



Article Shear, Consolidation Characteristics and Carbon Footprint Analysis of Clayey Soil Blended with Calcium Lignosulphonate and Granite Sand for Earthen Dam Application

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Abstract: Soil is a composite material of great interest to civil engineers. When the quality of such composite soils is poor, ground improvement techniques must be adopted to withstand the design load of superstructure. Existing soil stabilizers include lime and cement; however, their environmental safety and sustainable use during stabilization have been receiving increasing attention in recent years. This study investigated the use of granite sand (GS) and calcium lignosulphonate (CLS) as sustainable stabilizers that could be blended with clayey soils. The considered dosages of GS were 30%, 40% and 50%, and those of the CLS were 0.25%, 0.5%, 1% and 1.5%. Direct shear and consolidation tests were performed on the GS-CLS blended soil samples that were cured for 7 and 14 days. The amended stabilizers improved the shear parameters and consolidation characteristics at an optimum dosage of 30% GS and 0.5% CLS. Maximum improvements of 84% and 163% were observed in the cohesion and angles of internal friction, respectively. A significant change was also observed in the consolidation characteristics, making them practically applicable. The soil hydraulic conductivity was reduced by 14%, and the coefficient of consolidation increased by 203% for 30% GS and 05% CLS. Carbon footprint analyses were performed on the soil composition that would be best-suited for a typical homogenous earthen dam section. The results showed that the use of GS and CLS together reduced the carbon emissions by 6.57 and 7.7 times, compared to traditional stabilizers, such as cement and lime.

Keywords: calcium lignosulphonate; carbon footprint analysis; clay; consolidation; direct shear test; granite sand

1. Introduction

Clayey soils are frequently characterized as weak soils since they undergo dramatic volume changes in the presence of water. If these soils are encountered at the site, they will be replaced, or their properties will be improved. These situations are relatively common, and in such cases, soil stabilization will be the best method to achieve the required soil properties.

Improving soil properties by incorporating certain additives is known as soil stabilization. It can be subdivided into two primary techniques: mechanical and chemical stabilization. Chemical stabilization employs a variety of chemical processes in which the added materials will interact with the soil's minerals and composition to improve its engineering properties. Mechanical stabilization can be achieved by mixing various materials that will change the soil gradation and improve the soil properties. Depending on the type of property that requires improvement, one can choose from a wide range of stabilizers that are currently available on the market. Conventional admixtures consisting of lime, cement and fly ash may act as binders, enhance the soil strength and control its swelling.



Citation: Varsha, B.; Moghal, A.A.B.; Rehman, A.U.; Chittoori, B.C.S. Shear, Consolidation Characteristics and Carbon Footprint Analysis of Clayey Soil Blended with Calcium Lignosulphonate and Granite Sand for Earthen Dam Application. *Sustainability* **2023**, *15*, 6117. https://doi.org/10.3390/su15076117

Academic Editors: Sayanthan Ramakrishnan, Sathees Nava and Kirubajiny Pasupathy

Received: 28 February 2023 Revised: 29 March 2023 Accepted: 30 March 2023 Published: 1 April 2023



Copyright: © 2023 by the authors. Licensee MDPI, Basel, Switzerland. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC BY) license (https:// creativecommons.org/licenses/by/ 4.0/). Goodarzi et al. [1] investigated the stabilization of expansive clay mixed with cement and silica fumes; unconfined compressive strength (UCS) and swelling tests were performed on the samples, revealing a 35% increase of the strength and a 50% decrease of the swelling potential of the modified clays. Clayey soil treated with cement and volcanic ash enhanced the UCS and California bearing ratio (CBR) of samples cured for 28 days, by 83% and 126%, respectively [2]. Moghal et al. [3,4] performed consolidation tests with accelerated loading on two different soils containing lime as a stabilizer. The authors showed that the final void ratio value increased with the loading duration, and the addition of lime resulted in the formation of a cementitious gel which held the particles together, thereby providing resistance against compression. Although these traditional admixtures typically improve the engineering properties of weak soils, they may alter the groundwater quality and limit plant growth, therefore affecting the environment [5]. Indeed, the use of cement in ground improvements contributes around 2% of the total global CO₂ emissions [6]. Moreover, the addition of cement and lime to soils permanently changes their structure by producing pozzolanic reactions that are irreversible [7]. Cement-stabilized soils are prone to cracking because of their significant shrinkage, while lime-stabilized soils are not resistant to water and frost [7–9]. These limitations have led researchers to seek innovative techniques for stabilizing soils. Sustainable materials, such as fibers, granite dust, biopolymers, geopolymers, coal gangues, fly ash, calcium carbide residues and microbially induced calcite precipitations (MICP), which are byproducts of industrial waste, are among the materials that have received considerable scientific interest [10].

