shown in Fig. 6.

5.3 Effect on Soaked CBR

Addition of stone dust to black cotton soil, the soaked CBR values vary from 1.85%, 2.52%, 3.27%, 4.75% and 4.57% addition of 5% increments stone dust as shown in Fig. 7. Soaked CBR values change from 4.75%, 5.91%, 7.68%, 7.11% and 6.25% with the addition of 0%, 1%, 2%, 3% and 4% shredded tyre rubber as shown in Fig. 8. From the above figures, at 15% blending stone dust to black cotton soil attained maximum CBR value 4.75 and blending 2% shredded tyre rubber to optimum soil–stone dust mix attained 7.68, respectively.

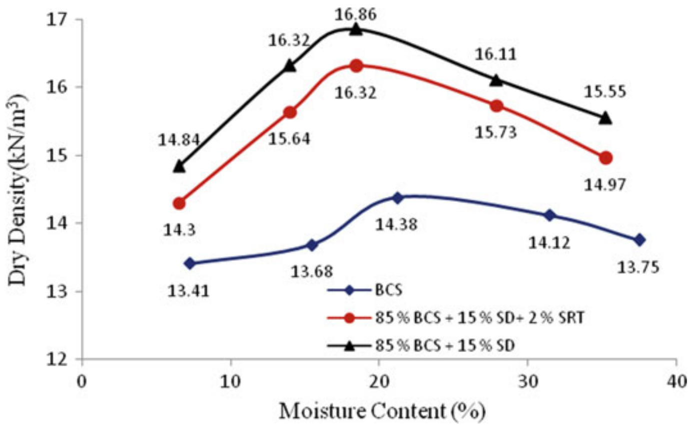


Fig. 6 Variation in compaction parameters of black cotton soil treated with different percentages of stone dust and shredded tyre rubber

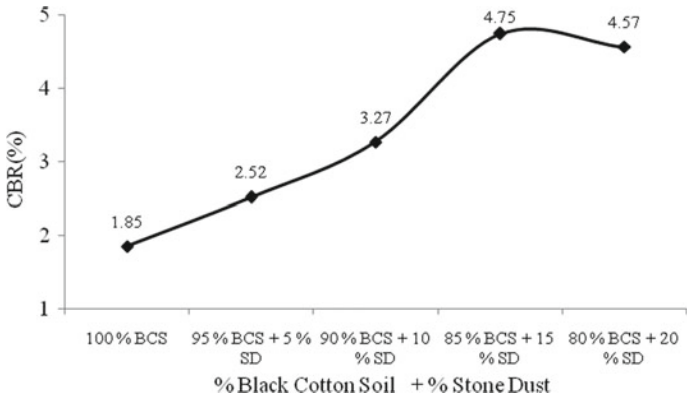


Fig. 7 Variation in soaked CBR values of black cotton soil treated with different percentages of stone dust

5.4 Effect on Unconfined Compressive Strength

The unconfined compressive strengths were conducted in the laboratory blending different percentages of stone dust and shredded tyre rubber as per IS 2720-Part X at different curing periods 0, 7, 14 and 28 as presented in Fig. 9. Blending stone dust to the black cotton soil, unconfined compressive strength values increasing up to 15% addition of stone dust and beyond it decreases. Considering 15% stone dust as optimum percentage, black cotton soil blend with different percentages of shredded tyre rubber UCS value is maximum at 2% shredded tyre rubber and beyond it decreases irrespective of curing. The maximum UCS values of 49.36 kN/m^2, 54.60 kN/m^2, 56.28 kN/m^2 and 59.30 kN/m^2 have been observed at optimum value

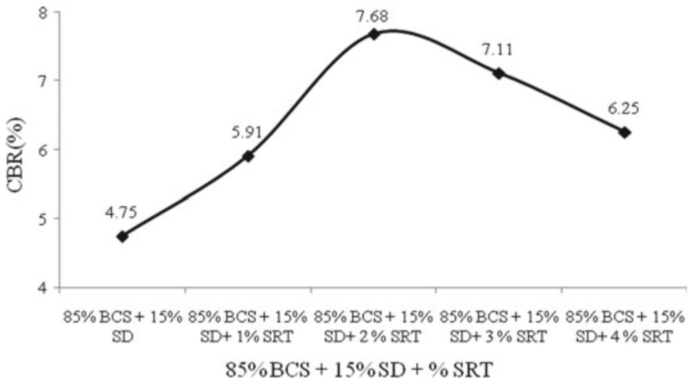


Fig. 8 Variation in soaked CBR values of black cotton soil treated with different percentages of stone dust and shredded tyre rubber

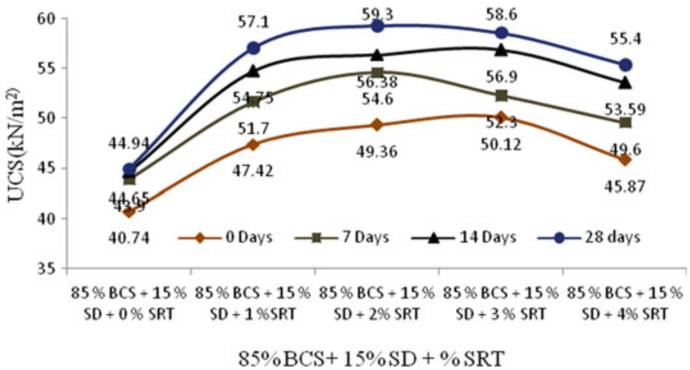


Fig. 9 Variation in unconfined compressive strength of black cotton soil treated with 15% of stone dust and different percentages of shredded tyre rubber at different curing periods

of 15% stone dust plus 2% shredded tyre rubber combination at different curing periods of 0, 7, 14 and 28 days respectively.

6 Conclusions

The following conclusions were made based on the experiments carried out in this investigation.

Addition of stone dust to the black cotton soil considerably decreases the liquid and plastic limits due to change in soil structure.

Maximum dry density increased from 14.38 kN/m^3 to 16.86 kN/m^3 when 15% stone dust blend to black cotton soil and decreased to 16.32 kN/m^3 when 2% shredded tyre rubber to optimum soil mix, respectively.

The soaked CBR value of black cotton soil treated with 15% stone dust and 2% shredded tyre rubber is found to be 4.75 and 7.68% which increases 160 and 315% and also it is satisfying standard specifications. CBR value for increase in percentage of stone dust and shredded tyre rubber that show the densification of soil takes place and more suitable for foundation.

Unconfined compressive strength increased by 40% when soil treated with 15% stone dust and 2% shredded tyre rubber and curing significantly increases the strength.

Stone dust is a waste product which is produced abundantly from quarry industry, as it is having a high specific gravity of 2.57 is used as stabilizing material. It has coarser particles which can be considered as a good binding agent and shredded tyre rubber can act as a reinforcing material. The properties of black cotton soil improve significantly by the addition to stone dust and shredded tyre rubber and their use in stabilization of black cotton soil can reduce construction cost and provide a better environmental solution for their disposal. The utilization of industrial wastes is an alternative to reduce the construction cost of roads particularly in the rural areas of developing countries.

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Experimental Study on Load-Settlement Behaviour of Granular Stone Column in Expansive Soil

Satish Barmade, Vinayak Kale, and Mahesh R. Gadekar

Abstract In today's construction industry as land reclamation is increasing rapidly so ground improvement has become necessity. Ground improvement is carried out by increasing vibration, structural fill or reinforcement, vegetation, admixtures, etc. out of all the methods, the stone column technique is a very effective method of improving the strength parameters of soil like bearing capacity and reducing settlement, particularly, for the construction of flexible structures, such as road embankments, oil storage tanks on soft soils. It offers a very economical and sustainable alternative to piling and deep foundation. The model test was performed on untreated soil and treated soil with 40 mm, 60 mm, 80 mm diameter stone column. The investigation focused on the influence of diameter of granular stone column. The tests were conducted on granular stone column having l/d ratio equal to 10. From the studies, the performance of smaller diameter stone column is superior to that of bigger diameter stone column. Due to its higher modulus of elasticity than that of soil, it absorbs more load than soil and reduces overall settlement.

