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Assessment of Soil Shear Strength Parameters: Insights from Direct Shear and Direct Simple Shear Testing

Denis Kalumba [1], Zainab Babalola [1], Frank Aneke [2*], Faridah Chebet [1], Laxmee Sobhee-Beetu [3]

[1] *Department of Civil Engineering, University of Cape Town, Rondebosch, Cape Town 7701, South Africa.*

[2] *Geotechnics and Materials Development Research Group (GMDRg) Civil Engineering Department, University of KwaZulu-Natal, Durban 4004, South Africa.*

[3] *Department of Civil Engineering and Geomatics, Cape Peninsula University of Technology, Symphony Way, Bellville, South Africa.*

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Abstract

The direct shear test is widely used to determine shear strength parameters (c, φ^0) of soil. However, its validity has been questioned due to several issues, such as uneven stress distribution, the creation of a predetermined failure plane, lateral constraints, difficulties in controlling drainage conditions, and limitations in measuring pore water pressure, which is essential for understanding soil behaviour under different conditions over time. This study addresses these concerns by comparing the shear strength parameters obtained from a direct shear test (DST) and a direct, simple shear test (DSST). To further explore these issues, a fully automated universal shear device was used to perform shear tests on clay, sand, and composite soil (clay + sand), covering both consolidated and shear phases of DST and DSST. Specimens were fabricated at their optimal moisture content, and the composite soil was developed by mixing clay with sand in proportions of 10%, 25%, 50%, and 75% of the mass of sand. This research aims to clarify the relationship between these two testing methodologies through comprehensive testing and to enhance the knowledge of the principal mechanism of the 2 tests. The findings revealed that the DST yielded higher shear strength values than the DSST results. It was also observed that the friction angle of sand specimens generally decreased with the addition of clay for both tests. Additionally, the c, the kaolinite soil in DST and DSST, decreased in the sand as the clay contents increased.

Keywords: Cohesion; Angle of Internal Friction; Clay Content; DST; DSST.

1. Introduction

In the design of geotechnical infrastructures, shear strength remains the most essential input parameter, yet its assessment poses significant challenges [1]. Also, shear strength parameters, namely cohesion (c) and angle of internal friction, (φ^0) are fundamental to geotechnical design, influencing slope stability, bearing capacity, and earth pressure calculations [2]. Laboratory testing remains the cornerstone for evaluating these parameters, with the Direct Shear Test (DST) and Direct Simple Shear Test (DSST) being among the most widely employed methods [3, 4]. While both tests aim to simulate shear behaviour under controlled conditions, their mechanics, boundary constraints, and stress paths differ significantly, leading to variations in measured parameters and interpretive reliability [5, 6].

The Direct Shear Test (DST), standardised under ASTM D3080, involves shearing a soil specimen along a predefined horizontal plane. Its simplicity, cost-effectiveness, and rapid data acquisition have made it a staple in

* Corresponding author: anekef@ukzn.ac.za

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geotechnical laboratories. However, DST suffers from stress non-uniformity and constrained deformation, which may not reflect in-situ soil behaviour accurately [7-10]. Conversely, the Direct Simple Shear Test (DSST), developed by the Swedish Geotechnical Institute in 1936, applies uniform shear stress across the specimen without enforcing a failure plane. This allows for progressive deformation and lateral displacement, better simulating field conditions around pile shafts or embankments. DSST is particularly valuable for evaluating cyclic loading and strain-softening behaviour, though it is more complex and less commonly used due to equipment limitations [11, 12].

Recent comparative studies have highlighted systematic differences between DST and DSST results. Zhang et al. [13] found that DST consistently yields higher shear strength values than DSST under identical soil conditions, attributing this to DST's rigid boundary constraints and forced failure plane. Similarly, Aneke et al. [14] emphasised that DSST captures a more distributed failure mechanism, making it more representative of field behaviour. Zhang et al. [15] extended this comparison to soil-geosynthetic interfaces, concluding that while DST provides conservative estimates, DSST offers nuanced insights into interface behaviour under realistic loading paths. Moreover, moisture content and fines significantly influence the disparity between the two tests. For instance, increasing water content reduces (φ^0) in both tests, but affects cohesion differently DSST shows an increase in cohesion with added water in clayey soils, whereas DST shows a decrease. Researchers have systematically contrasted DST and DSST results to quantify discrepancies in shear strength parameters. Trends indicate that DST often yields higher peak φ and c values than DSST under identical soil and loading conditions. Rigid boundary constraints and the imposed failure plane in DST accelerate mobilisation of shear resistance. DSST's distributed deformation and volumetric changes produce lower, albeit more field-representative, shear parameters. These differences underscore the importance of matching test protocols to specific design scenarios [16- 19],

Gundersen et al. [20] suggested that soil type, gradation, mineralogy, and fabric firmly control shear responses. Clean sands exhibit pronounced dilatancy under simple shear, whereas fine-grained clays develop significant excess pore pressure during undrained loading. The presence of fines or organic content alters interparticle contact mechanics, leading to divergent DST and DSST outcomes. Stress history and anisotropy further modulate test results, but these factors are rarely incorporated into standard protocols. A comprehensive evaluation must account for the full spectrum of soil characteristics. Besides soil type, gradation, mineralogy, and fabric exert, Hassona et al. [21] supported that post-peak softening and residual shear strength govern large-deformation stability in slopes and embankments. DSST's ability to sustain large shear displacements makes it better suited for residual strength evaluation. With its fixed failure plane, DST often underestimates volumetric collapse and residual friction angles. However, residual behaviour has been studied for only a narrow range of soils, leaving a knowledge gap in applying post-peak parameters to diverse geotechnical scenarios.

Despite extensive research, several critical gaps undermine the robust application of DS and DSS test results such gaps are as follows: a) lack of standardised sample preparation protocols ensuring equivalent density and fabric for both tests, b) insufficient studies combining in situ stress path replication with DSS testing to benchmark against DS outcomes, c) limited integration of microstructural observations to explain macroscopic strength differences, d) absence of a unified constitutive model calibrated by parallel DS and DSS data sets, and e) sparse data on anisotropic and layered soils under cyclic and multi-directional loading conditions. Among all these identified gaps, this study attempts to bridge the gap mentioned in "a". Addressing these gaps will be essential to enhance the predictive capability and transferability of laboratory-derived shear strength parameters to field conditions.

In conclusion, the direct shear and direct simple shear tests remain cornerstone methods for assessing soil shear strength, offering unique insights into soil behaviour under shear. However, disparities arising from boundary conditions, sample preparation, and stress path differences limit their direct comparability and field applicability. By standardising specimen preparation, synchronising stress paths, integrating microstructural analyses, and developing a unified constitutive model, the proposed approach aims to bridge the current gap. This comprehensive framework will enhance confidence in laboratory-derived shear strength parameters and improve the fidelity of geotechnical design.

2. Theoretical Approach

The theoretical foundation of this study is based on the principles of classical soil mechanics, particularly the behaviour of soils under shear stress. Soil shear strength is a critical parameter in geotechnical engineering, influencing the stability of slopes, foundations, earth-retaining structures, etc. The shear strength of soil is governed by two primary components: cohesion (c) and angle of internal friction, (φ^0) as described by the Mohr-Coulomb failure criterion. This study employs two laboratory methods, the Direct Shear Test (DST) and the Direct Simple Shear Test (DSS), to evaluate these parameters.

The DST assumes failure along a predefined horizontal plane and is widely used due to its simplicity and applicability to granular soils. However, it may not accurately represent stress conditions in cohesive soils. The DSST**,** on the other hand, allows for uniform shear deformation under constant vertical stress, providing a more realistic simulation of field conditions, especially for saturated clays. The theoretical comparison between these methods is grounded in their ability to replicate in-situ stress paths and failure mechanisms.

By analysing the stress-strain behaviour and failure envelopes obtained from both tests, the study aims to validate each method's reliability and applicability theoretically. The approach also considers the limitations of each test in terms of boundary conditions, drainage control, and sample disturbance. Ultimately, the theoretical framework supports interpreting experimental results and guides recommendations for selecting appropriate testing methods based on soil type and engineering context.