Stabilizers are chosen so that, initially, the mechanical enhancement of soil properties will be achieved by making the soil texture well-graded by incorporating soil particles of missing sizes. Polymers, wood fibers, glass and plastics are such stabilizers that can mechanically reinforce soils. Muawia and Moghal [11] and Shaker et al. [12] performed a series of direct shear, compaction, hydraulic conductivity and linear shrinkage tests on two different high plastic clayey soils stabilized using two types of fibers, i.e., fiber cast and fiber mesh. These studies concluded that the fiber cast and fiber mesh were equally effective in lowering the linear shrinkage behavior of the soils, but the fiber mesh provided a better interlocking of the particles than the fiber cast. For fine-grained soils, such as clays, adding sand will improve the soil gradation, and the formed mix will possess both cohesion and friction [13]. However, excessive river mining typically renders it difficult to acquire natural sand for stabilization. Hence, granite sand (GS), a by-product of quarry industries, is considered the best alternative for natural sand in soil stabilization, helping to avoid the detrimental environmental effects caused by excessive river mining. Indeed, its use as a stabilizer presents advantages over its discarding as waste. Various studies have investigated the use of GS as an effective soil stabilizer. Eltawati et al. [14] added granite powder to an intermediate plasticity clay (CI) soil, which yielded a 2.8 times better shear strength than untreated soil (at 8%). The inclusion of granite powder into the soil also enhanced its CBR value from 3.65% to 16.5%. Sudhakar et al. [15] observed that the swell potential of an expansive clay decreased by 51.32%, with the addition of 15% of quarry dust, and the authors also noted an enhancement in the clay's UCS value. The performance of a GS will depend on the size of the soil particles used in the stabilization, with finer particles having a greater influence on the latter than larger ones [16]. Although granite dust will increase the density and shear strength of a soil mix, it will not always suffice to help reach the required soil criteria, and hence, the soil mix will require a binding agent that can keep the heterogeneous soil mix together [17].

Lignosulphonate is a byproduct of the paper manufacturing and wood processing industries, with an estimated global annual production of 50 million tons. This supports its tremendous utilization potential regarding biodegradable, non-hazardous, non-toxic and non-corrosive properties [18–20]. In general, only a small portion of wood supplies will be used for manufacturing pulp and paper, with the remainder being abandoned as waste. This discarded waste comprises nearly 25% of municipal solid waste in landfill sites [21,22]. Due to their unique properties, these wastes can be used in a wide range of applications,

such as animal feed, pesticides, surfactants, additives in oil drilling, stabilizers in colloidal suspensions and plasticizers in concrete admixtures [23]. Indraratna et al. [24] concluded that the ultimate strength and stiffness of treated soils increased with the curing period and dosage of lignosulphonate. However, the dosage of lignosulphonate will depend on the percentage of fines in the soil. Excess lignosulfonate will lead to polymer chain breakage [25]. The presence of lignosulfonate reduces the adsorbed moisture content in treated soils and hence reduces their swelling potential [26].

Sustainability is critical considering the growing climate issues globally. The construction industry alone contributes 25% of the world's total carbon emissions, as reported by the Intergovernmental Panel on Climate Change (IPCC) [27]. Considering the complexity of soils, only a few studies have assessed the carbon footprint of geotechnical works, such as landfill capping layers and ground improvement works [28].

In this study, the selected soil was replaced by GS to improve their shear and performance characteristics. However, this replacement could adversely affect the soil cohesion as the percentage fines of inert material, i.e., GS, increases. Hence, a chemical binder CLS was added to the treated soils to improve their cohesion. Carbon footprint analyses (CFA) were performed to evaluate the carbon emissions from a typical homogenous earthen dam constructed using a best-performing mix of GS and CLS. The results obtained and the mechanisms involved are detailed in the following sections.

2. Materials and Methods

2.1. Clay

The studied soil was acquired from the National Institute of Technology, Warangal, Telangana. The material was tested for its index properties and shear parameters, and the results are reported in Table 1. The particle size distribution of the soil is shown in Figure 1. The liquid limit and plastic limit of the soil tested according to ASTM 4318-17 [29] were found to be 45% and 23.28%, respectively. According to these findings, the soil was classified as a CI soil, as per the ASTM D2487-06 [30] (Unified Soil Classification System) (Table 1). The chemical composition of the selected soil is presented in Table 2.



Figure 1. Particle size distribution curves for clay and GS.

Characteristics	Soil	GS	Code
Specific gravity	2.62	2.70	ASTM D854-14 [31]
% fines	57	13	ASTM D422-63 [32]
USCS classification	CI	SP-SM	ASTM D2487-06 [30]
Maximum dry density (kN/m ³)	17.9	20.3	ACTN (DC09 12 [22]
Optimum moisture content (%)	18.1	7.9	ASTM D698-12 [33]
Cohesion (kPa)	20.0	19.3	
Angle of internal friction (°)	18.4	47.9	ASIM D3080-11 [34]
Note: USCS_Unified Soil Classification System:	CI_Intermediat	o Compressible (Jaw

Table 1. Properties of selected soil and GS.