Keywords Stone column · Expansive soil · Ground improvement

1 Introduction

In India about 5.46 lakh sq.km area (i.e., 16.6% of the total geographical area of the country) is covered by black cotton soil and this soil is mainly found in Maharashtra, Madhya Pradesh, Karnataka, Andhra Pradesh, Gujarat and Tamil Nadu. In the recent past, a large number of ports and industries are being built. In addition, the

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land available for the development of residential, institutional, commercial, industrial, infrastructure, etc., are scarce particularly in urban areas [1]. This requires the utilization of sections having weak strata with varied engineering characteristics. Several of these areas are lined with thick and soft clay deposits with very low shear strength and high compressibility [2]. Piles are normally being used in this land to carry the load of a superstructure. For a low rise construction, the cost of piles may be very high and people often use some ground improvement technique [3].

Out of several techniques offered for improvement of the weak strata, the stone columns used in a large extent due to the wide savings in cost and in programme schedules over the conventional piling solutions. Stone columns are used in weak deposits to increase the load-carrying capacity and reduce settlement of foundations [4].

Expansive soil loses its strength on wetting and drying. Although the phenomenon of swelling and shrinkage is common with most of the soils (except sand and gravel), it is exhibited considerably in clayey soil. A large number of methods that are commonly suggested to improve the behaviour of such soils include the soil replacement, mechanical and chemical stabilization, and thermal treatment [1]. A brief review of the common methods and their shortcomings has been discussed by Kumar and Jain (2016). The literature presented that granular piles are extensively being used to improve the load-carrying capacity and to reduce settlements of soft expansive soil [5]. Further, the load-carrying capacity can be increased by mixing nylon fibres or encasing the pile by geogrid [5, 6].

The present study aims at the load-settlement behaviour of untreated and treated soil with varying diameter stone columns. The influence of diameter of granular stone column was accessed having column's l/d ratio equal to 10 [7].

2 Experimental Work

The experimental work consists of two parts

1. Determination of physical and mechanical properties of soft soil and stone column materials.
2. Testing of model embankments resting on soft soil strengthened by stone columns.

2.1 Experimental Setup

An experimental setup with an approximate scale of 1/10–1/20 of the prototype was designed. A plastic tank was used to host the bed of soil and all its accessories. The tank has an internal diameter 1000 mm and depth 800 mm. The tank was sufficiently rigid and exhibited no lateral deformation during the preparation of soil bed and during the test. The loading frame consists of two steel rods welded together with

Table 1 Properties of soil and stone column material

Property	Soil	Stone
Liquid limit (%)	55.5	–
Plastic limit (%)	24.0	–
Plasticity index	31.5	–
Specific gravity	2.58	2.70
Soil classification	CH	GW
Maximum dry unit weight (kN/m^3)	15.1	17.5
Optimum moisture content (%)	19.6	

steel plates at bottom and top steel plate was movable for arrangement of fixing the jack. Load was applied using hydraulic jack and measured using proving ring and displacement was measured using dial gauge.

2.2 Properties of Soil and Stone Column Material

The soft soil used for experiment was collected at depth of 5 m from Bapu track, Nashik. The soil and stone column material was characterized using routine laboratory tests such as Atterberg limits [8], specific gravity [9], grain size distribution [10] and standard proctor test [11]. Table 1 presents the properties of soil and stone column material used in the present study.

2.3 Preparation of Soft Clay Bed

Tank was filled with the required amount of soil placed in a layer of 15 cm with uniform compaction to achieve a uniform dry density of 15 kN/m^3. Each layer was levelled gently using a 50 × 150 mm wood tamper. This process was continued till the soil bed depth of 75 cm was obtained.

2.4 Construction and Design of Stone Column

Stone column diameter: Installation of stone columns in soils is basically a self-compensating process, that is, softer the soil larger the diameter of the stone column. Due to lateral displacement of stones during vibrations/ramming, the completed diameter of the hole is always greater than the initial diameter of the probe or the casing. In the present study, stone column diameter of 40 mm, 60 mm and 80 mm was used.

Fig. 1 Pattern of stone column in model

Pattern of stone column: Stone columns should be installed preferably in an equilateral triangular pattern which gives the densest packing, although a square pattern may also be used. Figure 1 shows the equilateral triangular pattern of stone column used in study. The spacing (S) of stone column was 2.5 times diameter of stone column (S = 100 mm, 150 mm and 200 mm for stone column having diameter 40 mm, 60 mm and 80 mm, respectively).

The position of the stone columns was marked according to the proposed configuration patterns of stone columns given in IS 15284 (Part-1): 2003. The column was constructed by the replacement method [12]. A hollow steel pipe was pushed down the bed to the specific depth (i.e. l/d ratio = 10) at required spacing. After that, the casing was removed. The stones were placed in layer of 50 mm and compacted by using the tamping rod.

2.5 Test Procedure

The model tests were performed on prepared soil bed with and without stone column. The circular plate of size up to equivalent diameter was placed at the top of the centre of stone column, and dial gauges were mounted to measure the settlements of the soil bed. Load was applied through hydraulic jack and recorded using proving ring. Load per mm settlement was observed.

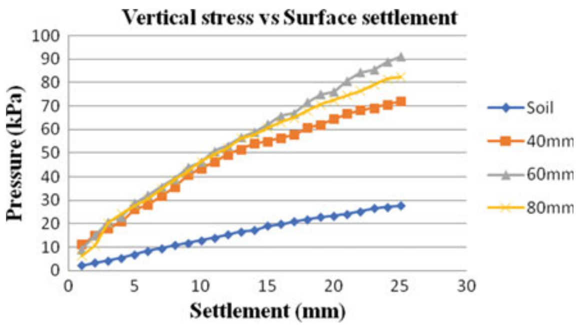

Fig. 2 Vertical stress versus surface settlement on soil and soil having stone columns

3 Results and Discussion

The tests were conducted on bed of untreated soil and treated soil. Figure 2 shows the effect of stone column on bearing pressure. The maximum bearing pressure of soil for 25 mm settlement with the 60 mm stone column was 91.23 kPa and for the 80 mm stone column was 82.56 kPa. The bearing pressure was increased with the stone column. The increase in bearing pressure was observed with increase in the diameter of stone column; however, with further increase in stone column diameter (80 mm), the bearing pressure was decreased. This may be attributed to the bulging of stone column in the large diameter stone column.

4 Conclusions

- The bearing pressure on untreated soil and soil with stone columns of 40 mm, 60 mm and 80 mm were 27.72 kPa, 72.09 kPa, 91.23 kPa and 82.56 kPa, respectively, for 25 mm settlement.
- The performance of the smaller (60 mm) diameter stone column was superior to that of the bigger (80 mm) diameter of stone column.
- The stone column was the good alternative to pile foundation.
- This may also promote the vertical drainage function by acting as a good filter.
- The mode of failure for an embankment model resting on untreated, very soft clay was close to local shear failure, whereas the mode gradually changed towards the general shear failure when using stone columns.

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Stability Analysis of Embankment on Stabilized Expansive Soil

T. V. Sowmyashree and Muttana S. Balreddy

Abstract In India, black cotton soil forms to be major soil deposits. They have been found to be the most problematic from engineering considerations since they exhibit swelling and shrinking when exposed to changes in moisture content. Vitrified tile sludge (VTS) is a waste material obtained during tile production and can be efficient in stabilizing embankments, soft soils, highway subgrade and other geotechnical application areas. High embankment slope has been stabilized using VTS as stabilizer. VTS was mixed by dry weight to soil with varying percentages of 0, 10, 20, 30 and 40% for conducting index properties, compaction characteristics, permeability, shear strength, consolidation and California bearing ratio tests. The experimental results indicate that the optimum VTS content was found to be 30%. The slope stability analysis was carried out for un-stabilized and stabilized embankment using Geo-Studio software. Slope stability is analysed for the high embankment of stabilized black cotton soil by taking its characteristics as input such as cohesion (c), angle of internal friction (ϕ) and maximum dry unit weight. The factor of safety of stabilized soil is more than minimum value as prescribed [1].