3. Materials and Methodological Program

3.1. Klipheuwel Sand (KS)

The KS, characterised by its reddish-brown hue, has a natural moisture content of 2.63%. As per the Unified Soil Classification System, it is sourced from Cape Town, South Africa, and is classified as well-graded sand with minimal fines and angular particles. Its local availability and substantial abundance influenced the choice of this sand in the Cape Town area. Its cleanliness and easy handling contributed to the test results' consistency.

3.2. Kaolin

Kaolin serves as a representative example of soft soil. Consequently, it was chosen for this study to evaluate the outcomes under conditions analogous to those of Klipheuwel sand. Furthermore, its consistent and easily manageable properties facilitated the reproducibility of the results. The kaolinite clay utilised in this research is sourced and provided locally in Cape Town.

3.3. Composite Material

The sieve analysis of the geomaterial is shown in Figure 1, and the test was performed according to ASTM D1140 [22] protocol. The sieve analysis for Kaolinite clay indicated that the sampled soil comprised 70% clay, 25% silt, and 5% sand fractions, totalling 100% of the fine fraction. The kaolinite soil used in this study is classified as a low plasticity soil (CL) with an average particle diameter (D50) of less than 0.075 mm, following Unified Soil Classification System (USCS) testing procedures. The composite soil was prepared using various proportions of 10%, 25%, 50%, and 75% of the mass of sand. However, a 50% sand and 50% clay ratio was utilised for the sieve analysis to maintain equal percentages of Klipheuwel sand and kaolinite clay in the composite soil material.

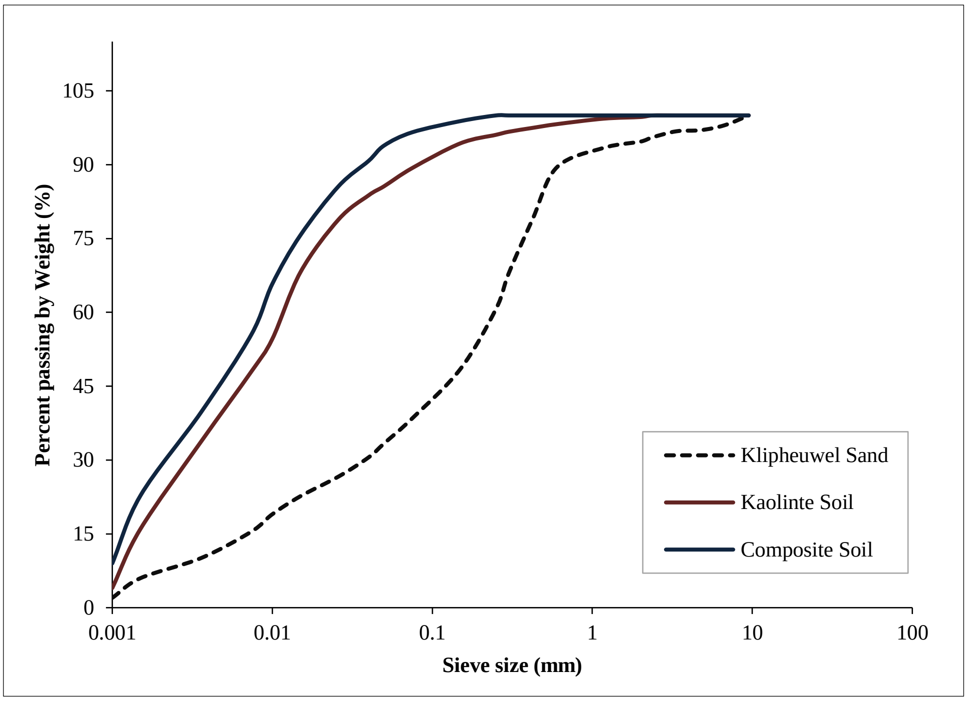


Figure 1. Soil gradation curve

Furthermore, Kaolinite clay's sieve analysis results showed that the sampled soil recorded 70%, 25%, and 5% of clay, silt, and sand fractions, respectively, to a total fine fraction of 100%. The kaolinite soil used herein is categorised as soil with low plasticity (CL), with an average particle diameter at D_{50} of less than 0.075 mm based on the Unified Soil Classification System (USCS). The composite soil was developed using 10%, 25%, 50%, and 75% sand. However, the 50% sand and 50% clay ratio was tested for the sieve analysis to maintain equal percentages of fKlipheuwel sand and kaolinite clay in the composite soil material.

The preliminary findings on the examined geomaterials are summarised in Table 1. However, the test result revealed that Kaolinite clay has a liquid limit of 36%, a plastic limit of 21.4%, and a plasticity index of 14.6%. As per the chart provided by Mitchell & Soga [23], soils with a liquid limit between 30% and 40%, a plastic limit ranging from 20% to 25%, and a plasticity index of 10% to 15% are classified as low plasticity soils.

Table 1. Mechanical properties of the geomaterial used herein

Soil Properties	Kaolin	Kliphuewel	Composite
Gs (Specific Gravity)	2.6	2.65	2.7
OMC (%) Optimum Moisture Content	23	10.7	15.21
MDD (kg/m^3) Maximum Dry Density	18.65	15.23	17.56
Particle Size (mm)	0.065	0.045	3.35
Cu (Uniformity Coefficient)	1.75	6	3
Cc (Coefficient of Curvature)	>>	2.5	2.3
LL (%) Liquid Limit	36	11	24
PL (%) Plastic Limit	21.4	>>	11.12
PI (%) Plasticity Index	14.6	>>	12.9

3.4. Sample Preparation Procedures

Figure 2 is a flowchart, and it illustrates the methodological workflow for assessing soil shear strength parameters using two laboratory techniques: Direct Shear Test (DST) and Direct Simple Shear Test (DSS). It begins with Sample Collection, where soil specimens are obtained from field sites, ensuring they represent the in-situ conditions. The next step is Sample Preparation, which involves trimming, conditioning, and placing the samples into appropriate testing moulds while maintaining controlled moisture and density. The Testing Phase is divided into two branches: DST and DSS, each designed to simulate different shear conditions. In the Direct Shear Test (DST), a horizontal shear force is applied along a predefined plane under constant normal stress to measure shear resistance. The Direct Simple Shear Test (DSS) applies shear deformation while maintaining vertical stress, allowing for more uniform stress distribution and better simulation of field conditions. Both tests generate data on shear stress and strain, critical for evaluating soil behaviour under load. The Data Analysis involves calculating key shear strength parameters such as cohesion (c) and internal friction angle, (φ^0) from the test results. These parameters are then compared between DST and DSS to understand the influence of testing methods on soil strength characterisation. Finally, the Interpretation phase synthesises the findings to inform geotechnical design decisions, highlighting the suitability of each test for different soil types and engineering applications. Its worthy to note that each test condition was performed on a minimum of three identical specimens, and the results reported in the manuscript represent the arithmetic mean of these replicates.