Note: USCS—Unified Soil Classification System; CI—Intermediate Compressible Clay

Table 2. Chemical composition of the selected soil.

Chemical Composition	Value (%)
Silica (SiO ₂)	53.51
Alumina (Al_2O_3)	21.93
Ferric Oxide (Fe_2O_3)	9.34
Calcium Oxide (CaO)	2.98
Magnesium Oxide (MgO)	2.65
Titanium Oxide (TiO ₂)	1.37
Sodium Oxide (Na ₂ O)	1.03
LOI	4.8

2.2. Granite Sand

Granite sand is a byproduct of the quarry industry obtained from the primary crushing of aggregates. The GS used in this study was sourced from a quarry in Gudipadu, Telangana. It is a non-plastic substance that was dumped in significant amounts near the quarry sites. The material was characterized by a rugged surface and gray color, as shown in Figure 2a. The characteristics of the GS are listed in Table 1, and the particle size distribution is depicted in Figure 1. With 13% of fines and 86% of a sand fraction, it was classified as a poorly graded silty sand (SP-SM) as per ASTM D2487-06 [30]. Compared to natural sand, GS typically has higher shear strength and better permeability [16].



Figure 2. (a) Granite Sand (b) Calcium Lignosulphonate.

2.3. Calcium Lignosulphonate

The CLS used in this work was acquired from a local vendor, Aditya Chemicals, from the Hanamkonda region of Telangana. The CLS was yellowish-brown in color, as depicted in Figure 2b. It consists of a lignin-based polymeric stabilizer consisting of sulphonate, phenyl hydroxyl and alcoholic hydroxyl as a hydrophilic group and carbon chain under a hydrophobic group. The pH of CLS typically ranges from 4 to 6 [24,35,36]. Unlike traditional admixtures like cement and lime, lignosulphonate is a non-toxic and non-corrosive material, making it an eco-friendly soil stabilizer [36]. Indeed, Peric et al. [37]

stated that replacing traditional stabilizers with lignin materials would reduce man-made CO₂ emissions.

2.4. Mixing Strategy and Sample Preparation

The studied samples are prepared by replacing the clay (C) with GS in ratios of 70:30, 60:40 and 50:50. The dosages and mixing strategy were chosen following [38,39]. A lumpfree solution of CLS was prepared by adding a measured amount of water (optimum moisture content, OMC of soil) to the CLS (L) in proportions of 0.25%, 0.5%, 1% and 1.5% by the total weight of the CG mixture, as shown in Figure 3. These dosages of CLS were fixed with reference to [10,25]. All combinations of clay–GS–CLS (CGL) mixtures in this study are presented in Table 3. However, the dosage of the CLS depended on the percentage of fines in the soil [25]. All samples were prepared at the OMC and MDD (maximum dry density) of clay. The CGL mixtures were then left to mellow for 24 hrs. Mellowing allowed the CLS to spread across the soil uniformly, enhancing its efficiency in forming the polymer chains [40]. The mellowed soil was then molded into samples of 6 cm \times 6 cm \times 2.5 cm in size for direct shear tests as well as 6 cm diameter and 2 cm in height for consolidation tests. The samples were covered with a damp gunny sack and cured for 7 and 14 days in sealed plastic bags. They were weighed before and after curing to track any changes to their moisture contents. The samples were discarded if the loss of water content after the curing period exceeded 0.5%.



Figure 3. Mixing strategy involved in the study.

2.5. Direct Shear Test

Direct shear tests were conducted on the CGL mixtures under consolidated drained conditions, as per ASTM D3080 [34]. The cured samples were transferred to a direct shear box and left for consolidation under normal stress. Following the complete consolidation of the samples under that particular normal stress, the samples were subjected to shearing until they failed. The test was conducted at three different normal stresses: 100 kPa, 150 kPa and 200 kPa.

Description	Dosages (%)		
	Clay	GS	CLS
С	100	0	0
CG1	70	30	0
CG2	60	40	0
CG3	50	50	0
CG1L1	70	30	0.25
CG1L2	70	30	0.5
CG1L3	70	30	1.0
CG1L4	70	30	1.5
CG2L1	60	40	0.25
CG2L2	60	40	0.5
CG2L3	60	40	1.0
CG2L4	60	40	1.5
CG3L1	50	50	0.25
CG3L2	50	50	0.5
CG3L3	50	50	1.0
CG3L4	50	50	1.5

Table 3. Proportions of materials used in the study.

