Assessment of Stabilized Silt Clay Sand with Oil Palm Fibre Bunch (OPFB) Local Fibre for Slope Foundations: A Case of Coastal Soils of Mombasa, Kenya

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Authors' contributions

This work was carried out in collaboration between both authors. Author JKM designed the study, performed the statistical analysis, wrote the protocol and wrote the first draft of the manuscript. Author EW managed the literature searches, analyses and reporting. Both authors read and approved the final manuscript.

Article Information

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ABSTRACT

Improvement of shear strength parameters is essential for designing the OPFB fiber mix with silt clay sand for slope stability. The objective of this study was to assess the stabilized silt clay with oil palm fibre bunch (OPFB) local fibre for slope foundation. Series of laboratory tests were conducted on various materials under study and the results revealed that, OPFB mix can be used as an additive to cement for purpose of improving engineering properties of the Silt Clay sand to cut down costs without compromising the set standards. It was established that, the shear strength parameters of the soil-fibre mixture (φ and C) can be improved significantly up to an optimum and reach a certain point where it starts to decline. The shear stress–strain curves obtained from the CU triaxial tests for reinforced sands with 30 mm fibre length together with those for unreinforced silty sand were compared; the result indicated that, fibre-reinforced specimen showed higher deviator stress at 0.25% fibre and reduces at 0.5% fibre. The strain corresponding to the peak

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deviator stress was increased by fibre content. Patterns of stress–strain curves for all reinforcedsamples indicated improvement in the deviator stress for all compositions and fibre content. Deviator stress of fibre-reinforced soil showed a slight increase with increasing pore pressure. The increase of the fibre content caused an increase in pore water pressure due to inclination of specimens to decrease the volume. Changes in the shear strength of fibre-reinforced soil indicated that soil strength parameters (internal friction angle φ' and cohesion C') increase as the internal friction surface increases between fibre and soil at certain point.

Keywords: Silty clay sand; fibre; slope foundation; reinforcement; oil palm fibre Bunch.

1. INTRODUCTION

A wide range of reinforcements have been used to improve soil performance. Increasing the soil strength has caused increased interest in identifying new accessible resources for reinforcement. Short discrete fibres made of polymeric or natural material have been used to improve the shear strength of soil [1,2,3]. Studies were performed recently using polymeric fibres [4,5,6,7,8]. It has been suggested that natural resources may provide superior materials for improving soil structure, based on their cost-effectiveness and environment friendly aspects [9]. Oil palm empty fruit bunch (OPEFB) fibre was chosen for this study due to its reliable strength and bulk availability in Malaysia. Oil palm belongs to the species Elaeis guineensis of the family Palmacea and originated in the tropical forests of West Africa. Currently under intensive industrial cultivation in Southeast Asia, oil palm is one of the largest crops in Malaysia and Indonesia [10]. OPEFB fibres are biodegradable and must be protected from any circumferential agents to ensure long-term performance. Modification of the fibre surface through physical and chemical methods reduces the hydrophilic nature of the fibre and decreases the rate of biodegradation in natural settings [11]. Coating of the OPEFB fibres with acrylic butadiene styrene (ABS) thermoplastic protected the OPEFB fibres from biodegradability. This study examines the effect of coating the fibres with acrylic butadiene Styrene (ABS) thermoplastic on fiber reinforcement performance. Randomly distributed fibre-reinforced soils have recently attracted increasing attention in geotechnical applications. Short discrete fibres can provide isotropic increase in the strength of the soil composite without introducing continuous planes of weakness. The discrete fibres are simply added and mixed randomly with soil, much like cement, lime, or other additives[6] Fibres can be used to combine with other admixtures such as cement and fly ash[12,13] to increase the shear strength and ductility of the soil. Fibre-reinforced soil is also suitable for failed slopes repair, where, the irregular shape of the soil patches limits the use of textile reinforcements, making the fibre reinforcement an attractive alternative. The use of fibre as reinforcement is a useful method for improvement and stabilization of subgrade in road construction [13].

Typically, unconfined compression strength (UCS) test were conducted to evaluate fibre reinforcement of soil [14]. Fibre inclusion can increase the peak shear strength, limit the post peak reductions in shear resistance and decrease the stiffness of the soil. According to [15] reported that the shear strength reinforced soil increases with an increase in fiber concentration and aspect ratio, but it also depends on the relative size of the grains and fiber length. The longer fibres contribute more to the composite strength than the shorter fibres in constant aspect ratio. Previous studies have shown that the concentration and length of the fiber have significant influence on the CBR value. [13] Studied the effectiveness of fiber reinforcement in sub grade soil along with fly ash. They reported that the coir fiber shows better resilient response with respect to cyclic loading against synthetic fiber owing to a higher coefficient of friction. According to [16], the effect of supplementing rice husk ash, pond ash, cement and fiber on the compaction and strength behavior of clay is to bring both economic and technological benefits as well as increasingly find use of locally available materials in synthesis of zeolite and geopolymers. In addition, the mixture decreases the maximum dry density and at the same time increasing in the optimum moisture content thus resulting to a satisfactory strength and can be used for embankments construction and stabilization of sub-grade soils. Several studies showed that the fatigue life is also improved by adding up the cement and fiber, and the method of stabilization using fibers (natural and synthetic fibers) will be more economical

because it is easily available [17,18]. Test results show that the engineering properties depend on many factors such as fiber content, lime or cement content, and length of curing. The proper curing has a great effect on the strength, and this is attributed to the hydration process of the cement with soil and fiber. In previous studies, similar strength improvement was reported [16].

Soil stabilization has often been the main concern of researchers in geotechnical sciences, and civil engineers have always looked for solutions to stabilize and sustain the soil, besides having an economical design [19]. A land-based structure of any type is only as strong as its foundation. This implies that, soil is an essential and critical element influencing the success of a construction project. Soil is either part of the foundation or one of the raw materials used in the construction process. Therefore, understanding the engineering properties of soil is crucial in obtaining strength and economic permanence. Soil stabilization is the process of maximizing the suitability of soil for a given construction purpose [20].

Improvement of soil engineering properties is an inevitable necessity, when the structures are founded on problematic soils. Expansive, collapsible, liquefiable, soluble, dispersive, silty fine sands and highly organic weak soils are the most serious kinds of problematic soils. Silty sand soils are kinds of the problematic soils which found in different areas of the world and are susceptible to collapse when come in contact with water.