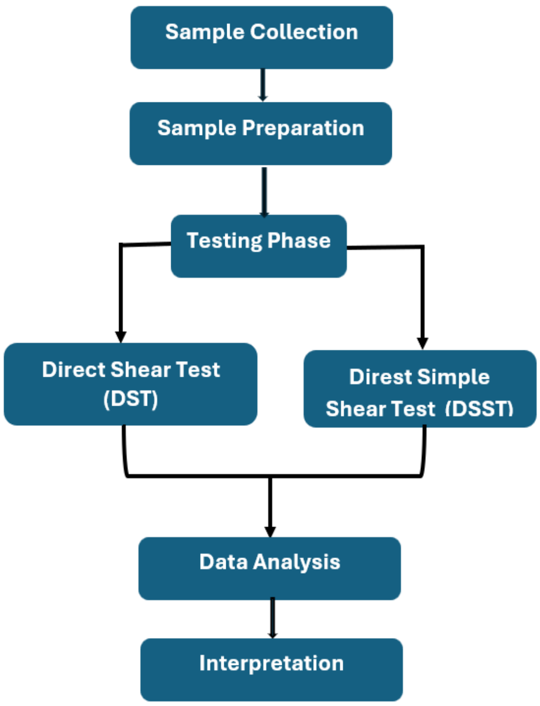


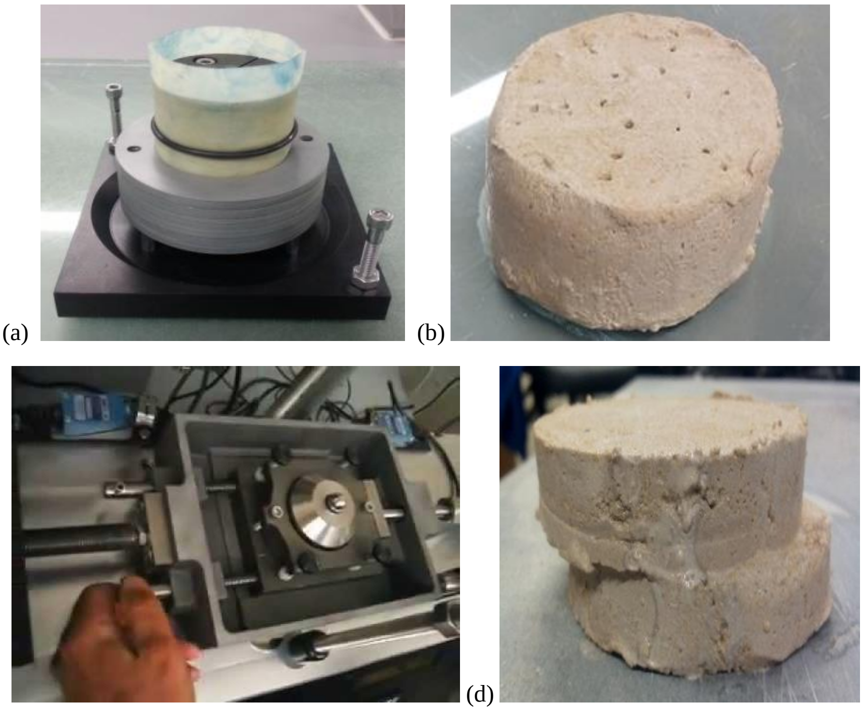
Figure 2. Flowchart Methodological Process

All soil types i.e., pure kaolinite, Klipheuwel sand, and a 50:50 (w/w) composite blend were first oven-dried at 105 °C for 24 h and lightly macerated with a porcelain mortar. Standard Proctor trials (ASTM D698) were performed to determine the quantities of the molding misture contents. The results of the compaction tests yielded maximum dry densities and optimum moisture contents of 1.48 g/cm³ at 24.5 % moisture for kaolinite, 1.68 g/cm³ at 9.2 % moisture for Klipheuwel sand, and 1.58 g/cm³ at 17.8 % moisture for the composite respectively. To prepare specimens, each dried soil was weighed to achieve 95 % of its maximum dry density in a 100 mm×116 mm mold (± 0.1 g) and wetted incrementally with deionized water to reach 95 % of its OMC and the following targeted moisture contents were achieved 23.3 % for kaolinite, 8.7 % for sand, 16.9 % for composite). The moistened mixtures were then placed in sealed plastic bags for six hours, with gentle kneading every hour to ensure uniform moisture distribution before compaction. After compaction, the fabricated specimens produced and specimens passed the targeted moisture contents and dry densities. The failed specimens were discarded and were replaced with another fabricated that satisfied the targed moisture contents and dry densities values.

4. Methodological Programs

4.1. DST and DSST

The DST and DSST were performed in this study following ASTM D3080 [24] and ASTM D6528 [25], respectively. The test started by first fabricating the specimens. The obtained soil samples were mixed at varying moisture contents until a homogenous blend was achieved. The prepared soil samples were trimmed to fit into the shear box cell. Subsequently, the shear box setup apparatus was set up by placing the soil sample in the shear box and ensuring it was evenly distributed. The shear box is typically divided into two halves, with a single shear plane. After the setup, a standard load was placed on the soil sample as the shear box apparatus was automated, and the loads were gradually applied to allow for consolidation. The applied shear load to the sample was allowed to shear at a controlled strain rate of 1mm/min, which enables it to be slow enough to ensure drained conditions. The load is applied until the specimen fails or reaches the desired displacement at a confined pressure of 50 kPa, 100 kPa and 200 kPa as captured by the software controlling the testing programming. Upon the completion of the test, the shear stress and displacement during the test were captured and stored in the data logger. Generally, the specimens were compacted in three different layers, with all the samples tested at almost the same density and the tests undertaken. The procedures for conducting DST and DSST tests exhibit notable similarities. The direct shear test employs a circular shear box, while the direct, simple shear test utilises a circular ring. The circular shear box and the circular ring have a diameter of 64 mm and a height of 31 mm. Both tests used a sample with an identical predetermined mass of 400 g to ensure consistency and minimise potential discrepancies in the results. Furthermore, the soil samples in both tests were subjected to a uniform displacement rate of 1 mm/min during shearing. Figures 3-a to 3-c illustrate a specimen prepared for testing alongside a specimen post-test (DSST). Figures 3-a to 3-d depict the setup and specimen samples from the direct shear test after completion.



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

Figure 3. (a) Specimen ready for testing and; (b) Specimen after test (sand-clay composite); (c) Direct shear test setup and; (d) Sample after test (composite)

4.2. Universal Shear Device

The universal shear apparatus employed in this investigation was used to conduct shear tests with complete automation, encompassing both consolidation and shear phases. It utilises the ShearTrac-II configuration for DST and the ShearTrac-II-DSS for DSST, as illustrated in Figure 4. In contrast, the DST and DSST tests were performed on Klipheuwel and Kaolin soils. Samples consisting of sand and clay were evaluated at varying water contents. Additionally, a composite sample of sand and clay was tested with different proportions of clay. The moisture content (MC) levels examined were 5%, 10%, 15%, 20%, and 25%, while the clay content in the composite was varied at 10%, 25%, 50%, and 75%. Klipheuwel sand was dried at 106°C for 24 h to mitigate any influences from moisture variations. Conversely, Kaolin was not dried due to its inherently low moisture content.

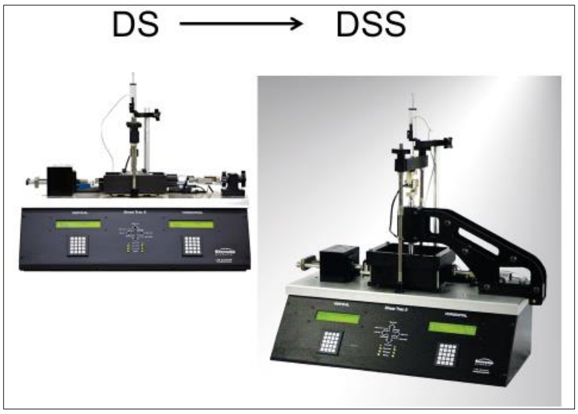


Figure 3. (a) ShearTrac-II (DS) and (b) ShearTrac-II-DSS

5. Results Analysis

5.1. Variation of Sand with the Stiffness Curve

Figure 5 depicts the relationship between shear stress and strain derived from direct and simple shear tests. The results revealed that the application of confining stress significantly impacts the results of both testing methods, with shear stress consistently increasing as confining stress rises. This behaviour can be attributed to enhanced interparticle friction and the densification of soil particles. Consequently, higher confining stress alters the stress path, leading to an increase in shear stress. Additionally, the maximum $shear\ stress, \tau$ recorded in the direct shear tests reached 193 kPa at a shear strain of 1.73%, whereas the peak shear stress from the direct, simple shear test was 119 kPa at a shear strain of 11.02%. This discrepancy is expected, as the failure plane in the direct shear box is located at the centre of the specimen rather than at its weakest point.