2.6. One-Dimensional Consolidation Test

One-dimensional consolidation tests were performed on the CGL mixtures according to ASTM D2435 [41], ignoring the rebound path. After curing, the prepared samples were placed in a consolidation cell with filter papers and porous stones at the top and bottom of the soil specimens. This entire setup was then placed into a loading frame and flooded with water to completely saturate the specimens. A seating pressure of 6.25 kPa was applied initially and incremented to 400 kPa, maintaining the load increment ratio as unity. The dial gauge readings were taken over 24 h for each load increment to plot the consolidation curves.

The coefficients of consolidation and hydraulic conductivity of the soils were calculated using Equations (1) and (2):

$$C_v = \frac{T_v d^2}{t} \tag{1}$$

$$k = C_v m_v \gamma_w \tag{2}$$

where C_v represents the coefficient of consolidation, T_v is the time factor, d is the drainage path and t is the time for consolidation. The hydraulic conductivity is represented by k; m_v is the coefficient of volume compressibility; and γ_w is the unit weight of water.

2.7. Scanning Electron Microscopy Studies

A ZEISS scanning electron microscope (SEM) is used to study the microstructures within the best-performing GS mixtures with varying dosages of CLS cured for 14 days.

2.8. Carbon Footprint Analysis (CFA)

Various gases in nature have the potential to contribute to global warming. Considering all of these gases in assessing a project's carbon footprint would make the analysis complex. Hence, Hammond and Jones [42,43] converted each of these gas emissions into one equivalent unit of CO_2 , called an embodied carbon equivalent factor (ECF). These normalized ECFs were considered in this study to assess the carbon emissions emitted during various construction phases.

All the steps involved in the construction process, such as acquiring the materials, hauling the materials to the site and the site operations, were considered for assessing the carbon emissions. The approach considered in this study was adopted from [28,44–47]. The following phases were considered in this study:

Phase 1: Raw materials considered for the project. Phase 2: Procurement and haulage of materials. Phase 3: Site operations involved in the construction.

3. Results and Discussion

3.1. Direct Shear Test

Direct shear tests were carried out on samples that had been cured for 7 and 14 days. Each sample was sheared at a 1 mm/min strain rate, as per ASTM D3080 [34]. Figure 4 shows the variations of the cohesion and angles of internal friction in the stabilized soil matrices with varying proportions of CLS and GS. The mechanisms involved and stress-strain behavior of the stabilized soil samples are depicted in Figures 5 and 6, respectively. The value of the brittleness index was also calculated from the peak stresses determined by the direct shear tests.



Figure 4. Cohesion and angle of internal friction of soil with varying dosages of GS and CLS cured for 7 days and 14 days.



Figure 5. Soil agglomerate along with peripheral bonding, basal bonding and excess CLS.

3.1.1. Effect of GS on Cohesion and Angle of Internal Friction

The replacement of the clay particles with GS reduced the cohesion of the soil and improved its angle of internal friction. Indeed, non-plastic GS fines in the soil matrix replaced the plastic fines of clay. This resulted in a reduction of the cohesion with an increase of the GS. Since the GS had a greater angle of internal friction, the replacement resulted in an increment of the angle of internal friction for the mixtures. These changes in cohesion values and angles of internal friction are depicted in Figure 4.



Figure 6. Stress-strain curves of soil with varying dosages of CLS and GS after 7 days.

3.1.2. Effect of CLS on the Cohesion and Angle of Internal Friction

The cohesion of the clay samples increased with an increase of the CLS up to 0.5%. Cohesion was reduced at greater dosages. Unlike conventional stabilizers, such as cement and lime, that form the pozzolanic reactions mentioned by Barman and Dash [7], the addition of CLS prompted the agglomeration of the CG mixture by forming basal and peripheral bonding between the particles of clay and the GS [26,40]. These flocs were further linked by polymer chains formed by the CLS, as shown in Figure 5. The initial dosage of CLS was not enough to form flocs and polymer chains, and hence the improvement of cohesion was observed up to CLS dosages of 0.5%. A further increase of the dosage led to an increase of repulsive forces, thereby reducing the interactions between the soil particles due to excess CLS.

The formation of flocs in the soil samples, due to the addition of CLS, made the soil matrix denser, which improved the angle of internal friction of the soil. Beyond the optimum, excess CLS accumulated on the flocs and made their surface slithery, as shown in Figure 5. However, the angles of internal friction increased with an increase of the curing period.