Soil improvement can be undertaken by a variety of ground improvement techniques such as compaction, reinforcement, drainage, and addition of natural and synthetic materials or a combination of physical and chemical methods [21]. Chemical stabilization or addition of different natural and synthetic material to the soil has been experienced in recent years. Lime, cement and pozzolanic materials are the most common construction materials which are extensively used for stabilization of soils. Recently different modern technologies such as nanoparticles were used for stabilization of the soils [22]. According to [23] geotechnical and mineralogical properties of a lime treated high organic soft clay soil have showed that increasing the curing age and lime percent, unconfined compressive strength is increased. Furthermore, the required lime content for satisfactorily stabilization of the high organic soft clay was found to be 70%

2. EXPERIMENTAL WORK

2.1 Material Properties

The silty clay sand soil was used in this experiments which was obtained from coastal areas particularly in areas of Tsunza, Mwache and Mteza where deposition had been made and this has been classified as SM according to the Unified Soil Classification System (USCS) with ASTM D2487 [24]. The average diameter of the sand particles at D50 was 0.68 mm. Based on the gradation curve, [19] shown in Fig. 1, the coefficient of uniformity is 4.82, and the coefficient of curvature is approximately 1.12.
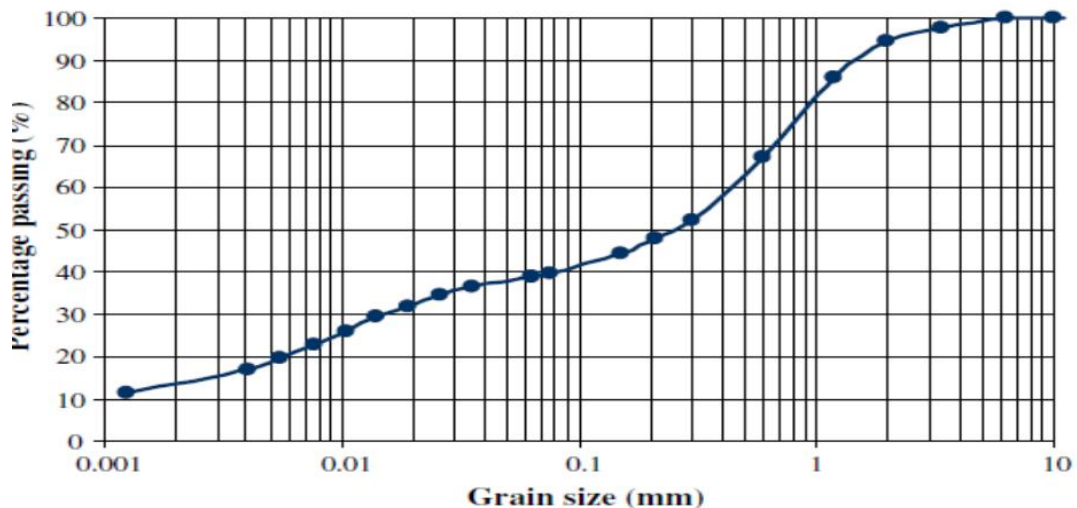

Fig. 1. Gradation curve for soil

Based on the secondary data obtained from previous studies indicated that sample soils collected from Mwache and Mteza were of different size as indicated in Table 1.

Tsunza is located where the Ndongo - Kundu bypass road will be constructed and most of its slopes and embankments will be filled with large amount of suitable materials either by borrowed or treated of the in-situ material to sustain slope stability Fig. 2. Fig. 3 shows the soil sample collected from Tsunza on 05th January 2020 for laboratory analysis.

Table 1. Summary of soil properties from Mwache and Mtezafor previous researcher

Sample	Particle size (mm) and distribution (%)		Texture class
Mwache	Sand (0.02-2.0 mm)*	68	Sandy loam
	Clay (<0.002mm)	14	
	Silt (0.00-0.02)	18	
Mteza	sand	56	Sandy clay loam
	Clay	32	
	Silt	12	

*N/B: *Particles greater than 2 mm are called stones, rocks, or gravels and are not considered to be soil materials*

Fig. 2. Location of soil sample collection; Tsunza

Fig. 3. Silt clay SAND soil sample collected from Tsunza

OPFB fibre is extracted from empty fruit bunches from oil palm by the retting process, as shown Fig. 4.

Fibers were subsequently cut to specified length of 30mm. The properties of these fibers were given as indicated in Table 2

Natural used cooking oil mixed with charcoal powder and wax (OCW) coating was used as means of protecting the OPEFB fibres from biodegradation. According to [25] the inclusion of randomly distributed fibers as reinforcing materials affects the consolidation settlement, hydraulic conductivity, swelling, shrinkage limit and desiccation cracking of the clay soils due to weather conditions.

An OCW solution was prepared in appropriate proportion that is sufficient for fiber to dry up. The cut OPEFB fibres will be incubated in the OCW solution and dried ready for use, see Fig. 5.

Table 2. Properties of OPFB fibers to be used in study

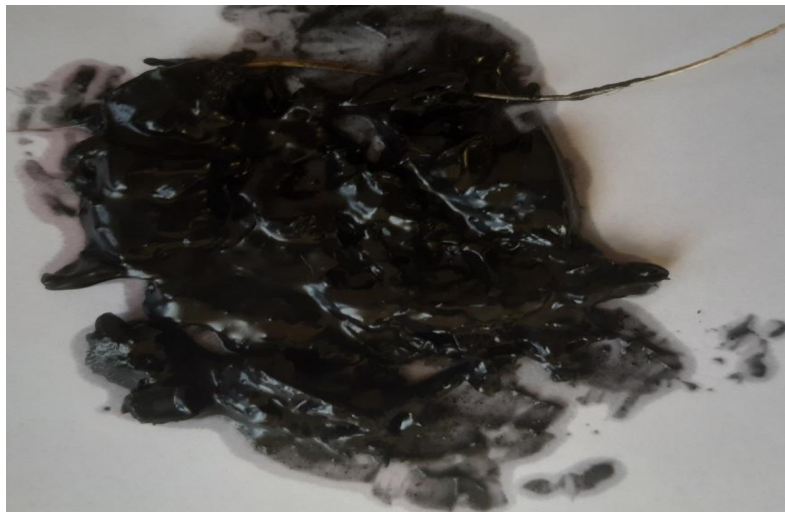| OPEFB fibre properties	Value
Specific gravity	1.46
Linear density (denier)	1650
Average diameter (mm)	0.40
Elongation at break (%)	15
Breaking tension strength (kPa)	2.83×10^5

Fig. 4. Oil palm empty fruit bunch fibres

Fig. 5. Natural oil mix with charcoal powder and wax (OCW)
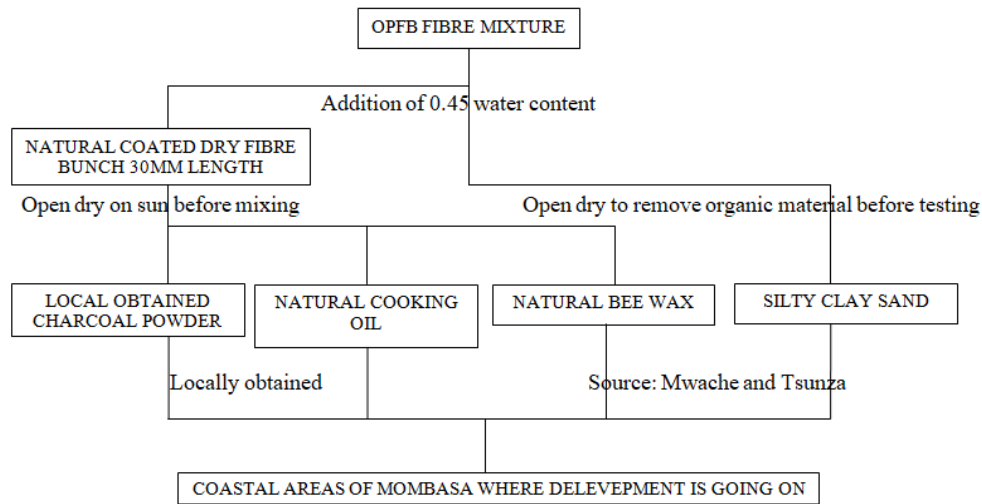
Fig. 6. Step by step flow diagram of the experimental work

2.2 Conceptualization of the Work

Obtained Silty Clay sand, 0%, 0.25%, and 0.5% of palm oil fibre and 0.3% of cement content were mixed with addition of 0.45 water content to form the mixture as shown in Fig. 6 chart for the shear strength analyses.