Keywords Vitrified tile sludge · Black cotton soil · Slope stability

1 Introduction

More than 90% of Indian road network including flexible pavement rests on soil subgrade. The performance of pavement mainly depends on subgrade and design procedures depend on stiffness and strength of pavement subgrade. Pavement resting on problematic soil such as expansive, soft soil and collapsible subgrade leads to complete or partial failure of pavements, in such cases lots of maintenance required

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due to repeated failures. That is why pavement structures should have subgrade of suitable engineering properties in order to increase service life and to minimize the thickness of flexible pavement structure. This can be acquired by the stabilization of expansive soil or replacement of weak clay with good soil [2]. Vitrified tile sludge (VTS) is the industrial waste generated from crushing of vitrified tiles in aqueous medium. The large quantity of the accumulated waste is dumped and left on costly land and causing wastage of good cultivated land. VTS has potential for use in highway embankment, subgrade and sub-base layer in combination with problematic soils. This leads to significant reduction in compressibility of expansive black cotton soil treated with VTS [3]. VTS affects the environment adversely and causes the problem to society. If we use these materials in the road sector, it may solve above problem and also reduce the cost of road material and convert waste into wealth. Several studies have been done on the stabilization of problematic soil using various industrial wastes [4–10]. This paper discusses the physical and geotechnical properties of VTS mixed with black cotton soil. Various tests were conducted on the VTS mixed with black cotton soil with varying percentages. Slope stability analysis was done for stabilized embankment using SLOPE/W Geo-Studio software.

2 Materials and Experiments

Expansive black cotton soil sample has been collected from Hiriyuru, Chitradurga district and vitrified tile sludge from H R Johnson Tile Limited, Kunigal, Tumakuru district, Karnataka, respectively. The grain size distribution of the soil sample was evaluated as per IS 2720-Part IV (1985). The black cotton soil contains 18% sand, 63% silt and 19% clay particles. The geotechnical properties of the BC soil and VTS are shown in Table 1. The chemical composition of VTS is shown in Table 2. The sample has been prepared by mixing VTS and soil in the proportion of 0, 10, 20, 30 and 40% to evaluate the index properties, compaction test, strength characteristics, CBR, etc. The tests were conducted according to Indian Standard specifications. Slope stability of the un-stabilized and stabilized slopes has been carried out using Geo-Studio software.

3 Stability Analysis Using Software

Stability analysis of embankment was carried out by using Geo-Studio software. Geo-Studio software includes the elementary features of SLOPE/W, SEEP/W, SIGMA/W, QUAKE/W, TEMP/W, CTRAN/W, etc. for analysing the slope stability and other related geotechnical analysis. In the present investigation, we have used SLOPE/W-Bishop's method, which gives the stability of the slope.

Table 1 Geotechnical properties of material

Engineering property	Laboratory test results	
	BC soil	VTS
Specific gravity	2.69	2.52
Grain size analysis		
Sand (%)	18	97
Silt (%)	63	3
Clay (%)	19	
Liquid limit (%)	53	–
Plastic limit (%)	21	–
Plasticity index	28	–
Shrinkage limit (%)	8	–
IS classification	CH	SW
MDD (kN/m^3)	17.63	–
OMC	17	–
Free swell index	37	–

Table 2 Chemical composition of VTS

Constituents (%)	Values
SiO_2	30.48
Al_2O_3	5.15
Fe_2O_3	4.86
CaO (%)	22.12
MgO (%)	8.1
K_2O	1.53
Na_2O	0.42

4 Geometry of Embankment

The stability analyses of embankment stabilized at optimum percentage VTS was carried out. A 15 m high embankment with Berm for MJB-Ch. 2 + 087 for existing SH-33&3 from Malavalli to Pavagada, Karnataka (See Fig. 1). As per IRC: 75 2015, high embankments are those exceeding 6 m in height, embankment of any height less than 6 m founded soft/compressible and/or loose strata.

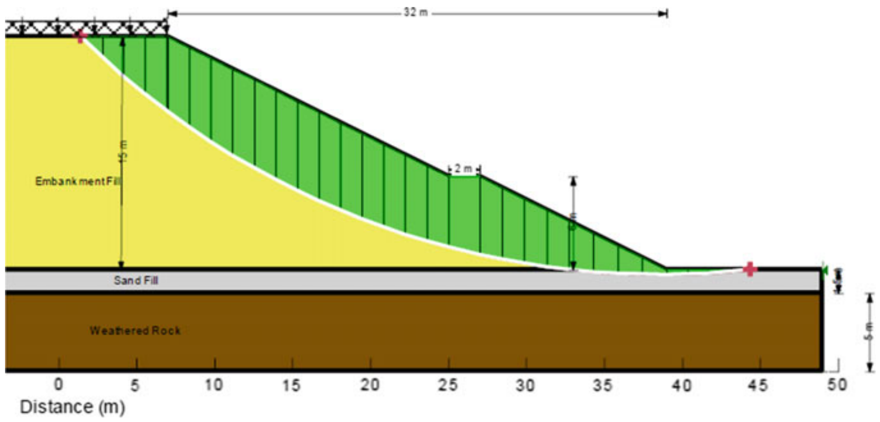

Fig. 1 Geometry of embankment

Table 3 Index properties of soil-VTS mixtures

VTS (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index
0	51	23	28
10	45	21	24
20	44	20	24
30	42	19	23
40	40	18	22

5 Results and Discussion

5.1 Index Properties

Liquid limit, plastic limit and free swell index tests were carried out for the different percentages of BC soil mixed with VTS. Table 3 shows the results of all samples. From Table 3, it can be observed that with the increase in percentage addition of VTS to BC soil, plasticity index decreases.

5.2 Compaction Characteristics

Modified proctor tests were conducted on different percentages of VTS mixed with BC soil. Table 4 shows the variation of maximum dry density (MDD) and optimum moisture content (OMC). It was found that, with the addition of VTS, the MDD increases, whereas the OMC decreases. The attraction of water molecules decrease

Table 4 Strength properties of soil-VTS mixtures

VTS (%)	MDD, kN/m^2	OMC (%)	c, kN/m^2	ϕ, degree	Permeability, X 10^{-4} cm/s	Soaked CBR (%)
0	17.63	17	80	13	0.03	2.29
10	18.024	15.1	60	18	0.3	7.38
20	18.78	14.53	40	21	1.1	9
30	19.5	14.04	25	23	4.74	10
40	19.94	13.48	30	19	5.48	9.03

with the addition of VTS to soil, hence, the decrease in OMC was observed [11]. The decrease in MDD is due to lesser content of sand size particle in the VTS as compared to the BC soil.

5.3 Permeability Test

Permeability tests were conducted on different percentages of VTS mixed with BC soil and presented in Table 4. The specimens were prepared by using static compaction method at their respective OMC and MDU. After ensuring full saturation, the specimens were tested to determine their coefficients of permeability using falling head permeability apparatus. The results show that with the addition of VTS the permeability increases. This property of adding VTS to BC soil demonstrates to be more efficient in order to have adequate drainage in high embankments.