3.1.3. Brittleness Index and Stress-Strain Curves

According to Bishop (1971), the brittleness index (I_B) characterizes the behavior of soil after reaching a peak stress value and identifies its nature, which is given by the expression:

$$I_B = \frac{\tau_p - \tau_r}{\tau_p} \tag{3}$$

where τ_p and τ_r represent the peak and residual shear stresses obtained from direct shear tests, respectively. An I_B value of one will correspond to the brittle behavior of the soil; a ductile behavior will be expected if the value is less than that. A slight increase in the brittleness index of the treated soil samples was observed relative to the untreated soil; however, the range of values (0.011 to 0.030) indicated that the soil samples were ductile when CLS was added to them. The same behavior was seen in the stress–strain curves, as shown in Figures 6 and 7. The difference between peak and residual stresses resulted in ductile failure. It was also evident that the peak stresses increased with the curing period.

A similar kind of behavior was previously observed by Ta'negonbadi and Noorzad [48] and Vydehi and Moghal [49] with CLS and biopolymers, respectively. Sariosseiri and Muhunthan [50], Jahandari et al. [51] and Oliveira et al. [52] observed that the soil samples considered in their studies exhibited a brittle behavior after the addition of traditional admixtures, such as cement, lime and fibers, respectively.



Figure 7. Stress-strain curves of soil with varying dosages of CLS and GS after 14 days.

3.2. One-Dimensional Consolidation Test

As mentioned previously, consolidation experiments were performed on the soil samples cured for 7 days and 14 days. The effects of the curing period, the GS and CLS contents on the coefficient of consolidation, hydraulic conductivity and void ratios of the soil, along with the relevant mechanism involved, are discussed in this section.

3.2.1. Effect of GS and CLS on Void Ratio

The replacement of clay particles with GS significantly affected the void ratio of the samples. As shown in Figures 8 and 9, the CG1 and CG2 series reached their maximum final void ratio with any dosage of CLS cured for seven days and 14 days, respectively, compared to their initial void ratio. Soils with higher GS contents showed a slight decrease in their void ratios, whereas soils with lower GS contents exhibited a high reduction in void ratios relative to their initial void ratio. This indicated an enhancement of the soil gradation with the addition of GS, as depicted in Figure 10, which was responsible for reducing the compressibility of soil with higher proportions of GS.

As shown in Figure 11a,b, the addition of an optimum amount of CLS led to the formation of polymer chains, whereas an excess of CLS led to repulsive forces being formed by the thin layer formed around the particles, as observed in Figure 11c,d. The same was depicted in the mechanism proposed in Figure 10. These repulsive forces likely reduced the interaction between the soil particles because of a dispersion effect [20]. At higher CLS contents, this dispersive effect was attributed to a higher compressibility. Hence, a greater reduction in the void ratio occurred. A similar behavior was observed in the graphs in Figures 8 and 9. The CLS was so effective when the samples were cured that it lowered the reduction in the final void ratios of the soil samples, making them resistant to compression over time.



Figure 8. Variation of void ratio versus consolidation pressure with varying dosages of CLS and GS after 7 days.



Figure 9. Variation of void ratio versus consolidation pressure with varying dosages of CLS and GS after 14 days.



Figure 10. Mechanism involved in consolidation process.



Figure 11. SEM images of 14 days cured samples. (a) Indication of polymer chains in CG1L2, (b) CG1L1, (c) CG1L2 and (d) CG1L4.

3.2.2. Coefficient of Consolidation and Hydraulic Conductivity

As shown in Figure 12, the variation of the dosage of GS in the soil led to proportional variations of the coefficients of consolidation and hydraulic conductivity of the samples. Increasing the proportion of GS in the soil matrix led to a greater replacement of the plastic fines with non-plastic fines. Soils with non-plastic fines take less time to reach primary consolidation [53]. Unlike plastic ones, non-plastic fines do not hold water; whenever a load is applied, water in the pores will be rapidly expelled, increasing the coefficient of consolidation of a sample. Since GS is more permeable than clay, the soil matrix behavior changes slightly towards GS, attributing to the higher hydraulic conductivity values. Hence, the soil samples with 30% GS exhibited a smaller coefficient of consolidation and hydraulic conductivity.

The coefficient of consolidation initially increased for a CLS dosage of 0.5%. The initial dosages of CLS brought the soil particles closer together by forming bonds and polymer chains, as observed in Figure 11b,c. Later, this value decreased when the dosage was increased. Higher dosages of CLS led to the formation of a thin film around the soil particles, as shown in Figures 10 and 11c,d. This film restricted the water dissipation from the pores. Hence, at higher dosages, the coefficient of consolidation and hydraulic conductivity of the soil samples decreased.



Figure 12. Coefficient of consolidation and hydraulic conductivity of soil with varying dosages of GS and CLS after 7 and 14 days.

3.3. Recommended Application from the Results

When used as stabilizers, the GS and CLS improved the geotechnical properties of the studied soil samples, enhancing their potential use in practical applications. The curing period increased the effectiveness of CLS. When present in the proposed proportions, the GS and CLS enhanced the soil properties to reach the requirements of small dam embankments to be used as homogenous dam embankments, impervious cores and blankets as per IS 12169-1987 [54] and IS 1498-1970 [55]. The soil sample with 30% GS and 0.5% CLS was considered the best-performing mixture.