3. METHODS AND MATERIALS

The study covered the use OPFB fibres mixed with silt clay sand in order to determine the strength and mechanical behavior of randomly distributed fibre-reinforced soil. Effect of fiber coating with Natural used cooking oil mix with bee wax and charcoal powder for durability of fibre. A series of triaxial compression tests were performed under undrained loading conditions. In addition, OPFB fiber coated with Natural used cooking oil mixed with bee wax and charcoal powder were tested to determine the effect of coating on reinforcement. Inclusion of randomly distributed discrete fibers significantly improves the shear strength of silt clayey sand much more compared to untreated with fibres. Coating fibres increases interface friction between fiber and soil particles by increasing the surface area, reinforced silt clayey sand containing percentage content of coated fibers of 30mm length exhibited approximate increase in friction angle and in cohesion under undrained loading conditions compared to those of unreinforced silty clay sand.

3.1 Sample Preparation for Trial Tests

The samples were prepared as indicated in Table 3.a&b.Fibre-reinforced silty Clay sand was prepared with fibre contents of 0, 0.25% and 0.50% by weight of dry soil. The fibre content (f) was defined as: f ¼ Wf =Ws where Wf were the dry weight of the fibres, and Ws is the dry weight of sand. The tests were repeated using 30mm length fibre. Fibres were mixed with soil randomly for triaxial tests.

Table 3.a&b. Summary of neat soil properties used in the study

a)

OPEFB fibre properties	Value
Specific gravity	1.46
Linear density (denier)	1650
Average diameter (mm)	0.40
Elongation at break (%)	15
Breaking tension strength (kPa)	2.83×10^5

b)

OPEFB fibre properties	Value
Specific gravity Consistency limit	2.58
Liquid limit (%)	25.2
Plastic limit (%)	19.2
Plasticity index	6.1
USCS Classification SM	SM
Compaction Test:	
Optimum moisture content (%)	7.4
Maximum dry density (KN/m3)	18.70

Samples were compacted in three layers into a 50 mm diameter and 100 mm high cylindrical mould, with optimum water content (W) of 7.4% and dry density of 18.7 kN/m3 (i.e., a relative density of Dr ¼75%). The specimen preparation method was adapted from the standard compaction test method (BS1377: Part 8:1990.), the specimen was initially prepared at the optimum water content and the mixing of soil with

fibres was performed manually. After mixing, the fibre soil mixture was stored in a covered container for 24 h.

Triaxial tests were conducted to determine the stress–strain and strength characteristics of reinforced soils with various fibre contents of 0, 0%.25% and 50%. All specimens were fully saturated with a minimum measured B value of 0.97. Axial load applied under strain-controlled condition with a strain rate of 0.1 mm/min under the confining pressure (cell pressure) of s3 equal to 190kPa, 300 kPa and 400kPa to define the shear strength parameters (effective angle of shear strength (F0) and cohesion (c0)) for both the unreinforced and reinforced silt clay sand.

Specimens from drill cores are prepared by cutting them to the specified length and are thereafter grinded and measured. There are high requirements on the flatness of the end surfaces in order to obtain an even load distribution. Recommended ratio of height/diameter of the specimens is between 2 and 3.A membrane is

mounted on the envelope surface of the specimen in order to seal the specimen from the surrounding pressure media. Deformation measurement equipment mounded on the specimen and the specimen is inserted into the pressure cell whereupon the cell is closed and filled with oil. A hydrostatic pressure is applied in the first step. The specimen is then further loaded by increasing the axial load under constant or increasing cell pressure up to failure or any other pre-defined load level as indicated in Figs. 7 and 8

— Cell pressure up to 100 MPa
— Air-hole pressure up to 100 MPa
— Breaking stress
— Angle of friction
— Parameter of elasticity: E, v
— Post-failure

Specimen inserted in between the loading platens, with equipment for deformation measurement attached on the specimen, before the cell is closed.
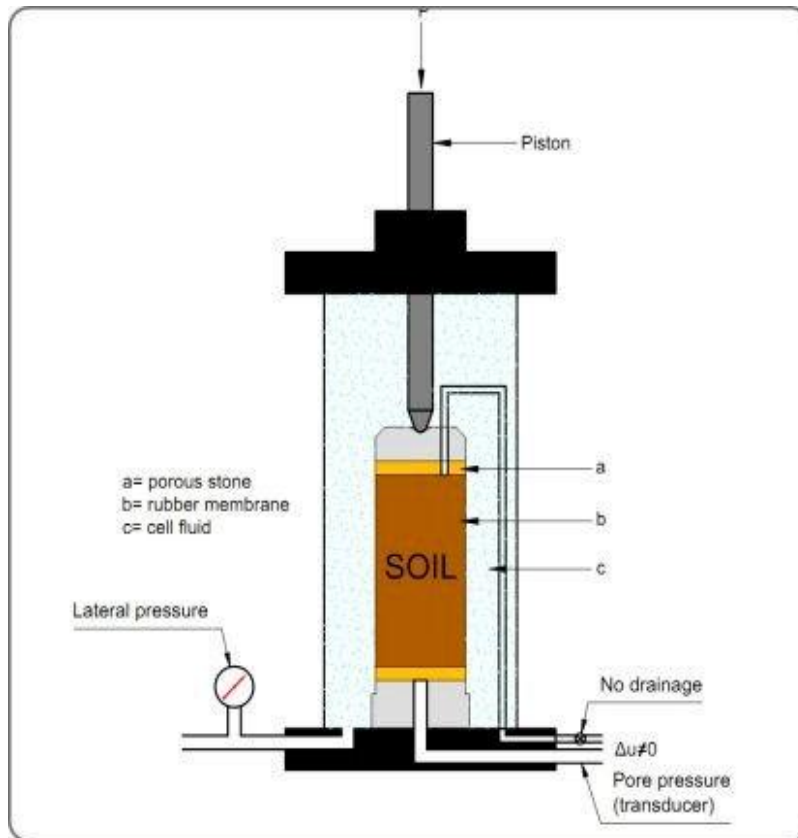