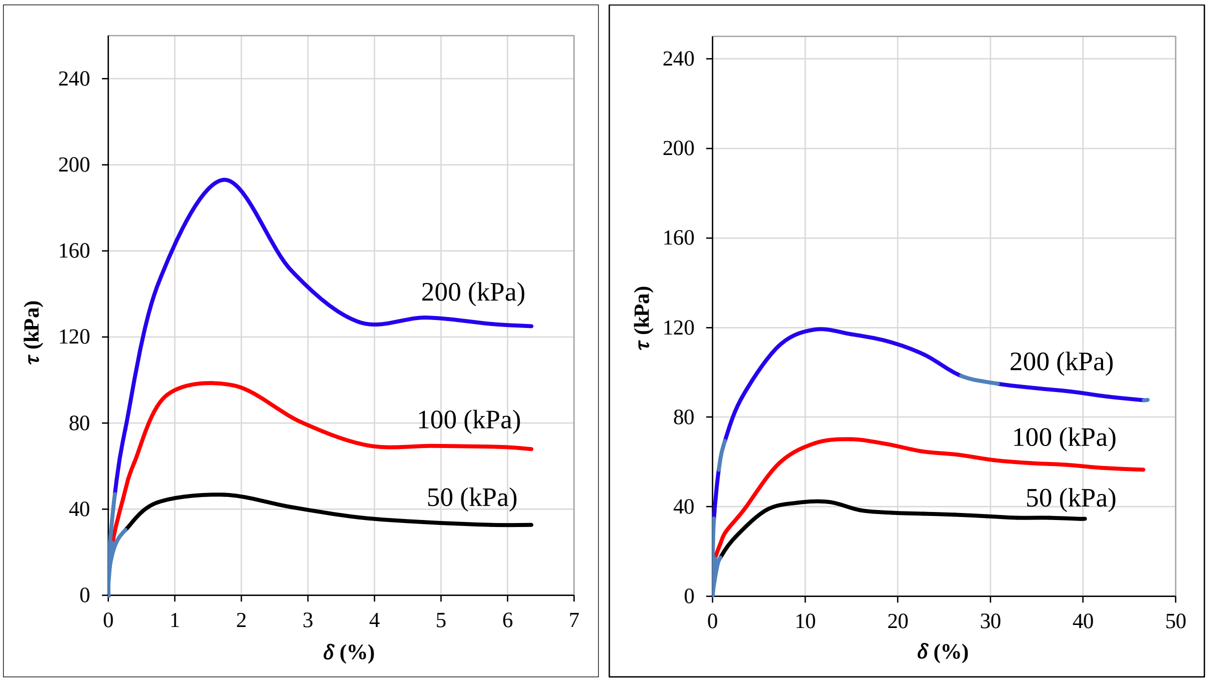


Figure 4. Variation of kaolinite clay with stiffness curve

In contrast, the DSST produces a more uniform stress distribution within the sample, yielding more accurate results. The peak shear stresses observed for all soil samples in the DST were consistently higher than those obtained from the DSST. Additionally, the sandy soil exhibited a brittle response in both the direct shear test and the direct simple shear test results. A similar observation was made by Ikechukwu et al. [26] and Hidayat et al. [27], indicating that the variation in (τ_p) it governs the response of practical stress principles and the path followed. Thus, the loading stress path plays a crucial role in determining the shear behaviour of the soil.

5.2. Variation of Kaolinite Clay with Stiffness Curve

The stress-strain curves for kaolinite clay obtained from both Direct Shear Tests (DST) and Direct Simple Shear Tests (DSST) are demonstrated in Figure 6. The tests were conducted at varying confining pressures of 50 kPa, 100 kPa, and 200 kPa. Like the findings in section 4.1, the DST protocol recorded a peak shear stress of 109 kPa at a confining pressure of 200 kPa. In contrast, the DSST protocol recorded a lower peak shear stress of 69.46 kPa at the same confining pressure. This trend is primarily linked to stress distribution. It is important to note that in the direct shear test, the shearing force is applied along a predefined horizontal plane, which might not necessarily be the weakest plane of the soil sample, potentially resulting in higher shear stress as the soil resists deformation along this plane [28].

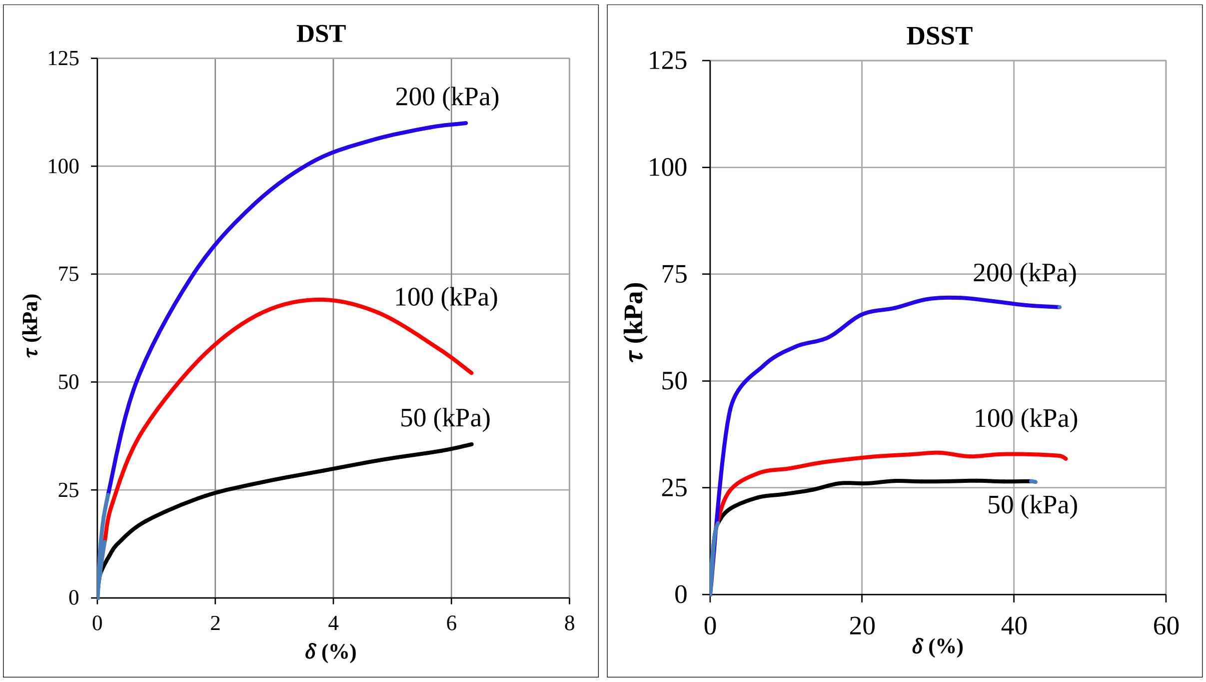
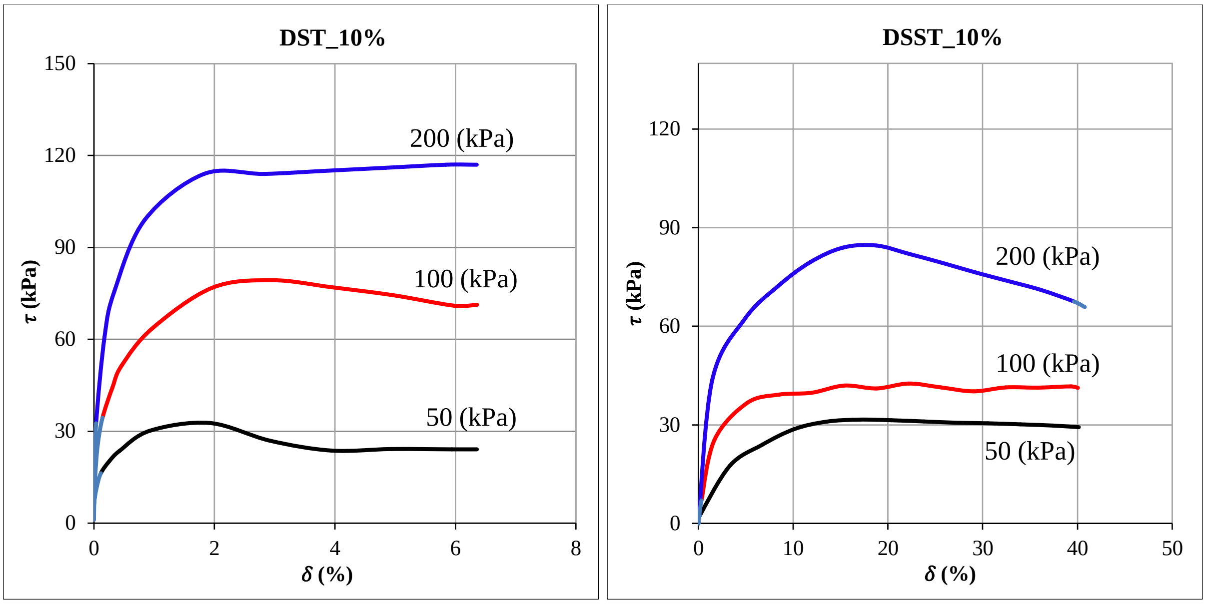