3.4. Carbon Emissions Resulting from the Construction of an Homogenous Earthen Dam

A typical homogenous earthen dam, as per IS 12169-1987 [54], was considered in this study. As seen in Figure 13, the dam's height was 5 m; its top width was 2 m; and its slope was 1:2 for both the upstream and downstream sides [54]. As suggested in the above sections, the soil with 30% GS and 0.5% CLS was considered the most appropriate for CFA. The soil mixture was compacted with a water content of 18.1% to reach a uniform density of 17.9 kN/m³.



Figure 13. Schematic diagram of homogenous earthen dam.

3.4.1. Phase 1: Estimating the Embodied Carbon Emissions from the Materials

The embodied carbon emissions from the clay, GS, CLS and water were estimated using the ECFs provided by [28,42,43]. All materials were taken and mixed as per the mixing strategy described in the methodology. Table 4 reports the values of Phase 1 carbon emissions. The calculations are provided in Appendix A.

Phase 1	Material	Amount (m ³)	Unit Weight (t/m ³)	Weight (t)	ECF	CO ₂ e (t)
	Clay	60,000	1.79	75,180	0.0056	421.01
Embodied	GS	60,000	1.79	32,220	0.0052	167.54
carbon of the	CLS	60,000	-	537	0.2000	107.40
material	Water	19439.4	1	19,439.4	0.0010	19.43
Total CO ₂ e (t) emission in Phase 1					715.38	

Table 4. Embodied carbon emission from Phase 1.

3.4.2. Phase 2: Estimating Embodied Carbon Emissions Resulting from Procurement and Haulage of Materials

Carbon emissions from the procurement and transfer of the materials to the site were quantified. A pickup excavator with a 10 ton/lit capacity for procurement and a heavyduty dumper with a 25 ton/lit capacity for haulage were considered. The haulage distance was considered to be 1 km (to and fro) for ease of quantification. The ECF values were considered for the fuel required for these vehicles, which were sourced from [28,56,57]. Table 5 lists the values of embodied carbon emissions produced by the procurement and haulage of the materials phase. The emissions from this phase were highly influenced by the vehicle capacities, haulage distance and type of fuel used by the vehicles.

Table 5. Embodied carbon emission from Phase 2.

Phase 2	Process	Vehicle	Capacity (t)/L	Trips	Total Fuel (L)	ECF	CO ₂ e (t)
	Clay Procurement	Pickup excavator	10	7518	7518	3.25	24,433.50
Excavation and	GS Procurement	Pickup excavator	10	3222	3222	3.25	10,471.50
Loading	CLS Procurement	Pickup excavator	10	54	54	3.25	175.50
		Total CO ₂ e (t) emis	sion in excavation a	nd loading p	hase		35,080.50
Phase 2	Process	Vehicle	Capacity (t)/L	Trips	Total Fuel (L)	ECF	CO ₂ e (t)
	Clay Haulage	Heavy duty dumper	25	3008	1504	3.25	4888
Haulago	GS Haulage	Heavy duty dumper	25	1289	645	3.25	2096.25
Taulage	CLS Haulage	Heavy duty dumper	25	22	11	3.25	35.75
	-	Total CO ₂ e	(t) emission in haula	age phase			7020
		Total CO ₂ e (t) em	ission in Phase 2				42,100.5

Note: Total fuel is calculated for half trip of a unit distance.

3.4.3. Phase 3: Estimating Embodied Carbon Emissions of the Site Operations

The carbon emissions produced by various operations taking place at the site were estimated. A bulldozer with a 10 ton/lit capacity for spreading soil, a slurry mixer with a 0.5-ton capacity for mixing CLS, a distributor truck with a 7000 L capacity for spraying CLS and a smooth wheel roller with a 12 ton/lit capacity for compacting soil were considered in the analysis. Table 6 reports the carbon emissions obtained during Phase 3. Similar to

Phase 2, the vehicle capacities and number of trips affected the total carbon emissions in this phase. The calculations and total carbon emissions from all three phases are reported in Appendix A and Table 7, respectively.

Table 6.	Embodied	carbon	emission	from	Phase	3.
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Phase 3	Process	Vehicle/Machine	Capacity	Trips	Total Fuel (L)	ECF	CO ₂ e (t)
	Spreading	Bulldozer	10 t/L	10,740	10740	3.25	34,905
	Mixing of CLS	Slurry mixer	0.5 t (50 lb)	1074	1074	3.25	3490.50
Site operations	Spraying of CLS	Distributor truck	7000 L	2.7	2.7	3.25	8.93
	Compaction	Smooth wheel roller	12 t/L	8950	8950	3.25	29,087.50
Total CO ₂ e (t) emission in Phase 3						67,491.93	

Phase	Operation	Embodied Carbon (CO ₂ e/t)
Phase 1	Material	715.38
Phase 2	Procurement	35,080.50
	Haulage	7020
Phase 3	Site operation	67,491.93
Total CO ₂ e (t) emission from all phases		110,307.81

Table 7. Embodied carbon emission with various stabilizers.