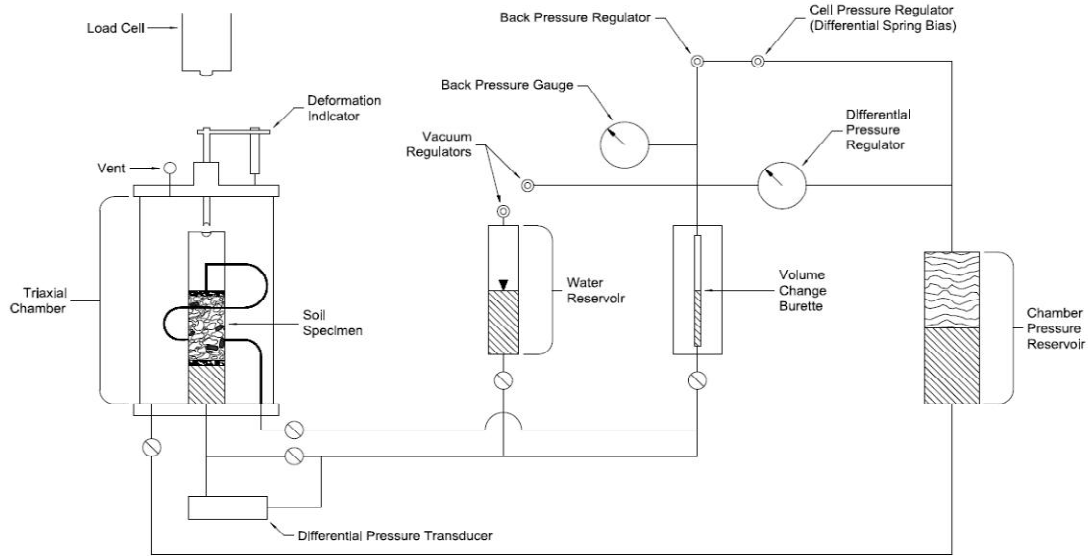**Fig. 7. Undrained triaxial test apparatus**

Fig. 8. Undrained triaxial test apparatus system

4. RESULTS AND ANALYSIS

OPFB treated Silt Clayey Sand was tried for its suitability for use as improved material for slope stability. Natural neat sample was found to be very weak to support structure at the slopes hence the researcher tried improving the soil by adding OPFB + cement in order to attain or meet minimum required shear strength and CBR, to be used in its treated form for slope stability after stabilizing the slope ready for structure foundation Table 4

4.1 Summary Results

The results for the tests conducted are presented below

4.2 Engineering Properties of the Neat Soils

The engineering properties of the neat soil were obtained by conducting Atterberg limits and particle size distribution analysis to classify its soil type.

4.3 Grading Characteristics

The grading curve in Fig. 9 is a graphical representation of the particle grading and distribution and was therefore useful in determining different sizes of soil samples for easy describing them.

From the grading curve, three characteristic sizes were derived.

— D10 = Maximum size of the smallest 10% of sample
— D30 = Maximum size of the smallest 30% of the sample
— D60 = Maximum size of the smallest 60% of the sample.

From these characteristics sizes, the following grading characteristics are defined:

1. Effective size = D_{10} mm.

2. Coefficient of Uniformity, Cu $= \dfrac{D_6}{D_{10}}$

3. Coefficient of Curvature (gradation) ,

Cc/Cg $= \dfrac{(D_{30})^2}{D_{60} \times D_{10}}$

4.3.1 Soil classification of collected sample

The silty clay SAND soil used in this experiment was obtained from coastal area of Tsunza and classified according to BS 1377: 1990: Part 2: Clause 9.

According to the chart below Fig. 10:

(a) (i) Percentage course to medium sand = 100-47 = 53%
(ii). Percentage fine sand = 47 – 15.4 = 31.6%
(iii) Percentage silt and clay = 10.8%
(iv) Percentage clay = 4.6%

Table 4. Summary of laboratory results

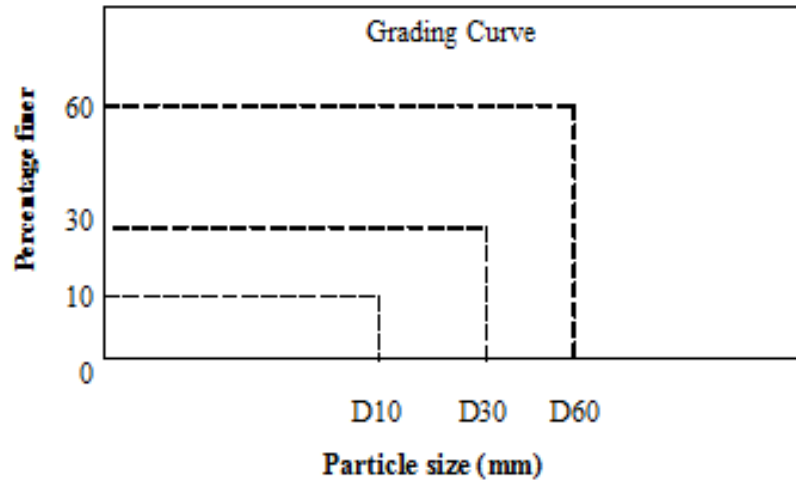| Silt clayey sand material state	Cement (%)	Coated Opfb Fiber (%)	Plasticity index	OMC (%)	MDD (Kg/m3)	CBR (%)	Total stress		Effective stress		Deviator stress Vs axial strain	Pore pressure Vs axial strain
							C (KN/m^2)	φ (°)	C' (KN/m^2)	φ' (°)	Kpa	Kpa
NEAT	0.00	0.00	6.10	7.40	1908	19.4	12	16	22	16	90	158
TREATED	0.30	0.25	-	7.00	1912	77.00	125	40	195	19	610	172
TREATED	0.30	0.50	-	6.70	1916	86.1	48	40	75	35	117	141

Fig. 9. Grading characteristics

Table 5. Summary of soil properties of neat sample collected from Tsunza

Sample	Particle size (MM) and distribution (%)		Texture class
TSUNZA	SAND (0.02-2.0MM)	84.6	Silt Clayey SAND
	SILT (0.002-0.02MM)	10.8	
	CLAY (<0.002MM)	4.6	

*Particles greater than 2mm are called stones, rocks or gravels and are not considered to be soil materials

(b) To determine whether neat soil is uniformly graded or well graded, Hazen proposed the following equations
Uniformity coefficient (Cu) = D60/D30
Where: D60 = 0.3, D30 = 0.03 and D10 = 0.17
There for Cu = 0.3/0.03 = 10> 4 Seems to be well graded soil.

(c) To determine the graduation of particles, Hazen proposed following equation
Coefficient of curvature (Cc) = $(D30)^2$/ (D60xD10)
Cc = $(0.17)^2$/ (0.3x0.03) = 3.2 > 3 the soil just pass through the boundary of uniformly distributed soil and well graded soils

The average diameter of the sand particles at D50 is 0.25 mm. Based on the gradation curve, the coefficient of uniformity is 10, and the coefficient of curvature is approximately 3.2.

4.3.2 Soil consistency of collected sample

These tests indicate the range of the plastic state, the range of water content between the liquid limits and plastic limits, which is an important measure of plastic behavior called the plasticity index (Ip) shown which lies less than 7 as shown in Table 6.

Table 6. Plasticity index

Plasticity index	Plasticity behavior
0	Non – plastic
< 7	Low plastic
7-17	Medium plastic
>17	Highly plastic

In this case Results for Plasticity index is 6.1% < 7 hence Low plastic

4.4 Determination of Engineering Properties of the Treated Soils Compare to Neat Samples

The required properties for the core construction soils were then identified from the test results soil stabilized with 0.3% Cement constantly throughout the experiment and varies percentage of OPFB fiber comparing with neat samples.