5.4 Unconfined Compression Strength Test

Unconfined compression testing was carried out on soaked and unsoaked specimens to determine the unconfined compressive strengths (UCS) of BC soil mixed with VTS. Cylindrical specimens with a diameter of 38 mm and a height of 76 mm were prepared by compacting the mixture at their respective OMC and MDU. The UCS tests were conducted for curing stage of 0, 3, 7, 14 and 28 days in desiccators. In following curing stage, cured specimens are soaked for 1 day. Specimens are soaked by submerging them in container filled with water like top of specimen is 1 cm beneath height of water. Soaked specimens are kept in air for drying about 1 h and the specimens were subjected to gradually increasing uniaxial load in unconfined compression apparatus. From these tests, it was observed that both soaked and unsoaked UCS increased with the increase in curing period up to 30% VTS and after that the strength decreases. The results are tabulated in Table 5.

Table 5 Unconfined compression strength characteristics

Test condition	Curing period (days)	VTS (%)				
		0	10	20	30	40
Soaked	3	9.87	15.64	28.32	31.28	25.42
	7	9.62	27.97	36.31	41.26	31.66
	14	8.13	34.73	48.56	54.09	44.91
	28	8.08	43.93	56.03	61.57	57.41
Unsoaked	3	113.67	129.26	219.19	236.32	177.5
	7	112.36	144.54	262.28	282.65	247.75
	14	111.58	165.02	272.11	298.12	264.99
	28	112.12	180.86	293.19	310.22	282.58

5.5 Static Triaxial Test

Triaxial tests were conducted on different percentages of VTS mixed with BC soil to determine the cohesion and angle of internal friction. Cylindrical specimens with a diameter of 38 mm and a height of 76 mm were prepared by compacting the mixture at their respective OMC and MDU and subjected to unconsolidated undrained triaxial tests at varying confining pressure of 50, 100 and 200 kPa. From the results, it is observed that, with the increase in the percentage of VTS up to 30%, cohesion decreases, whereas the frictional angle increases. The increase in frictional angle may be due to the increase in silt size particle [12]. The results are presented in Table 4.

5.6 California Bearing Ratio (CBR) Test

California bearing ratio (CBR) tests were conducted on both soaked and unsoaked condition for different percentages of VTS mixed with BC soil. CBR specimens were prepared by compacting the mixture at their respective OMC and MDU. The soaked CBR test results are presented in Table 4. The results show that with the addition of VTS up to 30%, the CBR increases and then decreases for both soaked and unsoaked condition (results not presented). This shows the increase in stabilized CBR value is due to the frictional resistance between the particles with addition of VTS.

5.7 Consolidation Test

Consolidation test was carried out on remoulded specimen with 30% VTS mixed with BC soil in an oedometer under double drainage condition. The C_c value for

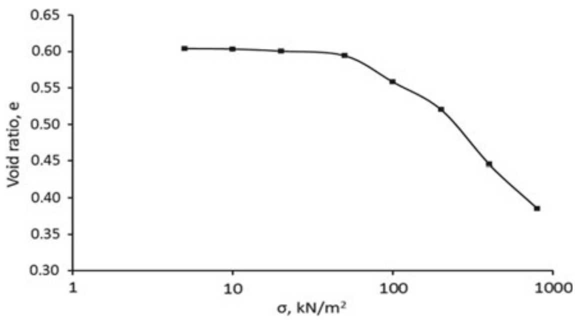

Fig. 2 e-log p curve for 30% VTS mixed with BC soil

BC soil of 0% VTS content was 0.31. It reduced to 0.19 by 30% optimum VTS content addition (see Fig. 2). The consolidation settlement (S_c) for BC soil of 0% VTS content was 0.812 mm and with the addition of 30% optimum VTS content, S_c decreased to 0.532 mm. This is because of the development of cementation of interparticle bonds that enhance the strength as well as decrease the compressibility.

5.8 Stability Analysis of Embankment

The stability analysis has been carried out by Bishop's method on un-stabilized and stabilized slopes using Geo-Studio software. The sand fill has been modelled as Mohr–Coulomb having the properties unit weight = 18 kN/m^3, cohesion = 5.4 kPa and $\phi = 32°$. The critical slip surface and FOS for un-stabilized and stabilized embankment (see Figs. 3 and 4). It can be clearly observed that the stability analysis of stabilized embankment and un-stabilized embankment has the factor of safety more than minimum value 1.25 as per IRC: 75 specifications. Since un-stabilized embankment is of fully black cotton soil which cannot be considered for practical purpose considering the other strength and index properties, so the stabilized embankment with factor of safety 2.786 is considered and is safe as per IRC: 75-2015.

6 Conclusions

Based on the test results and numerical modelling, the following conclusions have been made

- Addition of varied VTS content has improved the properties of the black cotton soil. Soaked UCS increased with the increase in curing period up to 30% VTS and after that, the strength decreases. Therefore, the replacement of vitrified tile sludge by 30% is found to be optimum.

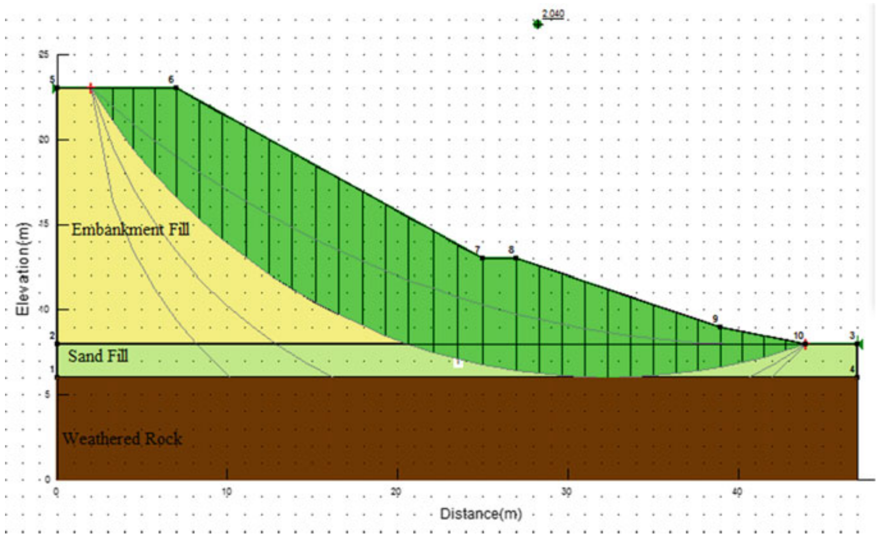

Fig. 3 Critical slip surface and FOS for un-stabilized embankment

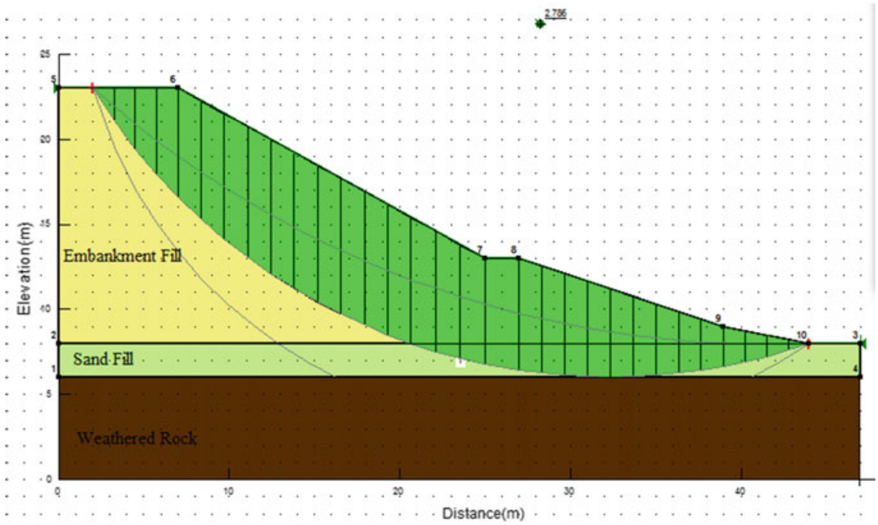

Fig. 4 Critical slip surface and FOS for stabilized embankment

- The MDD increases and OMC decreases with varying VTS content of 0–40%. Similarly, CBR increases with increase in VTS till optimum content.
- The findings indicate that the permeability increases with the addition of VTS. This property of adding VTS to BC soil proves to be more effective for proper drainage in high embankments.