3.4.4. Comparison of Carbon Emissions of GS and CLS with Traditional Stabilizers

The carbon emissions for the proposed earthen dam with cement and lime used as stabilizers were considered with optimum dosages of 4% and 6%, respectively; these emissions are listed in Table 8 [58,59]. Since Phases 2 and 3 were influenced by vehicle capacities, haulage distance and the type of fuel used by the vehicles, only Phase 1 was considered for the comparison to understand the influence of the different materials clearly. The carbon emissions with 4% cement and 6% lime contributed 45% and 53% of the total emissions, whereas combined GS and CLS contribute 3%, as shown in Figure 14. The ECF values for lime and cement were considered to be 0.76 and 0.95, respectively [28,42,43].



Figure 14. Comparison of carbon emissions of GS and CLS with cement and lime.

Material	Dosage (%)	Quantity Required (t)	Carbon I ECF	Emissions CO ₂ e (t)
Granite Sand (GS) Calcium	30	32,220	0.0052	167.54
Lignosulphonate (CLS)	0.5	537	0.2000	107.40
Total CO_2e (t) emission from GS and CLS274.94				274.94
Cement Lime	4 6	4296 6444	0.9500 0.7600	4702.07 5518.31

Table 8. Embodied carbon emission comparison of GS and CLS with cement and lime.

4. Conclusions

This study focused on improving the shear and consolidation characteristics of clayey soil stabilized with GS (30%, 40% and 50%) and CLS (0.25%, 0.5%, 1% and 1.5%) by conducting direct shear and consolidation tests on samples cured for 7 days and 14 days. Based on the results obtained, the following conclusions were drawn:

- The angle of internal friction of soil samples increased due to the replacement of plastic fines (clay) with non-plastic fines (GS).
- In the presence of CLS, cohesion values increased due to the formation of basal and peripheral bonding with clay and GS particles.
- The maximum improvements in the angles of internal friction and cohesion were 163% and 84%, respectively, with 30% GS and 0.5% CLS (CG1L2).
- The replacement of plastic fines with GS made the soil matrices more permeable and increased their value of hydraulic conductivity and coefficient of consolidation. However, this increment in soil permeability was controlled by CLS due to the formation of a thin film around individual particles.
- The cohesion, coefficient of consolidation and hydraulic conductivity of soil samples tended to decrease beyond optimum (0.5% of CLS) due to repulsive forces that limit particle interaction.
- The replacement of soil with GS and formation of flocs with CLS improved soil gradation, resulting in a smaller reduction of the void ratio, thus making those soil samples resistant to compression.
- The sample curing duration positively enhanced both shear and consolidation characteristics; the 30% GS and 0.5% CLS sample was determined as the optimum mixture.
- The CFA carried out for a typical earthen dam section revealed that the addition of GS and CLS to the studied soil could reduce the associated carbon emissions by 6.57 and 7.7 times compared to traditional stabilizers like cement and lime, respectively.

This study showed the efficacy of GS and CLS as sustainable binders in improving selected engineering properties of soils. The addition of GS and CLS to soils could therefore be useful for embankments, impervious cores and blankets. These binary stabilizers were proven to emit fewer carbon emissions than conventional stabilizers, such as cement and lime, and accordingly, they would be more sustainable and reduce the environmental impacts associated with their dumping. Future applications could also be explored with undrained testing data.

Author Contributions: Conceptualization, methodology, B.V. and A.A.B.M.; formal analysis, investigation, B.V.; resources, A.A.B.M. and A.U.R.; data curation, B.V. and A.A.B.M.; writing—original draft preparation, B.V. and A.A.B.M.; writing—review and editing, B.V., A.A.B.M., A.U.R. and B.C.S.C.; visualization, A.A.B.M.; supervision, A.A.B.M.; project administration, A.A.B.M.; funding acquisition, A.U.R. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by King Saud University through Researchers Supporting Project number (RSPD2023R701), King Saud University, Riyadh, Saudi Arabia.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: Not applicable.

Acknowledgments: The authors are thankful to King Saud University for funding this work through the Researchers Supporting Project number (RSPD2023R701), King Saud University, Riyadh, Saudi Arabia.

Conflicts of Interest: The authors declare no conflict of interest.