4.4.1 Compaction tests

The density and moisture content relationship was done to determine varies samples in comparison to Civil engineering properties Fig. 9 shows at optimum moisture of 45% at Maximum dry density.
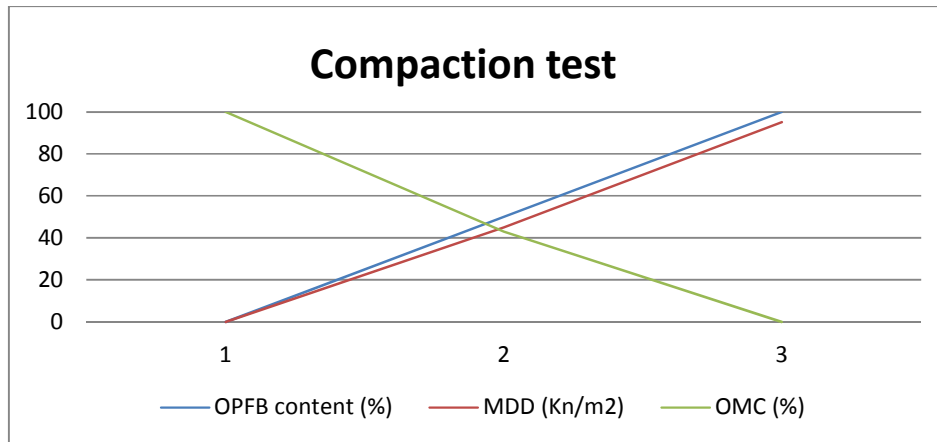
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Fig. 10. Proctor Compaction test Comparison shows water content required of 0.45

Table 7. Maximum dry density /moisture content relationship

Sample type	Neat	Treated	Treated
Sample No.	**1**	**2**	**3**
OPFB content (%)	0	0.25	0.5
MDD (Kn/m2)	1908	1912	1916
OMC (%)	7.4	7	6.7
Accumulative difference in Percentage			
OPFB content (%)	0	50	100
MDD (Kn/m2)	0	45	95
OMC (%)	100	43	0

In compaction of soils, the main aim is to keep the soil particles close together which leads to improve dry density of soil. The soil with maximum dry density is suitable for the several constructional purposes. But maximum dry density of soil through compaction will be possible at particular moisture content called optimum moisture content.

Generally dry soil contains soil particles which were not in contact with each other and when we try to compact this type of soil without water it becomes stiff and cracks and gaps was formed. When water was added to it, the water formed a thin film of layer around each soil particle and this film helps to increase the adhesive force between soil particles and water thus sticks together. Thereby the soil becomes denser under compaction. When increase of OPFB fiber there was an increase of MDD while OMC decreased hence affects the soil workability and the soil becomes stiff. The researcher here tried to compare these three samples of sand and determine its suitability and workability of the treated soil sample for construction consideration. According to the experiment the lines of OPFB and OMC passing each other at

the bar and were consider to the treated OPFB of 0.25% to be the best.

4.4.2 California Bearing Ratio (CBR)

The soil sample collected from Tsunza slope area was tested in the Geotechnical laboratory, CBR value of the neat sample was 19.4% for the soaked condition of 4 days at optimum moisture content and maximum dry density. After soil treatment the CBR increase to 77% and 86.1% for 0.25% and 0.5% fiber respectively with a constant addition of 0.3% cement admixture. Actual Dry density of the specimen was considered in this case due to the workability of the treated soils, as the CBR increases with the increase of OPFB fiber percentage, the workable dry density decreases and therefore affecting workability condition as indicated in Fig. 11.

4.4.3 Consolidated Undrained Triaxial (CU) test

A Series of triaxial compression tests were performed on fibre reinforced silt clay sand samples to evaluate improvement of soil strength. Consolidated undrained (CU) tests

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were carried out on fibre-reinforced silt clay sand soil according to BS 1377; PART 8:1990 standards. Triaxial tests were conducted to determine the stress–strain and strength characteristics of reinforced soils with various fibre contents of 0, 0%.25% and 50%. All specimens were fully saturated with a minimum measured B value of 0.97. Axial load applied under strain-controlled condition with a strain rate of 0.1 mm/min under the confining pressure (cell pressure) of s3 equal to 190kPa, 300 kPa and 400kPa to define the shear strength parameters (effective angle of shear strength (F0) and cohesion (c0)) for both the unreinforced and reinforced silt clay sand as indicated in Table 8 and Fig. 11.

Mohr-Coulomb describes this criterion is as rather good for the failure state of sands, for such neat soils the cohesion usually is practically zero, C = 0 and the friction angle usually varies from φ =30° to φ =45°, depending upon the angularity and the roundness of the particles. In this case all samples are constantly added with 0.3% of OPC cement hence Cohesiveness(C)of 0.25%OPFB is higher at 125% and that of friction angle is low at 40% as compared to others as shown in the Fig. 11 and Fig. 12 respectively.
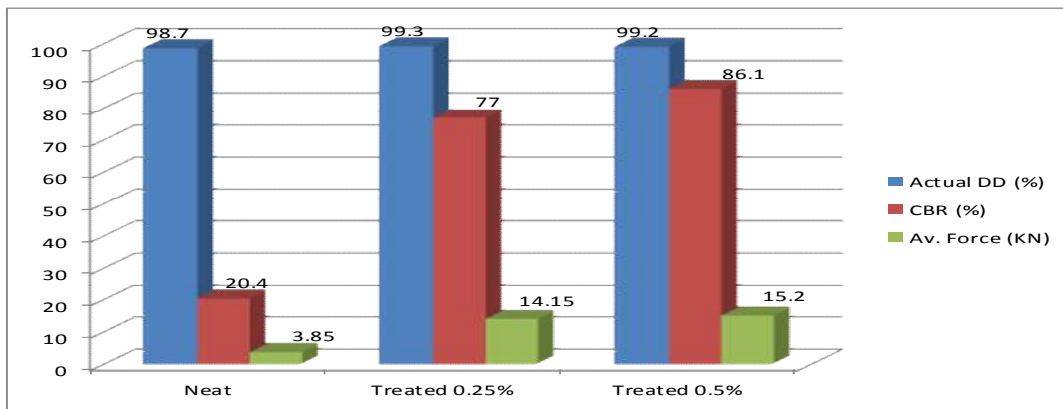

Fig. 11. Comparison of CBR of varies treated soil sample/actual dry density

Table 8.The table below shows a summary of test results conducted

All samples + 0.3% OPC cement	a) Total stress analysis		b) Effective stress analysis	
	C (KN/m²)	φ (°)	C' (KN/m²)	φ' (°)
Neat Soil Sample	12	16	22	16
Soil Sample + 0.25% Fibre	125	40	195	19
Soil Sample + 0.50% Fibre	48	40	75	35