Figure 5. Kaolinite clay stress-strain curves for DST and DSST test results

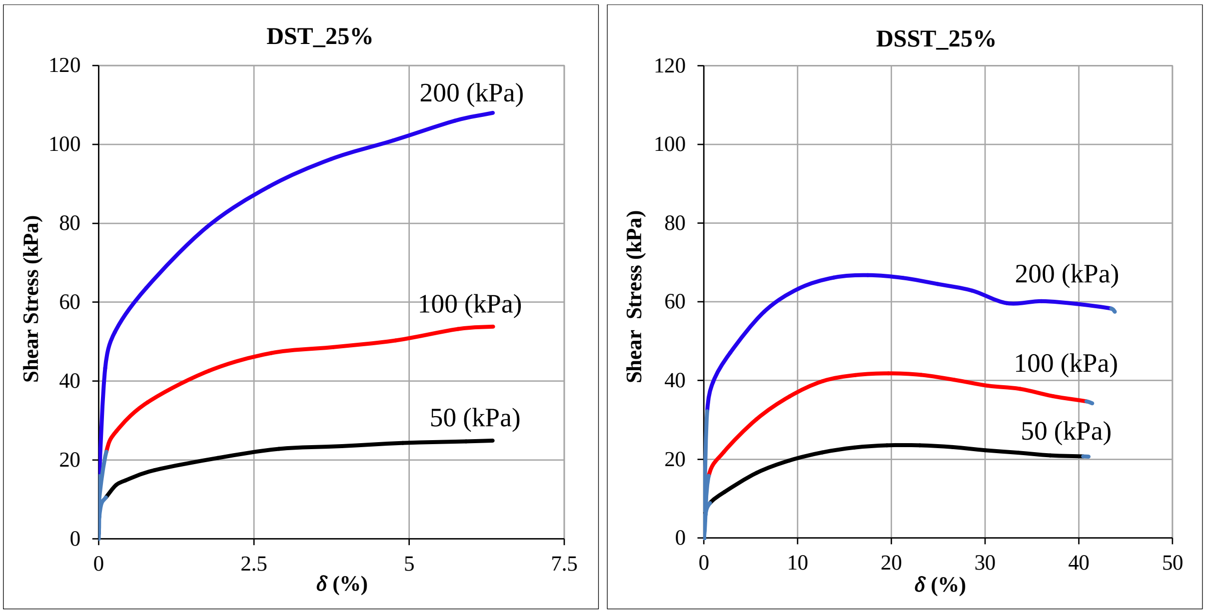
Additionally, the shearing mechanism plays a significant role. Direct shear testing induces shearing along a single plane, leading to stress concentrations and possible non-uniform deformation. In contrast, the DSST allows the entire specimen to deform without forming a single shearing surface, resulting in a more uniform stress distribution and lower shear stress value. The kaolinite clay exhibited ductile behaviour compared to the tested sand, which was anticipated. However, the shear strain for DST curves ranges from 2% to 6%, while the DSST shear strain ranges from 10% to 40%. These observations are consistent with the stress-strain relationships noted in the study. As previously reported, DST and DSST [29, 30]. Stated that lower shear strength is often recorded in DST at 55 mm horizontal displacement or higher than DSST at the same horizontal displacement

5.3. Variation of Composite Soil with Stress-Strain Curve

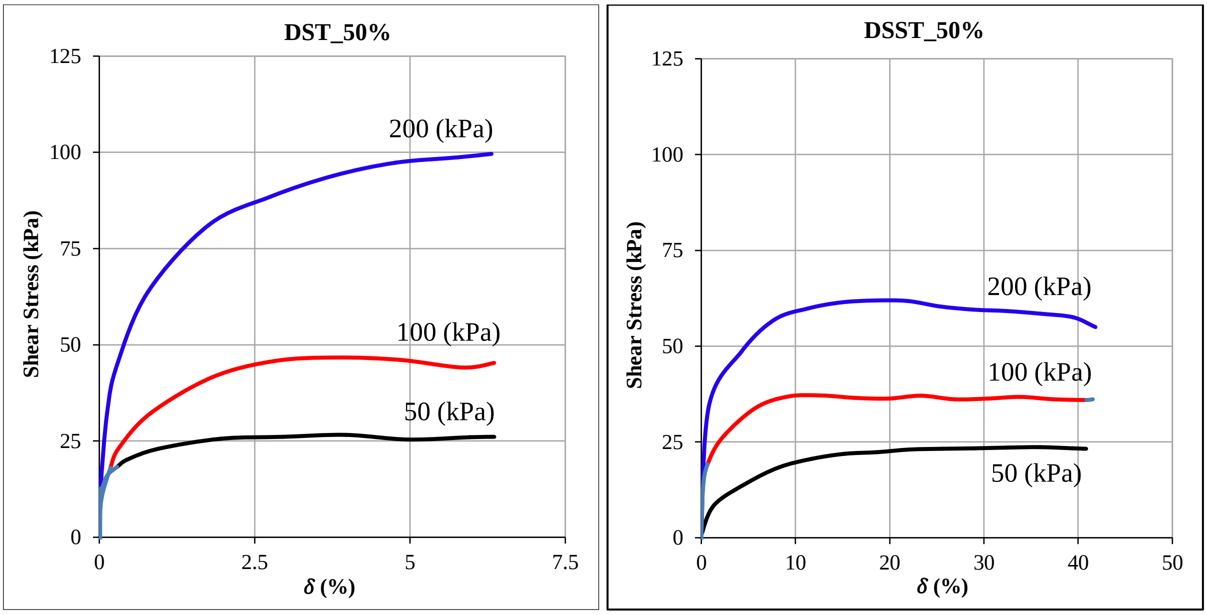
The relationship between shear stresses and composite soil is illustrated in Figures 7-a to 7-d. It was observed that the stress-strain curves exhibited a decline with increasing clay content. Furthermore, adding kaolinite clay resulted in a ductile response within the stress-strain curves. Notably, the data indicated that an increase in the proportion of kaolin clay mixed with Klipheuwel sand led to a reduction in peak shear strength across both testing scenarios. Consistent with expectations, the peak shear stress value recorded in the DST was greater than that of the DSST. The progressive increase in kaolin clay content in Klipheuwel sand correspondingly diminished the maximum shear stress and overall strength. The lowest maximum shear stress was recorded at a clay concentration of 75% for both tests. These findings align with previous research conducted by Wang et al. [31] and Zhao et al. [32], which posited that varying clay contents in sand mixtures induce a significantly overconsolidated behaviour characterised by a nonlinear Mohr-Coulomb failure envelope. Additionally, the silt within the sand matrix markedly affected the anisotropic behaviour of transition silt-clay soils' stress-strain and strength properties.