Appendix A

A typical homogenous earthen dam is considered to assess the carbon emissions emitted when constructed with GS (30%) and CLS (0.5%). Dimensions of the section of 1-km stretch considered are as follows:

Height	: 5 m			
Top width	: 2 m			
Side slopes $(u/s \text{ and } d/s) : 1:2$				
Bottom width	: 22 m			
Dosages and propertie	es of materials considered:			
Required density	$: 1.79 \text{ t/m}^3$			
Water content	: 18.1%			
Clay (C)	: 70%			
Granite Sand (GS)	: 30%			
Calcium Lignosulpho	nate (CLS): 0.5% of C–GS mix			
Volume of the earthen	dam and quantities of materials calculated from given data:			
Volume	$= 0.5 \times (Top width + Bottom width) \times Height \times Length$			
	$= 0.5 \times (2 + 22) \times 5 \times 1000$			
	$= 60,000 \text{ m}^3$			
Total Quantity of soil	= Density \times Volume			
	$= 1.79 \times 60,000$			
	= 107,400 t			
Clay (C)	$= 0.7 \times 107,400$			
	= 75,180 t			
Granite Sand (GS)	$= 0.3 \times 107,400$			
	= 32,220 t			
Calcium Lignosulpho	nate (CLS) = $0.005 \times 107,400$			
	= 537 t			
Water	$= 0.181 \times 107,400$			
	= 19,439.4 t			
Embodied carbon em	issions from Phase 1 (Materials):			
CO_2e from material = E	CF of material $ imes$ Quantity			
CO ₂ e from C	$= 0.0056 \times 75,180$			
	= 421.01 t			
CO ₂ e from GS	$= 0.0052 \times 32,220$			
	= 167.54 t			
CO ₂ e from CLS	$= 0.2000 \times 537$			
	= 107.40 t			
CO ₂ e from water	$= 0.0010 \times 19,439.4$			
	= 19.43 t			
Total CO_2e emitted from Phase1 = 715.38 t				
Embodied carbon em	issions from Phase 2 (Procurement and Haulage):			

A pickup excavator of 10 ton/lit capacity for procurement and a heavy-duty dumper of 25 ton/lit capacity for haulage are considered. The haulage distance is considered 1 km (to and fro) for easy quantification.

Total fuel for procurement	= Quantity/(Capacity of vehicle)
Total fuel for haulage	= $(0.5 \times Quantity)/(Capacity of vehicle)$

CO ₂ e from fuel	= ECF of fuel \times Total fuel
Procurement of C:	
Total fuel	= 75,180/10
	= 7518 L
CO ₂ e emitted	$= 3.25 \times 7518$
	= 24,433.50 t
Procurement of GS:	
Total fuel	= 32,220/10
	= 3222 L
CO ₂ e emitted	= 3.25 × 3222
-	= 10,471.50 t
Procurement of CLS:	
Total fuel	= 537/10
	= 54 L
CO ₂ e emitted	$= 3.25 \times 54$
	= 175.50 t
Haulage of C:	
Total fuel	$= 0.5 \times 75,180/25$
	= 1504 L
CO ₂ e emitted	$= 3.25 \times 1504$
	= 4888 t
Haulage of GS:	
Total fuel	= 0.5 × 32,220/25
	= 645 L
CO ₂ e emitted	$= 3.25 \times 645$
	= 2096.25 t
Haulage of CLS:	
Total fuel	$= 0.5 \times 537/25$
	= 11 L
CO ₂ e emitted	= 3.25 × 11
	= 35.75 t

Total CO₂e emitted from Phase 2 = 42,100.5 t

Embodied carbon emissions from Phase 3 (Site Operations):

A bulldozer of 10 ton/lit capacity for spreading soil, a slurry mixer of 0.5-ton capacity for mixing CLS, a distributor truck of 7000 L capacity for spraying CLS and a smooth wheel roller of 12 ton/lit capacity for compaction of soil are considered.

Spreading of soil (Bulldozer):

- r 0 (-	
Total fuel	= 107,400/10
	= 10,740 L
CO ₂ e emitted	$= 3.25 \times 10,740$
	= 34,905 t
Mixing of CLS (Slur	ry mixer):
Total fuel	= 537/0.5
	= 1074 L
CO ₂ e emitted	$= 3.25 \times 1074$
	= 3490.50 t
Spraying of CLS (Di	stributor truck):
Total fuel	= 19,439.4/7000
	= 2.7 L
CO ₂ e emitted	$= 3.25 \times 2.7$
	= 8.93 t
Compaction of soil (Smooth wheel roller):
Total fuel	= 107,400/12
	= 8950 L

$\begin{array}{rl} \text{CO}_2\text{e emitted} &= 3.25 \times 8950 \\ &= 29,087.50 \text{ t} \end{array}$ Total CO₂e emitted from Phase 3 = 67,491.93 t Total CO₂e emitted from all phases = 110,307.81 t

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