	Neat	Treated 0.25%	Treated 0.5%
Total Cohesion (C)	12	125	48
Effective cohesion (C')	22	205	75
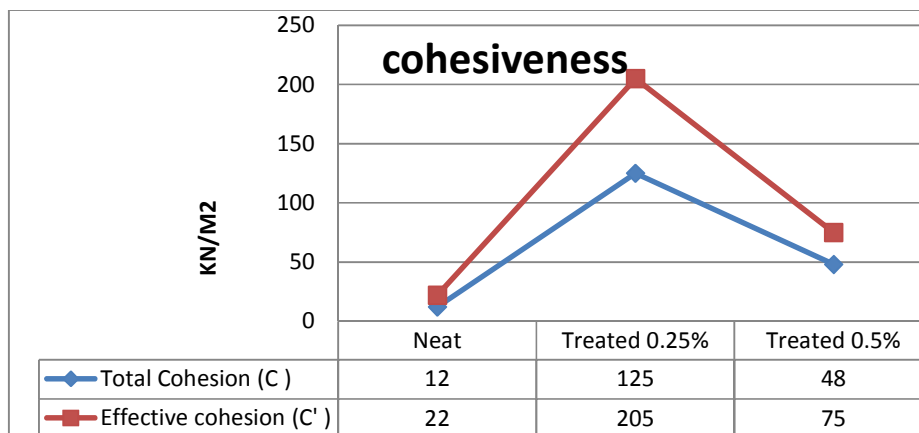
Fig. 12. Comparison of cohesiveness of soil sample

Friction angle

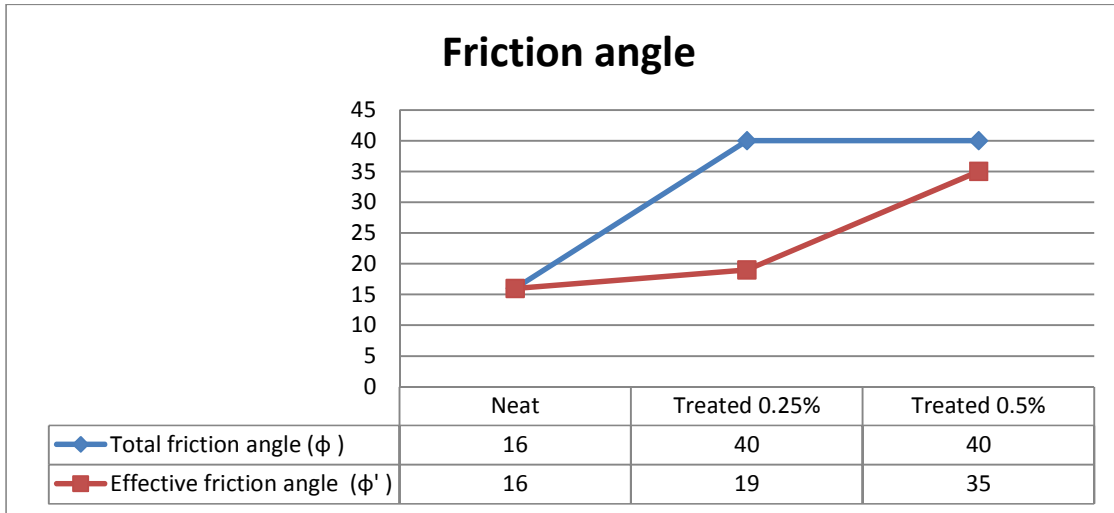|	Neat	Treated 0.25%	Treated 0.5%
Total friction angle (ϕ)	16	40	40
Effective friction angle (ϕ')	16	19	35

Fig. 13. Comparison of friction angle of soil sample

4.4.4 Deviator Stress against Axial strain

Fig. 14 compares the stress–strain behavior of both unreinforced and fibre-reinforced specimens with different fibre content using the triaxial undrained test. The effect of fibre on stress–strain behavior of soil at treated 0.25% fibre-reinforced soil achieved approximately equal peak deviator stress than the treated 0.5% fibre-reinforced as compare to the unreinforced soil. Under undrained test conditions, fibre inclusion did exhibit an effect on deviator stress.

4.4.5 Pore pressure against axial strain

Comparisons were done in the pore pressure–strain behavior of both unreinforced and fibre-reinforced specimens with different fibre content using the triaxial undrained test. The pore water pressure generated during shearing increased with increasing the fibre content up to optimum

and starts to decrease at certain point per cycle. Consequently, added fibre content will not have much effect with increased pore water pressure which, in turn, does not affect the shear strength of the reinforced silty sand. Since positive water pressure is related to the tendency towards volume shrinkage, this observation shows that fibre reinforcement restrains the dilatancy of the reinforced soil Fig. 15.

5. DISCUSSION

The results indicated that the shear strength parameters of the soil-fibre mixture (i.e. **φ'** and C') can be improved significantly up to optimum and at certain point start going down. The shear stress–strain curves obtained from the CU triaxial tests for reinforced sands with 30 mm fibre length together with those for unreinforced silty sand. Compared to the unreinforced specimens, the fibre-reinforced specimens showed higher deviator stress when reaches 0.25% fibre and
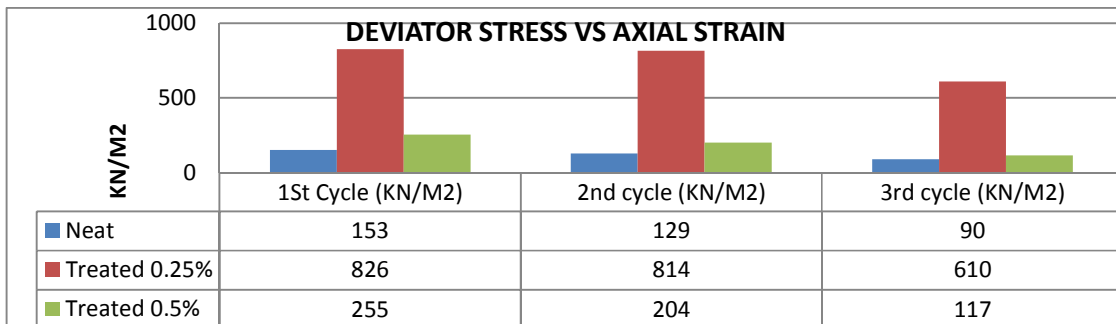
DEVIATOR STRESS VS AXIAL STRAIN

KN/M2

	1St Cycle (KN/M2)	2nd cycle (KN/M2)	3rd cycle (KN/M2)
Neat	153	129	90
Treated 0.25%	826	814	610
Treated 0.5%	255	204	117

Fig. 14. Comparison of deviator stress against axial strain of samples per cycle test

PORE PRESSURE VS AXIAL STRAIN

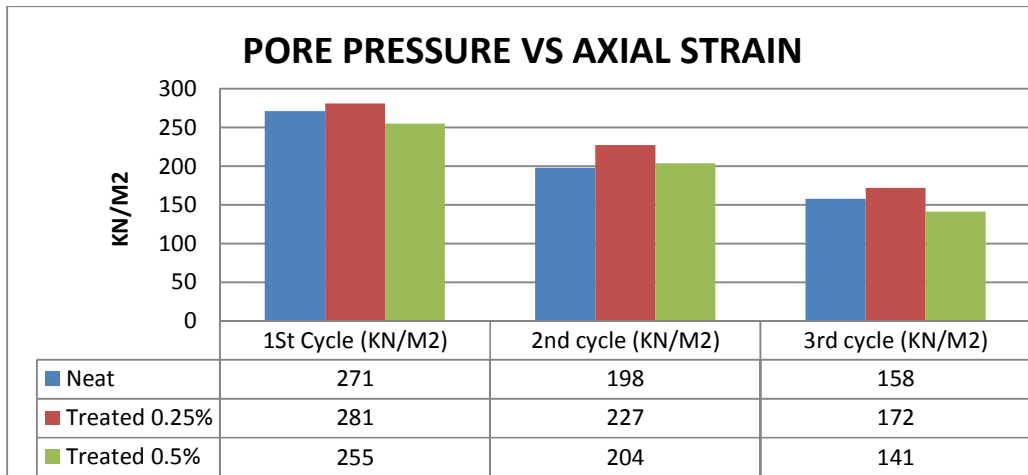|	1St Cycle (KN/M2)	2nd cycle (KN/M2)	3rd cycle (KN/M2)
■ Neat	271	198	158
■ Treated 0.25%	281	227	172
■ Treated 0.5%	255	204	141