- Cohesion value decreases with the increase in the percentage of VTS up to 30%, whereas the frictional angle increases.
- Addition of 30% VTS showed a decrease in consolidation settlement (Sc) when compared with 0% VTS. This is due to the growth of interparticle bond cementation, which increases strength and reduces compressibility
- Since un-stabilized embankment consists of completely black cotton soil which cannot be regarded for practical purposes in view of the other strength and index characteristics, the stabilized embankment with safety factor 2.786 is therefore regarded and is secure as per IRC: 75-2015.

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A Laboratory Study on the Mechanical Behaviour of Dredged Soil Admixed with Waste Rubber Tyre Powder and Cement

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Abstract Due to the rapid development in automobile industry, the amount of tyre wastes has been increasing every year throughout the world. An attractive method to reduce the tyre waste produced is the use of recycled waste materials for civil engineering application, and also, it may be used as a stabilizer in soils so that it may help in improving the engineering properties of soft soil. Dredge material which belongs to soft soil deposits usually has low bearing capacity, high compressibility and undergoes settlement over a long period of time, hence cannot be used as a construction or foundation material. In this study, tyre rubber powder has been used to see its influence on the mechanical properties of dredged soil. Dredged soil was collected from the catchment of Dal Lake- Nishat. Various tests like specific gravity, gradation analysis, Atterberg's limit, compaction tests, direct shear tests, unconfined compressive strength, California bearing ratio tests have been done in order to characterize and find the shear strength parameters. This study involves performing compaction test and UCS on the soil incorporated with rubber powder (passing 425μ) at varying percentages of 1.5, 3, 4.5, 6, 9, 12 and 15%. Also, to further improve the strength, cement at a constant percentage (2%) was added to rubberized soil and the effect on UCS characteristics at various curing periods of 3 and 7 days was analysed.

Keywords Tyre rubber powder · Cement · Dredged soil · Unconfined compressive strength

1 Introduction

Due to the rapid urbanization and population explosion particularly in developing countries, there is shortage of construction materials as well as scarcity of suitable construction sites. Thus, it forces the geotechnical engineers and specialists to adopt soft soil sites for construction. Soft soil deposits are very weak having low bearing capacity and undergo settlement for a considerable period of time [1]. To assess the

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mechanical behaviour of such soils, geotechnical engineers face a difficult task [2]. Using these spoils as construction material, foundation medium, backfill, in dykes and embankments [3–5] becomes a substantial problem for geotechnical engineers. This study deals with waste material generated by dredging the world famous Dal Lake. Dal Lake in Srinagar, Jammu and Kashmir, India, is the second largest Lake in state and has been integral to tourism and recreation in Kashmir. The Lake receives large quantity of sediments with the runoff from its catchment during the downward movement of water from Marsar. A large amount of sewage from the settlements around the Lake and the habitation living on hamlets within the Lake, on floating islands and houseboats enters the Lake without treatment. Huge quantity of solid wastes and superfluous fertilizers from inlet channels produces algal blooms which ultimately results in eutrophication. In September 2014, the massive flood which occurred when the river Jhelum rose to the dangerous level due to intense rainfall from 1st to 7th of September 2014, a huge amount of water entered the Dal Lake due to breech in embankment at Ram Munshi Bagh. This water brought large amount of silt and biological load in the Lake, thus causing siltation and pollution of Dal. As a result of this continuous disposal of wastes, silt deposition and encroachment, the load carrying capacity of the Lake has been reduced. So, it becomes necessary to carry out dredging of this lake [5–7].

On the other hand, with the change in living style and attitude of people, there is increase in number of automobiles, and the kilometre coverage by the vehicles is also increasing. All this resulting in increased demand of tyres as original equipment and as replacement has also increased. About 1.1 million types of new vehicles are added each year to the Indian roads. As a result, large quantity of waste tyres is produced. The disposal of these used tyres has become a global problem due to problems associated with conventional landfill disposal methods. Most of the times, tyres are being dumped on ground or added to garbage create environmental and health problems. There are possibilities of using these tyres in civil engineering applications such as highway embankments and backfills behind retaining structures over weak or compressible soils [8, 9]. The reclamation and recycling of waste rubber has been reviewed extensively in Adhikari and Maiti [10]. Many researchers have assessed some fundamental engineering properties of tyre crumb soil mixtures, such as compaction characteristics, compressibility and permeability, shear strength, modulus of elasticity and Poisson's ratio [11–13]. Jan et al. [14] found that maximum dry density and optimum moisture content of clayey soil gets decreased as the percentage of rubber gets increased. Researchers like [13] found that unconfined compressive strength of black cotton soils gets increased up to optimum rubber content, after incorporating rubber powder in varying percentages.

2 Materials and Methods

2.1 Materials

In the present investigation, samples of dredged soil have been collected from Dal Lake-Nishat. At the site, samples were sealed and transported in polythene bags for studying various in situ properties.

2.2 Testing Methodology

Various laboratory tests on soil sample like gradation, consistency, Atterberg's limits, unconfined compression test, direct shear test, California bearing ratio test have been performed. After this, crumb rubber of size passing 425 μm was added into the soil at percentages like 1.5, 3, 4.5, 6, 9, 12 and 15%. The properties of dredged soil having done as per relevant IS codes [15–21] are listed in Table 1.

3 Results and Discussions

3.1 Compaction Tests

Standard proctor compaction tests were conducted on the various combinations of clay-cement-rubber fibre to determine their maximum dry unit weight and optimum moisture content. The results obtained from various combinations of rubber cement and soil are shown in Fig. 1. The results revealed that as the percentage of rubber increases from 0 to 15%, maximum dry unit weight of soil got decreased and also optimum moisture content got decreased, although it varies over a narrow range. The decrease in maximum dry unit weight of the soil may be attributed to low specific gravity of rubber powder (1.15) and reason for decrease in OMC of soil may be due to the low water absorption [7–23]. On the other hand, when cement at a constant percentage of 2% was added to rubberized soil mixtures, MDD of the soil got increased up to an optimum rubber percentage of 6% and beyond that it decreases as shown in Fig. 2. Also, the addition of cement decreases the optimum moisture content of rubberized soil. This increase in maximum dry unit weight upon addition of 2% cement of rubberized soil, up to rubber content of 6% is due to the higher specific gravity of cement compared to rubber and soil. The decreaset in optimum moisture content may be due to the low water absorption capacity of rubber fibres [24] and also due to alteration in particle size distribution upon addition of cement [25].