DST_10%
150
120
90
60
30
0
τ (kPa)
0
2
4
6
8
δ (%)
200 (kPa)
100 (kPa)
50 (kPa)
DSST_10%
120
90
60
30
0
τ (kPa)
0
10
20
30
40
50
δ (%)
200 (kPa)
100 (kPa)
50 (kPa)

(a)


DST_25%
120
100
80
60
40
20
0
Shear Stress (kPa)
0
2.5
5
7.5
δ (%)
200 (kPa)
100 (kPa)
50 (kPa)
DSST_25%
120
100
80
60
40
20
0
Shear Stress (kPa)
0
10
20
30
40
50
δ (%)
200 (kPa)
100 (kPa)
50 (kPa)

(b)


DST_50%
125
100
75
50
25
0
Shear Stress (kPa)
0
2.5
5
7.5
δ (%)
200 (kPa)
100 (kPa)
50 (kPa)
DSST_50%
125
100
75
50
25
0
Shear Stress (kPa)
0
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20
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δ (%)
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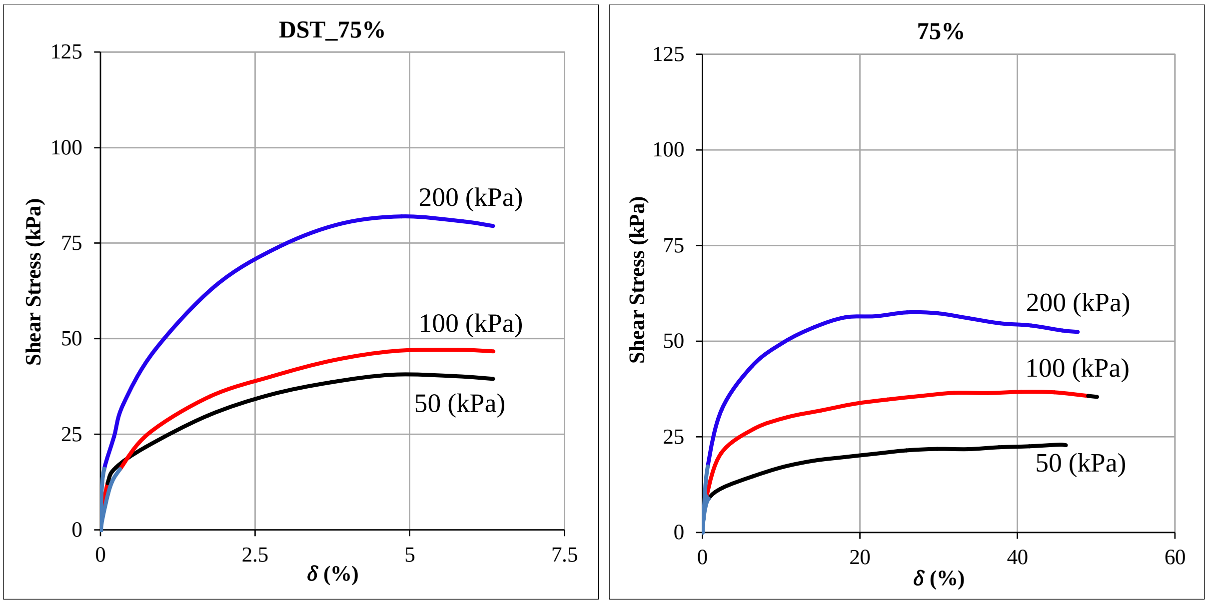
(c)



(d)

Figure 7 (a-d). 10%, 25%, 50% and 75% kaolinite clay stress-strain curves for DST and DSST test results

It is common knowledge that shear stress is the force per unit area exerted by a material when subjected to a shear force. However, understanding the shear stress properties of the composite soil investigated herein is essential for various engineering applications, such as foundation design, slope stability, and earth-retaining structures. The composite soil investigated in this study was developed by blending Klipheuwel sand and kaolinite clay at varying proportions.

The combination of these 2 soils caused the composite soil to have a distinct characteristic: sand is coarse, with larger particles and higher permeability, while clay is fine-grained, with smaller particles and lower permeability. Therefore, the developed composite soil exhibits unique shear stress behaviours influenced by the proportion of each component in the mixture. Thus, peak shear stress values from all the Kaolin clay concentrations added to Klipheuwel sand obtained from DST and SST are given in Table 2.

Table 2. Peak τ for Klipheuwel sand with varying percentages of Kaolin clay

Clay content	50 kN/M^2		100 kn/M^2		200 kN//m^2	
	DST	DSST	DST	DSST	DST	DSST
10%	32.7	31.58	79.3	42.56	117	84.57
25%	24.9	23.53	53.8	41.82	108	66.78
50%	26.1	23.65	46.7	37.13	99.6	61.91
75%	40.6	22.97	47.1	36.76	82	57.54

5.4. Variation of Sand with Shear Strength Parameters

Figure 8 illustrates the Coulomb failure envelopes for Klipheuwel sand, which were obtained from both DST and DSST. The analysis focused on the correlation between maximum shear stress and the applied normal pressure, as represented by the Coulomb failure envelope. The failure curve was constructed using data from all specimens subjected to testing. For each test, the maximum shear stress values were plotted against normal stress for every given confined pressure of 50 kPa, 100 kPa, and 200 kPa. A line of best fit was added to the graph based on the line equation to ascertain the shear strength parameters, namely the internal friction angle and cohesion. The internal friction angle was derived from the arctangent of the slope of the best-fit line, while the Y-intercept indicated the apparent cohesion value.

It is important to note that soil density significantly affects shear strength. Shear strength parameters are primarily influenced by soil particles' internal friction and cohesion, which depend on density. Therefore, the densities of the sand specimens tested under DST and DSST were kept consistent. The internal friction angle for the fabricated sand specimen in the direct shear test was 44.20º, with a cohesion value of 0 kPa. The internal friction angle obtained from the DSST for the same specimen was 27.02º, with a cohesion value of 17.64 kPa. The densities of the sand specimens tested were slightly different, with the direct shear test specimens having a density of 1.66 Mg/m³, while those tested using the direct, simple shear test had a density of 1.65 Mg/m³. These findings are consistent with previous research conducted by Baig et al. [33], Natarajan and Siva Sankara Reddy [34], and Narayan et al. [35], which suggests that although density does influence shear strength parameters, the degree of saturation has a more significant impact. Furthermore, their studies highlight that soil shear strength is not a constant property but can vary considerably under different conditions, depending on factors such as density and moisture content.

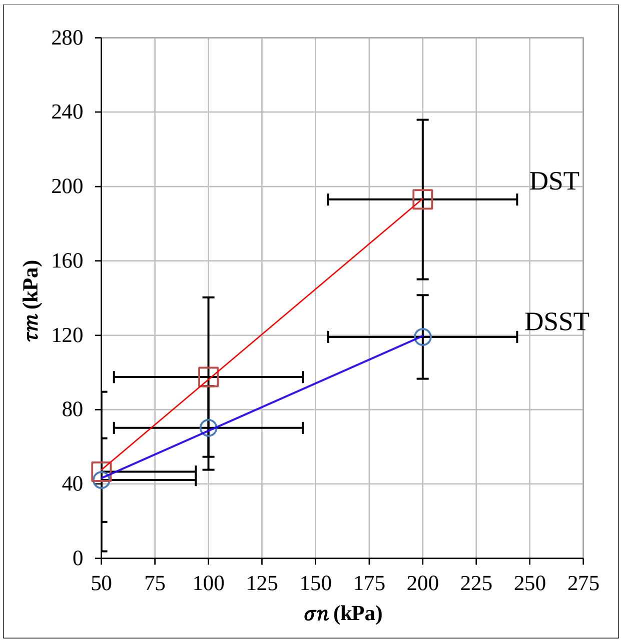


Figure 6. Failure envelopes for Klipheuwel sand

5.5. Variation of Kaolinite Clay with Shear Strength Parameters

The relationship between the shear strength parameters of the kaolinite clay, precisely the angle of internal friction and shear strength, is depicted in Figure 9. This graph illustrates the Kaolin clay failure envelopes derived from direct shear and direct, simple shear tests. The kaolinite clay, classified as a cohesive soil, exhibited an internal friction angle of 25.81º during the direct shear test. Consequently, the cohesion value decreased to 16.50 when subjected to direct, simple shear test (DSST) procedures. This represents a percentage reduction of 36.1% when comparing the results from the DST and DSST. Additionally, cohesion values of 35.6 kPa and 26.7 kPa were recorded for the DST and DSST formats, respectively, indicating a reduction of 25% in cohesion values between the two testing methods. The density of the specimens used in the direct shear test was measured at 1.62 Mg/m^3, while the density of the specimens tested under the DSST was slightly higher at 1.63 Mg/m3 DSST.