Fig. 15. Comparison of deviator stress against axial strain of samples per cycle test

reduces when 0.5% fibre. The strain corresponding to the peak deviator stress is increased by fibre content. The results indicated that, the effect of fibre length 30 mm on the peak stress of soil with fibre content (f) of 0.25%. Patterns of stress–strain curves for all reinforced samples revealed an improvement in the deviator stress for all compositions and fibre content. Deviator stress of fibre-reinforced soil shows slide increase with increasing pore pressure.

The increase of the fibre content cause the increase in pore water pressure due to inclination of specimens to decrease the volume.Changes in the shear strength of fibre-reinforced soil indicate that soil strength parameters (φ' and C') increase as the internal friction surface increases between fibre and soil at certain point. Figs. 11.-12 illustrates the increasing the reinforced soil shear strength parameter (φ' and C') with increasing fibre content. The shear stress increased non-linearly with increasing length of fibre up to 30 mm. This result suggests that, long fibres and very high fibre contents reduce the interlock of soil particles and therefore, fibre-soil particles do not act as a single coherent mass. According to Prabakar and Sridhar, 1902), cohesion increases linearly with fibre content. Consequently, interface friction of fibre and soil increases. Increase in the tensile strength and stiffness of fibres leads to an increase in shear strength of fibre-reinforced soil because of the limiting effect on the dilatancy of silty sand.

5.1 Slope Stability Parameters and Formula

Shear strength depends upon effective stress and not total stress. Coulomb's equation must therefore be modified in terms of effective stress and becomes:

$$Tr = C' + \sigma' \tan \phi'$$ (i)

Where

C' =Unit cohesion, with respect to effective stresses

σ' =Effective normal stress acting on failure plane

ϕ' =Angle of shearing resistance, with respect to effective stresses

Calculating effective shear resistance for slope foundation design

$$\tau_f = c' + \frac{1}{2} (\sigma'_1 + \sigma'_3) \tan \varphi$$

Basic Assumptions

✓ The excavated soil is simplified as plane soil slopes.
✓ Soil is homogeneous.
✓ The horizontal stress of inner points along the depth causes soil slope instability.
✓ During excavation of the soil, the static lateral pressure coefficient remains unchanged.
✓ The impact of pore water and groundwater is not considered.
✓ Assume a slope of 1:1.2 to be kept on site, V:H (40°slope angle)
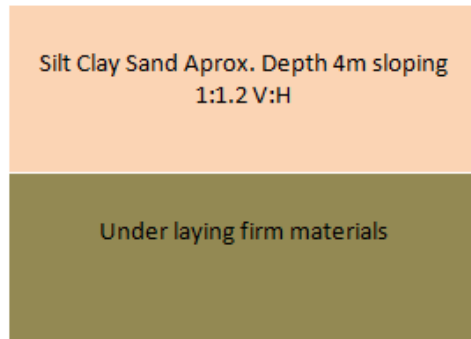✓ Maximum depth of the Silt Clay sand in the area was approximately 3m

Fig. 16. Approximate depth strata of Silt clay sand at In-situ site

5.2 Determination of Shear Resistance and Safety Factor

Formula Derivation using Jaky's formula: The extending direction of the soil slope is taken as the plane stress state, and elastic half-space plane stress analysis is performed for soil in a steady state under gravity stress; the main stress expression of any point is as shown in Fig. 17.

r= Dry density of soil (Kg/m3) x Gravitational force (g)

Where $\sigma 1$ is the maximum principal stress, $\sigma 3$ is the minimum principal stress, r is the soil gravity, 0 is the static lateral pressure coefficient of the soil, z is the distance from the ground surface to any point, σx is the horizontal stress at any point, and σz is the vertical stress at any point.

As shown in Fig. 17, assuming that the cutting or foundation soil is not excavated, when slope angle β = 0∘, then $\sigma x = K0rz$. In vertical excavation of the soil, when slope angle β = 90∘,

$$(\sigma_x = K_0 rz \left(1 - Sin\ \beta \right)) \quad \text{(ii)}$$

Then σ_x = 0. The horizontal stress of the soil excavation slope angle satisfies the following formula

$$(F_s = \frac{T_f}{T} = \frac{(c + \sigma Tan\ \varphi}{T}) \quad \text{(iii)}$$

According to the conventional definition of the factor of safety for some point within a soil mass, the safety factor is the ratio between the shear strength and shear stress at that point. In line with the Mohr–Coulomb criterion for the shear strength of soils, for stresses at some point within a soil mass, differences in the magnitude of shear stress in an arbitrary direction will result in shear strength differences. In other words, the factor of safety at a point in a soil mass, defined
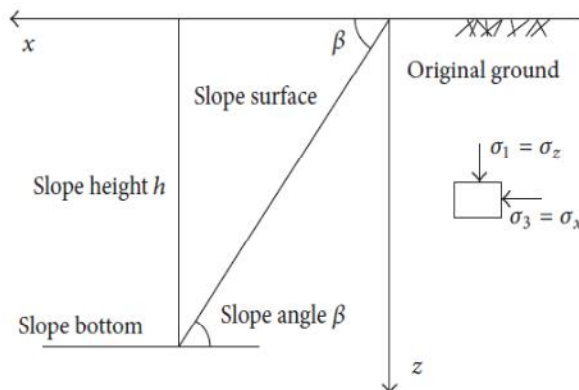
$$\sigma_1 = \sigma_z = rz,$$

$$\sigma_3 = \sigma_x = K_0 rz,$$

$$K_0 = 1 - \sin\varphi_0$$

Fig. 17. Simplified soil excavations

in formula above, will vary with the direction. This leads to complexities and difficulties in the methods of slope stability analysis and to a variety of assumptions in calculation theories. To ensure the uniqueness of safety factors computed at each point within a soil mass, the factor of safety was defined as described below.

$$F_s = \frac{T_f}{T_{max}} = \frac{c + ((\sigma_1 + \sigma_3)\tan\varphi}{T_{max}}) \quad \text{(iv)}$$

Given a point with a determined stress state within some soil mass, its margin of safety is the ratio between the shear strength corresponding to the maximum shear strength at that point and the total maximum shear strength as described in Fig. 18.