Table 1 Physical properties of dredged soil

Sr. No	Properties		Nishat
1.	Natural moisture content (%)		35.2
2.	Bulk unit weight (kN/m^3)		14
3.	Insitu dry unit weight (kN/m^3)		10.3
4.	Specific gravity (G)		2.43
5.	% Finer than 75 µm		93.93
6.	Clay (%)		10
7.	Silt (%)		83
8.	Sand (%)		6.06
9.	Gravel (%)		0.07
10.	Coefficient of uniformity, C_u		8
11.	Coefficient of curvature, C_c		1.008
12.	Suitability number, S_n		812
13.	Liquid limit (%)		47
14.	Plastic limit (%)		34
15.	Shrinkage limit (%)		17.0
16.	Plasticity index (%)		13
17.	P.I, A-line		19.71
18.	P.I, U-line		35.1
19.	Classification		MI
20.	Clay mineral		Kaolinite
21.	Flow index, I_f		25.4
22.	Toughness index, I_t		1.95
23.	Activity		1.3
24.	Consistency index, I_c		0.90
25.	Liquidity index, I_L		0.09
26.	DST @ In-situ	Cohesion, c (kN/m^2)	8.57
		Angle of internal friction, Φ (Deg)	29.4
27.	UC-test @ In-situ, q_u (kN/m^2)		19.17
28.	Optimum moisture content (%)		34.2
29.	Maximum dry unit weight (kN/m^3)		12.54
30.	California bearing ratio un-soaked (%)		5.6
31.	California bearing ratio soaked (%)		1.3

3.2 *Unconfined Compressive Strength*

Unconfined compression test is the simplest and quickest method to determine the shear strength of cohesive soils. Test specimens were prepared, compacted under standard compaction at γ_{dmax} and optimum moisture content. The samples were tested immediately, 3 days and 7 days, of curing. Test results revealed that as the percentage of crumb rubber increased as shown in Fig. 3, axial stress of soil increases up to an optimum rubber value of 4.5%, beyond which it got decreased [26]. The decrease in axial stress of soil may be attributed to the elastic nature of rubber which shows pronounced effect after its optimum value. The failure strain of normal soil is 4.5%, which increases to 8.6% with the inclusion of 15% rubber fibres. The increase in the axial strain of soil–crumb rubber specimens corresponding to the failure axial stress may be accredited to elastic reaction generated by the rubber tyre during compression results into prevention against generation of cracks [27].

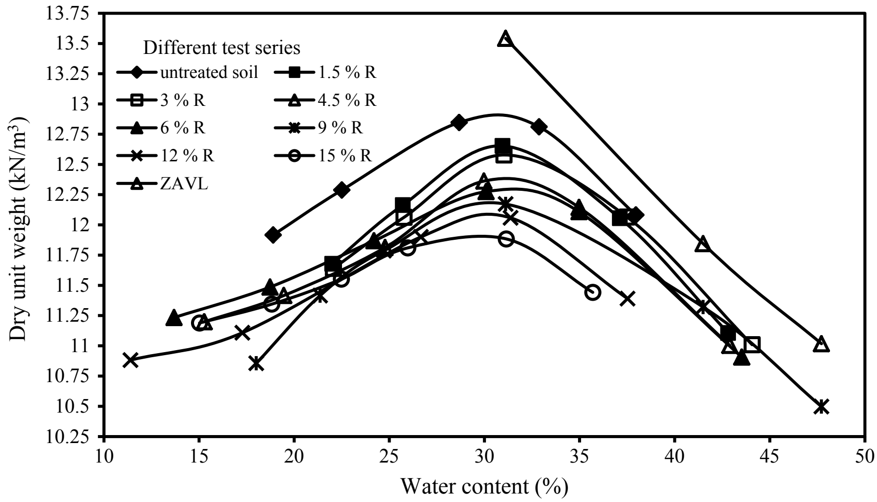

Fig. 1 Compaction curves for crumb rubber stabilized dredged soil

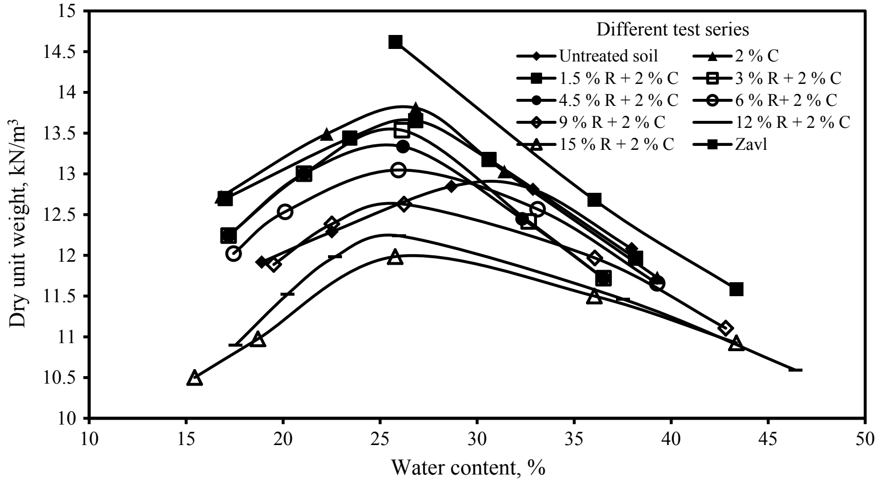

Fig. 2 Compaction curves for cement stabilized rubberized dredged soil

Also, it can be inferred from the results as shown in Fig. 4 that with the increase of the cement content in the rubberized soil, the peak axial stress increases up to rubber fibre content of 4.5%. The peak axial stress of cemented rubberized specimens increases as compared to the uncemented samples. The significant improvement in the peak axial stress may be due to cementation reactions. From Figs. 5 and 6, it is inferred that curing time also increases the strength with cement content and may be

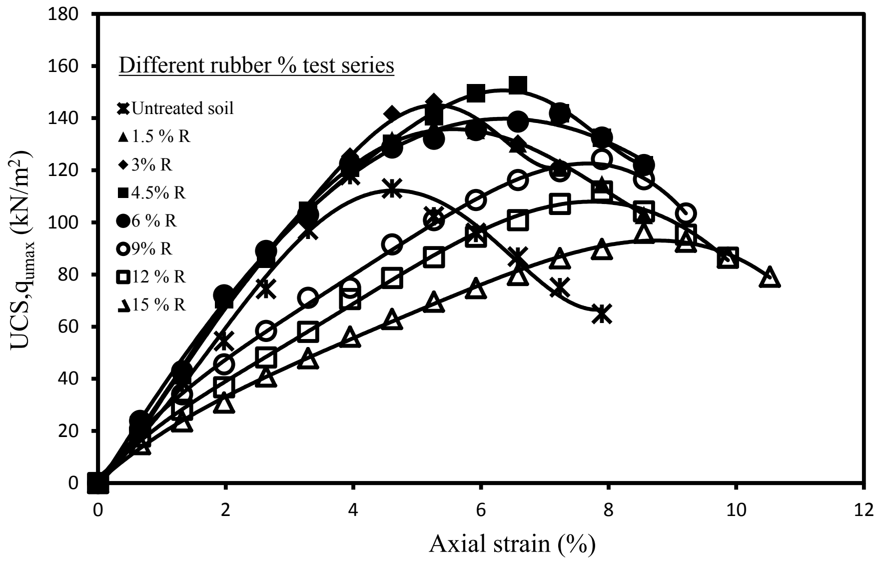

Fig. 3 Stress strain curves for rubber stabilized dredged soil

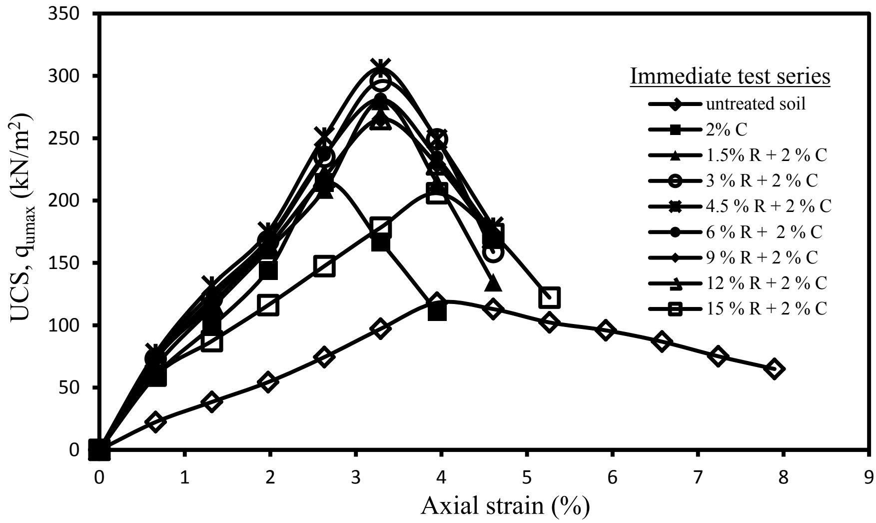

Fig. 4 Stress strain curves for cement added rubberized dredged soil (0 day)

due to the formation of secondary cementitious products. The maximum UCS was noted to be 460 kN/m^2