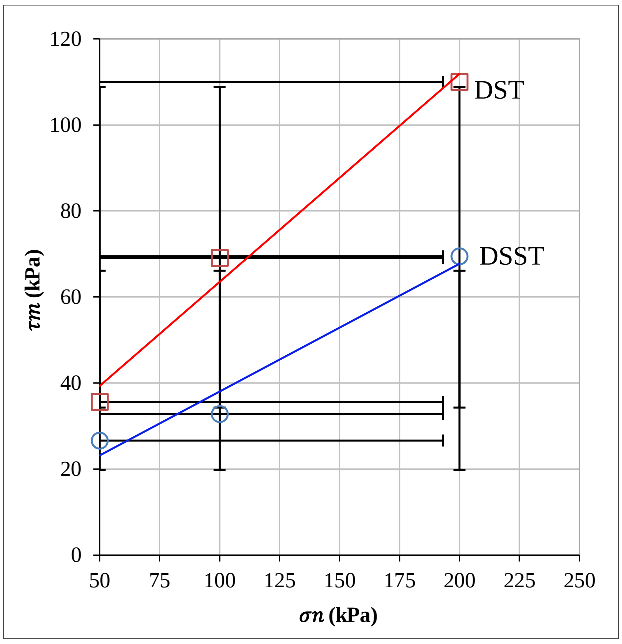


Figure 7. Strength envelopes for Kaolin clay Strength envelopes for Kaolin clay

The direct shear test (DST) imposes horizontal displacement across a preset shear plane, allowing measurement of peak frictional resistance and cohesive strength. By plotting shear stress against normal stress at failure, the friction angle (φ^0) and cohesion (c) are derived from the envelope slope and intercept, respectively. For the kaolinite samples, the 25.81° friction angle highlights moderate interparticle resistance characteristic of fine-grained clays.

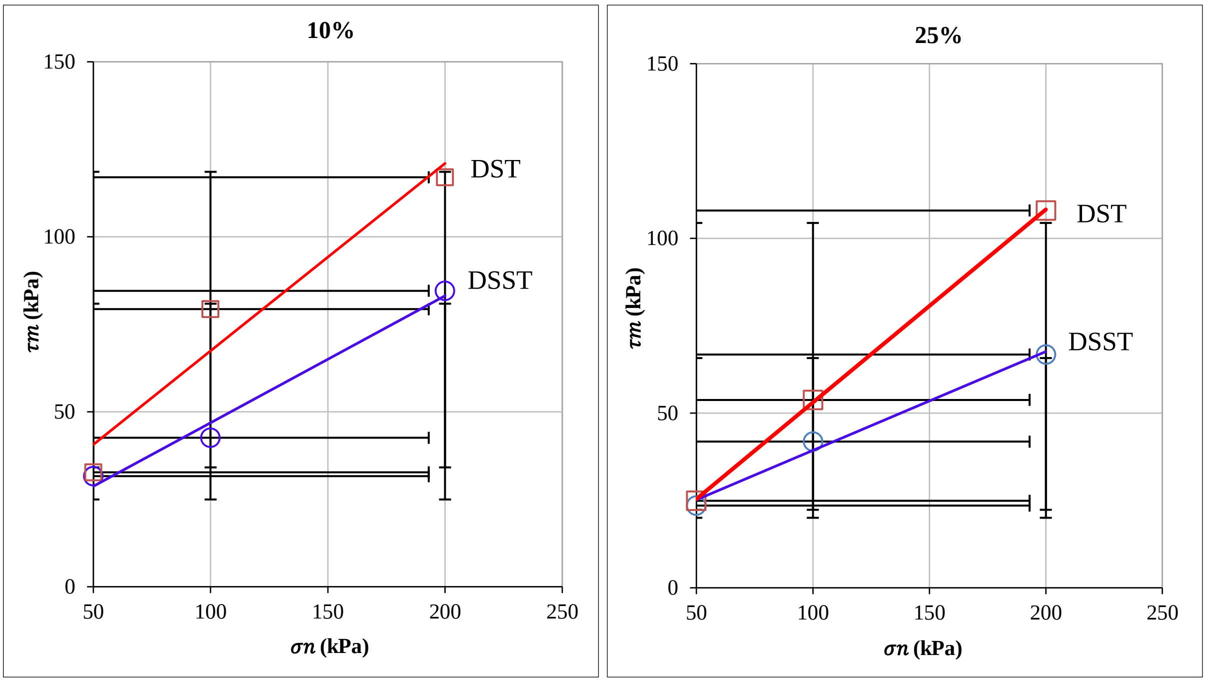
Under the direct simple shear test (DSST), specimens experience uniform shear deformation while maintaining constant vertical stress. This method often better simulates in-situ shear conditions. The DSST results showed cohesion reduced to 26.7 kPa from 35.6 kPa in the DST, a decrease of 25%. The disparity underscores how shear mode and strain distribution influence bonding and frictional engagement between clay particles. When comparing the two methods directly, cohesion under DSST was 36.1% lower than under DST. This steeper drop reflects that continuous shear in DSST disrupts interparticle contacts more effectively than the stepwise slicing in DST. Engineers must account for this difference when interpreting laboratory data for field applications, especially where progressive strain is expected. Density variations between tests were marginal: 1.62 Mg/m³ in DST versus 1.63 Mg/m³ in DSST. The slight increase under DSST may arise from minor volume changes during consolidation or from equipment calibration tolerances. Although small, such density shifts can impact measured strength parameters, so consistent sample preparation and compaction protocols are essential for reliable comparisons

5.6. Variation of Composite Soil with Shear Strength Parameters

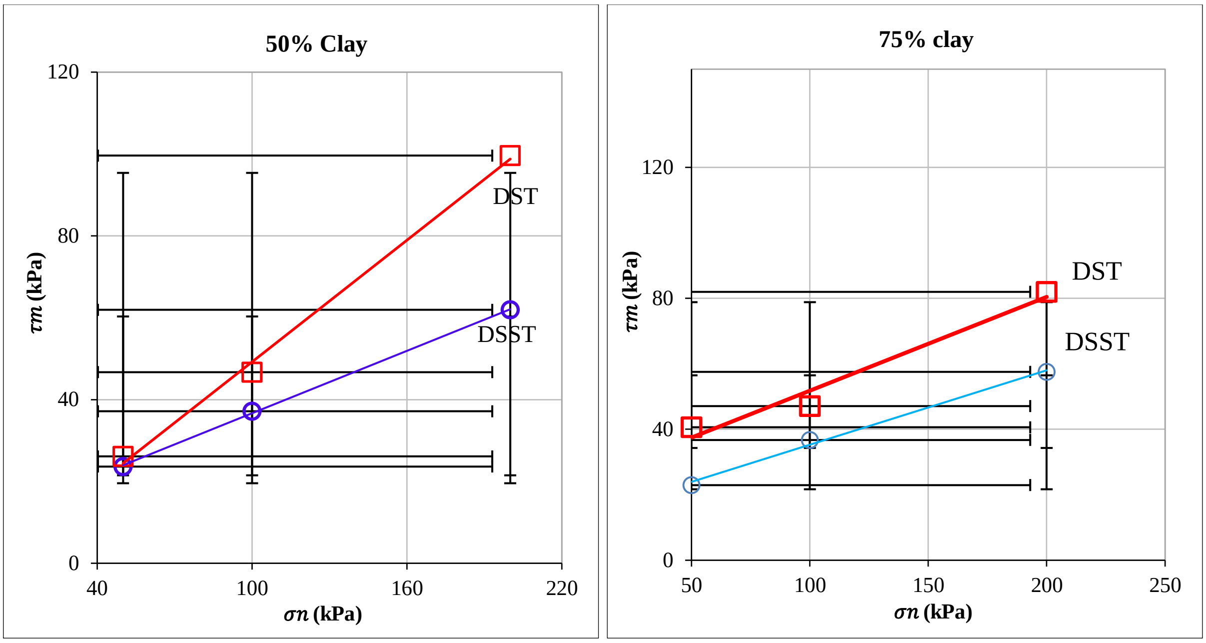
The variation in shear strength parameters of composite soil is illustrated in Figures 10-a and 10-b, which depict the failure envelopes. The addition of clay to the sand mixture profoundly affects shear strength parameters, as evidenced by Skuodis & Norkus [36] and Onyelowe et al. [37] findings. This influence is attributed to the high specific surface area and electronegativity of clay particles, which play a crucial role in the physicochemical interactions governing the behaviour of clay minerals. An increase in the clay content within the soil leads to heightened plasticity, swelling and shrinkage, compressibility, and cohesiveness while concurrently resulting in a marked reduction in the internal friction angle. Specifically, the internal friction angle decreased from 44.20º to 15.80º with the addition of 75% kaolinite clay in the direct shear test and from 27.02º to 12.80º in the direct, simple shear test under the same conditions. Both tests indicate a decline in cohesion and internal friction angle as the kaolinite content increases, with the percentage differences in these parameters summarised in Table 3.