Then, the safety margin of a slope is the ratio between the cumulative shear strength and the cumulative maximum shear stress within the slope's height; thus

$$F_s = \frac{\int_0^z T_f \, dz}{\int_0^z T_{max} \, dz} \quad \text{(v)}$$

Since:

$$T_{max} = \frac{1}{2}(\sigma_1 - \sigma_3) = \frac{1}{2}rz(1 - K_0(1 - \sin\beta)),$$

$$T_f = c_{uni} + \frac{1}{2}(\sigma_1 + \sigma_3)\tan\varphi_{uni} \quad \text{(vi)}$$

$$= c_{uni} + \frac{1}{2}rz(1 + K_0(1 - \sin\varphi))\tan\varphi_{uni}.$$

Therefore, (vii) are substituted into (vi) to obtain

$$F_s = \frac{4 c_{uni}(1 + K_0 \sin\beta}{rz(1 - K_0^2(1 - \sin\beta)^2}$$

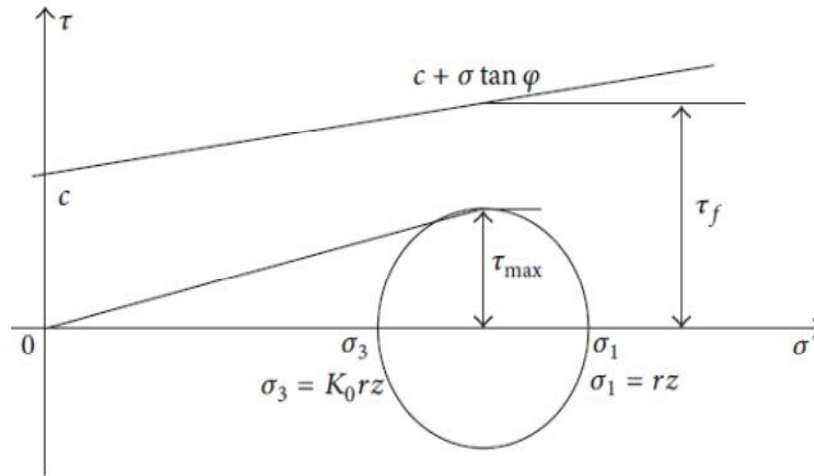$$+ \frac{rz(1 + K_0 - K_0\sin\beta)^2 \tan\varphi_{uni}}{rz(1 - K_0^2(1 - \sin\beta)^2)} \quad \text{(vii)}$$

Fig. 18. Schematic diagram of accumulated safety in soils

Table 9. Summary of data analyzes

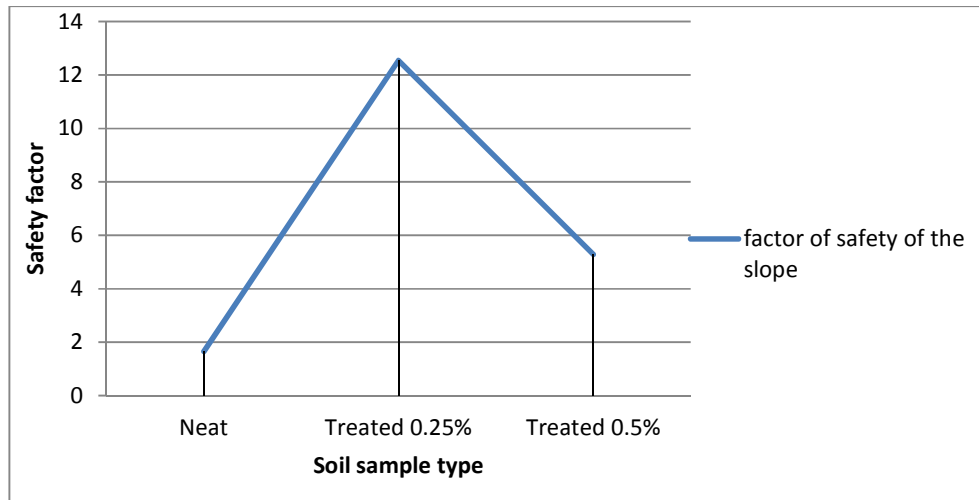| SILT CLAYEY SAND MATERIAL STATE	MDD (Kg/m3)	CBR (%)	Effective stress		Deviator stress Vs Axial strain	Pore pressure Vs Axial strain	Safety factor of slope stability of 4m
			C' (KN/m^2)	φ' (°)	Kpa	Kpa	Unit less
NEAT	1908	19.4	22	16	90	158	1.65
TREATED 0.25%	1912	77.00	195	19	610	172	12.54
TREATED 0.50%	1916	86.1	75	35	117	141	5.28

Fig. 19. Safety factor of slope stability

Therefore by substituting the above equation to obtain

6. CONCLUSION

Based on the findings from this study, it was established that the shear strength parameters of the soil-fibre mixture can be enhanced greatly up to optimum and at certain point starts to decline. The shear stress–strain curves obtained from the CU triaxial tests for reinforced sands with 30 mm fibre length together with those for unreinforced silty sand. On the other hand, the unreinforced specimens for fibre-reinforced showed higher deviator stress when reaches 0.25% fibre and reduces when 0.5% fibre. The findings further established that, increase in fibre content cause an increase in pore water pressure due to inclination of specimens thus decrease the volume. It was concluded that, the shear stress increased non-linearly with increasing length of fibre up to 30 mm. This result then suggests that, long fibres and very high fibre contents reduce the interlock of soil particles and therefore, fibre-soil particles do not act as a single coherent mass.

A series of triaxial tests were conducted to study the influence of Oil palm empty fruit bunch (OPEFB) fibre inclusion on the shear strength behaviour of silt clay sand soils under undrained triaxial tests conditions. The addition of OPEFB fibres was significantly found to increase the peak shear strength of silty sand soil. Increasing fibre content leads to increasing strain at failure and, consequently, to more ductile behaviour. The shear stress increased non-linearly with

increasing length of fibre up to 30 mm. This result suggests that long fibres and very high fibre contents reduce the interlock of soil particles and therefore, fibre-soil particles do not act as a single coherent mass [9]. Volume dilatation decreases by increasing the fibre content under drained conditions, and fibre inclusion increases the positive water pressure under undrained conditions due to increase of the shear strength of the soil. Cohesion increases linearly with fibre content. The increase in the length of fibre increases the value of internal friction angle. There is no specific trend in the variation of friction angle increase with increase of the fibre length.

7. RECOMMENDATION

The above science research paper was to obtain combined formula of slope engineering of obtaining parameters of the shear strength theory and the static lateral pressure coefficient. The slope stability and safety were evaluated with the same method. Through the higher safety factor of slope stability, recommendations were taken for the soil sample of 0.25% fibre. All sample's safety factor were greater than one (1) which means they are okay in certain degree. If incase any of the sample was less than one (1) then soil cannot with stand the slope stability. The effects of different factors on soil slope safety and stability were analyzed as shown in Fig. 19, including the intermediate principal stress parameter, twin shear stress state parameters, and static lateral pressure coefficient. These results indicate that the intermediate principal stress and static lateral

pressure coefficient cannot be ignored in slope stability analysis. This study only examined the effects of shear strength theory parameters and the static lateral pressure coefficient on the slope safety factor. In order to determine the parameters and practical applications of the soil sample, further research and verification are needed. The theoretical formula was derived, calculated, and analyzed from the perspective of the total stress state. The effects on pore water pressure and groundwater should be further examined from the perspective of effective stress.

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COMPETING INTERESTS

Authors have declared that no competing interests exist.

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