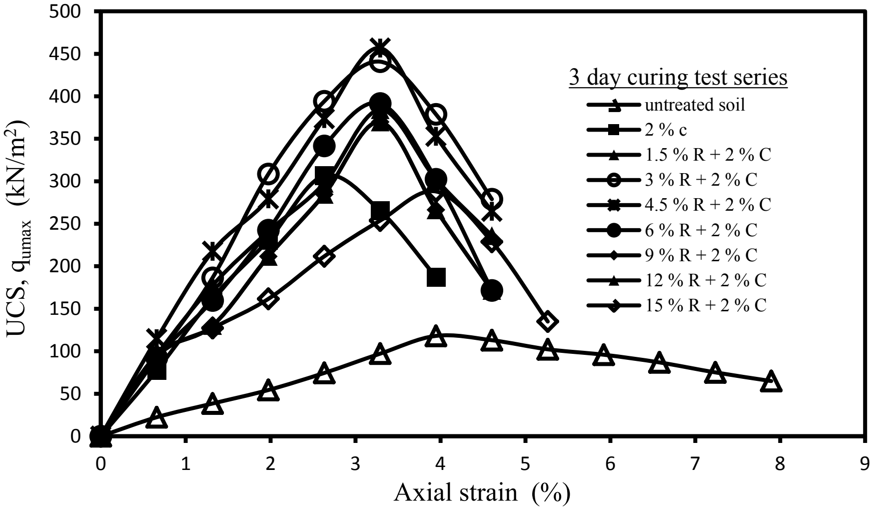

Fig. 5 Stress strain curves for cement added rubberized dredged soil (3 day)

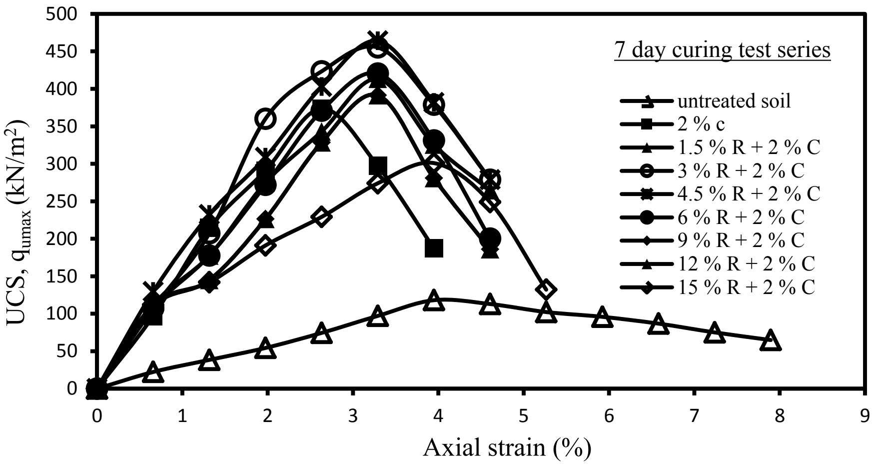

Fig. 6 Stress strain curves for cement added rubberized dredged soil (7 day)

4 Conclusion

On the basis of this investigation, following conclusions can be made:

1. Dredged material consists of poorly graded inorganic silt of medium plasticity with poor fill material characteristics
2. Dredged material under investigation possesses medium to high compressibility with high rate of loss of shear strength

3. Maximum dry unit weight and optimum moisture content of dredged soil decreased as the content of crumb rubber in the mixture increases
4. The UCS test results indicated that the addition of crumb rubber up to optimum value of 4.5% in the soil marginally improves the shear strength. However, axial strain corresponding to failure increases as the percentage of crumb rubber increases. The addition of cement increases the UCS of the soil dramatically but induces brittleness in the composite
5. The curing period increases the inter particle bonding due to pozzolanic reactions that take place by the formation of cementitious compounds.

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Improvement of Strength Reinforced by Sugarcane Fibre

Hasan M. Rangwala, Lavkush Agrawal, Vivek Makhecha, and Ishan Gondaliya

Abstract Soil is a base of structure, which supports the structure from beneath and distributes the load effectively. If the stability of the soil is not adequate, then failure of structure occurs in form of settlement, cracks, etc. Which can be prevented by soil reinforcement which is introduced in the field of geotechnical engineering by improving properties of soil. Which is the most popular techniques used for the improvement of poor soil. It causes significant improving in shear strength, bearing capacity, as well as economy. Many research has been conducted for stabilization of soil by using cementing, chemical materials, e.g. Fly ash, cement, Calcium chloride, etc. Today world is facing severe problem of disposal of agricultural waste. There are many natural fibre and synthetic fibre available in the market like Jute fibre, coconut fibre, palm fibre, sugar cane fibre and glass fibre, nylon fibre, polypropylene fibre, etc. Sugar cane fibre have chosen for soil stabilization in this study. Sugar cane fibre have been taken from sugarcane waste that is after extrusion of juice. In this experimental study, the fibre content has been taken 0.2%, 0.4% and 0.6% of the soil. After conducting series of experiment concluded that after mixing sugar cane fibre 0.6% of soil that is optimum fibre content, significant increment in angle of friction and decrement in the cohesion of soil is obtained.

Keywords Ground improvement · Sugarcane fibre · Triaxial test

1 Introduction

Geotechnical engineers often encounter problems in designing foundations of structures on highly compressible clayey soil due to its poor bearing capacity, low shearing strength, etc. Soil reinforcement is an effective and reliable technique for improving strength and stability of soils.

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Reinforced earth is a composite material, a combination of soil and reinforcement duly placed to bear the tensile stresses developed and to improve the resistance of soil in the direction of maximum stress. Fibre reinforcement is one of the novel emerging soil reinforcement techniques. Randomly distributed fibres provide interlocking and friction resistance to resist the movement of soil particles, which considerably increase the load carrying capacity. These reinforcement is widely used in the embankment, slope stabilization, pavement application in the current times. For efficient application of fibre reinforcement, proper understanding of effect of different parameters like fibre parameters and soil parameters on the behaviour of fibre reinforced soil is required.

Use of natural fibres such as cotton, bamboo, jute, coir, etc. as soil reinforcing materials has been prevalent for a long time. The concern in natural fibre reinforcement in soil has increased rapidly due to the growing environmental consciousness and understanding of the need for sustainable development to replace man-made artificial fibres.

Various researches have performed experimental studies on randomly distributed fibres using natural fibres and synthetics fibres. Hejazi et al. [2], Gowthaman et al. [1] and Shukla et al. [3] discusses the studies performed on various materials for randomly distributed fibres.

In this study, an attempt has been made to evaluate the strength of a soil-fibre mix using sugarcane waste fibre as reinforcing material. It has also attempted to find an optimal dose of fibre for the soil in consideration to achieve highest possible strength.

2 Test Sample

2.1 Soil Sample

The soil sample was obtained from Nirma University campus from a depth of 1.5 m below the ground level. Characteristics of soil is given in Table 1.

Table 1 Soil properties

S. No.	Property	Value
1	Percentage fines	36%
2	Plastic limit	23%
3	Liquid limit	20%
4	Soil type	SM
5	Bulk density	1.89 gm/cc
6	Field moisture content	9%

2.2 Sugarcane Waste Fibre

Sugar cane waste was obtained from local juice shop. The waste was washed by normal water to remove residual glucose and also its odour. Washing sugarcane waste would also increase durability of the fibre due to removal of biodegradable bacteria attach to the waste. A proper drying of sugarcane was done of air drying for 3-days and oven drying at 70 °C for 1 day.

The dried fibre was cut to the length of 2–3 cm length and their average diameter is around 0.5 mm. Hence, aspect ratio of fibre is around range of 40–60.

3 Experimental Study

An experimental study was devised to arrive at the optimum value of percentage of sugarcane waste fibre in the soil that can be added to achieve higher strength. Standard proctor test was performed to get OMC and MDD. Triaxial compression tests was then performed on sample prepared at the water content as OMC and the dry density of MDD.

3.1 Standard Proctor Test

Series of standard proctor tests were performed to determine the optimum value at which the soil-fibre mixture will have the highest dry density. The parametric study was carried out considering various fibre content. Fibre content of 0% (i.e. unreinforced soil), 0.2, 0.4, 0.6 and 0.8% are considered in this study. Results of the standard proctor test for various fibre content is illustrated in Fig. 1. It has been observed from

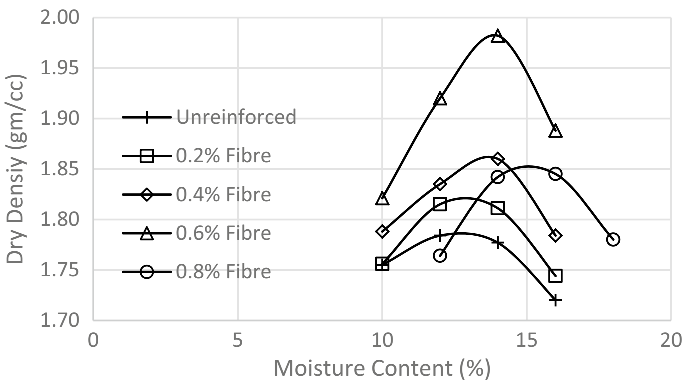

Fig. 1 Compaction curves for various soil mixes

results of standard proctor test that the density of soil-fibre mix decrease for 0.8% fibre, and hence, it has not been considered in the further study.

3.2 Triaxial Compression Test

Series of triaxial compression tests were performed to observe the effect of sugarcane waste fibre on shear strength of soil mix. In order to determine the optimum doses of fibre content, the parametric study was carried out considering various fibre content. Fibre content of 0% (i.e. unreinforced soil), 0.2, 0.4 and 0.6% are considered in the study.

4 Results and Discussion

This section included the discussion on the results and observation of test performed to discover the effect of inclusion of the sugarcane waste fibre and its optimized dose based on compaction and shear parameters.

4.1 Standard Proctor Test

An illustrated in Fig. 2, optimum moisture content (OMC) increase with an increase in the fibre content. This may be due to the fact that sugarcane waste fibre would also soak the water from the mix. Figure 3 shown variation of maximum dry density (MDD) with an increase in fibre content. MDD is observed to be increasing up to 0.6% of sugarcane waste fibre and decreased with the further increase in the fibre.

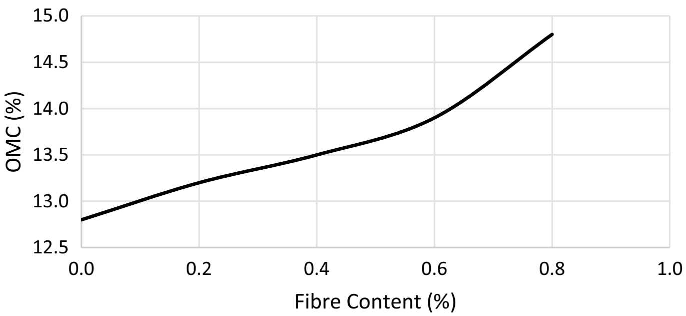

Fig. 2 Variation of OMC with fibre content

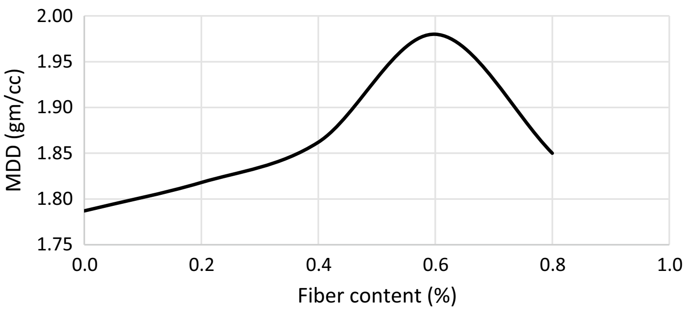

Fig. 3 Variation of OMC with fibre content

This may be due to the fact that soil material is replaced by a light material of sugarcane waste fibre.

4.2 Triaxial Compression Test

Shear parameters were found for each of the soil-fibre mix by plotting the Mohr's circle from the data observed during the triaxial compression test performed on a sample with 38 mm diameter and 76 mm height. Figure 4 shows the variation of cohesion (c) with fibre content. The cohesion decreases with an increase in the fibre content, this is due to the fact that the attraction between the soil particles is blocked by the sugarcane waste fibre. Figure 5 shows the variation of angle of internal friction (φ) with fibre content. The angle of internal friction increase with an increase in the fibre percentages, this is due the fact that the inclusion of friction between soil and fibre. The increase was observed to be as high as 500% in the angle of internal friction while the decrease in the cohesion was also observed up to 50% in comparison to the unreinforced soil, i.e. % fibre.

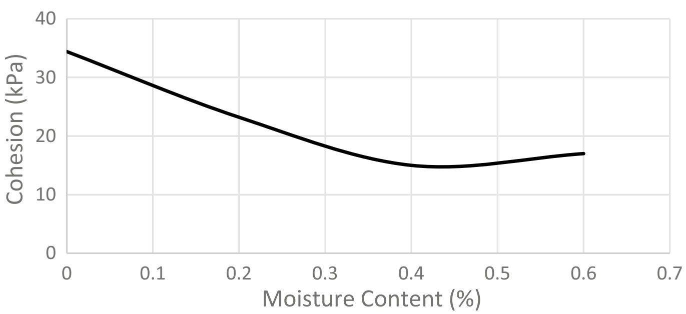

Fig. 4 Variation of cohesion with fibre content

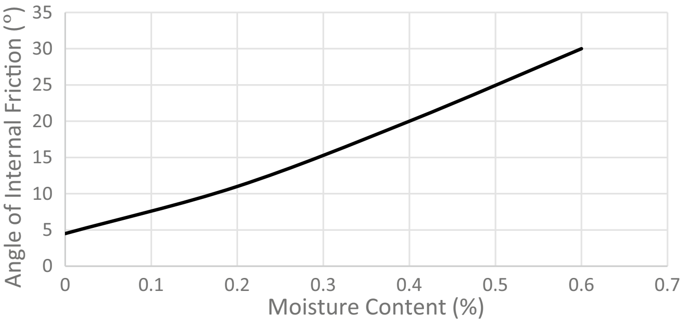

Fig. 5 Variation of angle of internal friction with fibre content

5 Conclusion

An experimental study to the effect of sugarcane waste fibre as an alternative sustainable reinforcement material for the improvement if shear strength was performed. The study was carried out using the fibre obtained from the sugarcane waste, obtained after the piling the same to make its fresh juice.

The compaction and shear parameters for the soil-fibre mix were determined for various mixes of soil and fibre considered for the study. The fibre was taken as 0, 0.2, 0.4 and 0.6% by weight of the soil, for the determination of shear strength. It was found that the shear strength increases significantly with an increase in the fibre content. Moreover, from study, it was also observed that 0.6% sugarcane waste fibre gives the maximum strength.

The advantages of the considered fibres include low density, easy availability, low (zero) price and minimal pre-treatment to increase durability of fibre. Further, fibres are biodegradable and carbon dioxide neutral and their energy can be recovered in an environmentally acceptable way. The main advantage of these materials is that they are locally available and cost effective.

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