Table 3. c and φ^0 values obtained from both DST and DSST

Klipheuwel sand with Kaolin clay	DST		DSST		% Difference	% Average
% Clay	c (kPa)	ϕ (º)	c (kPa)	ϕ (º)	ϕ	ϕ
10	13.9	28.2	10.8	19.9	29.2	34.1
25	16.2	28.0	11.1	15.8	45.4	
50	18.5	26.4	11.3	14.3	45.9	
75	23.2	15.2	12.6	12.8	15.7	



(a)



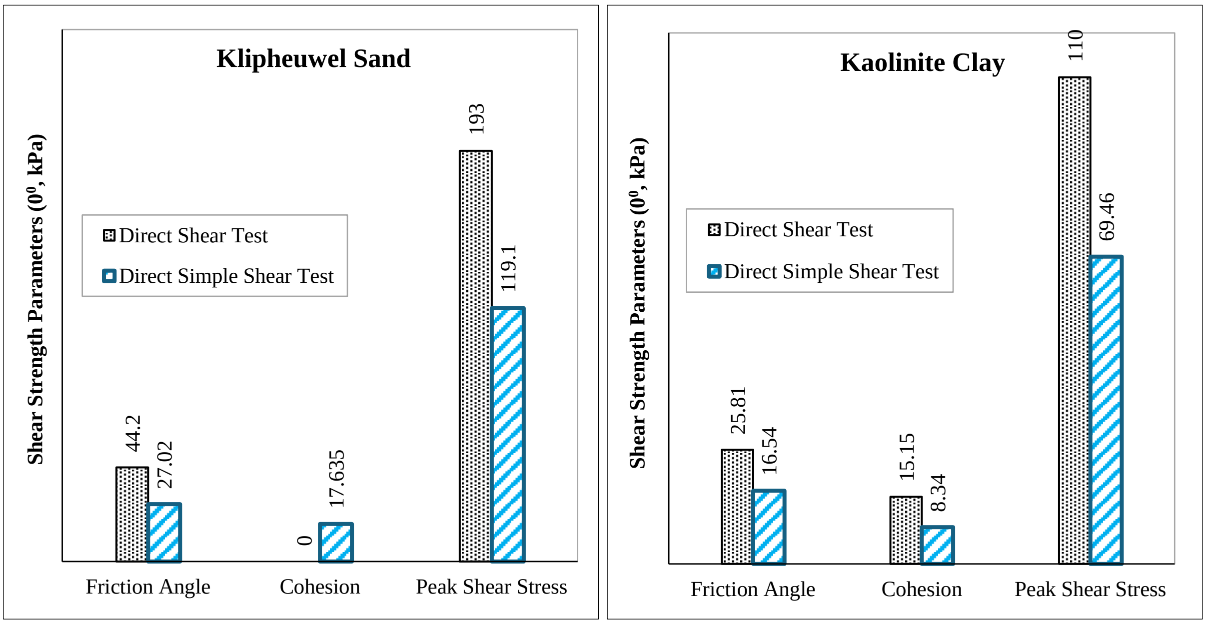
(b)

Figure 10 (a) Strength envelopes for composite soil 10% and 25% clay blends σ_n; (b) Strength envelopes for composite soil with 50% and 75% clay blends

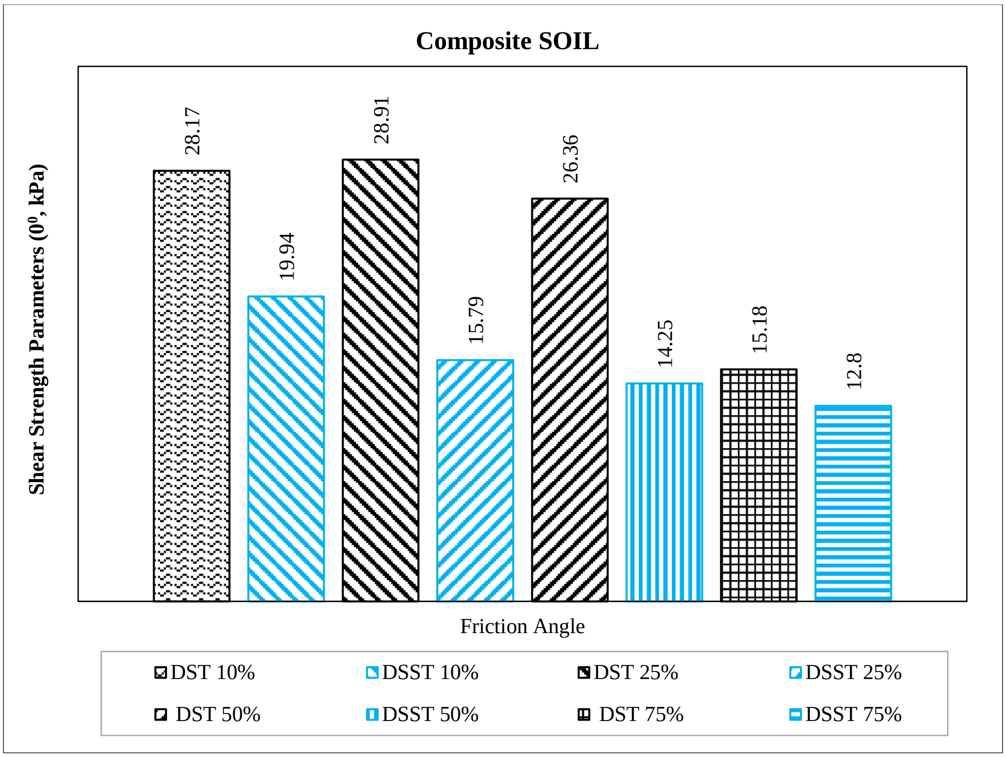
The findings of this research are consistent with the studies conducted by Vithana et al. [38], Kong et al. [39], and Aneke & Onyelowe [40], which established that cohesion rises with an increase in clay content within sand mixtures. Conversely, the internal friction angle diminishes as clay content increases. The progressive incorporation of clay into Klipheuwel sand enhanced cohesion while lowering the internal friction angle, as evidenced by the DST and DSST testing methodologies.

5.7. Correlation Factor

The DST and DSST relationship is complex and influenced by the specific conditions under each test. Despite these complexities, general correlations can be established based on the shear strength parameters derived from both testing methods. Figures 11-a and 11-b present the internal friction angle results for various specimens, including Klipheuwel sand, kaolinite clay, and composite soil. The results indicate that the Direct Simple Shear Test (DST) consistently generates a higher internal friction angle than the Direct Simple Shear Test (DSST). This pattern is evident across all soil samples analysed. The variation is especially significant in kaolinite clay, demonstrating an average percentage difference of 9.04% in the internal friction angle relative to Klipheuwel sand. Table 4 presents a detailed summary of the friction angle values for all specimens tested.



(a)



(a)

Figure 11. (a) Shear strength parameters relationship between DST and DSST sand and Kaolinite soil; (b) Shear strength parameters relationship between DST and DSST for composite soil

Table 4. Difference in percentages of φ^0 for all the soil samples

Soil sample	DST	DSST	% Difference	% Average
Klipheuwel	44.2	27.0	38.9	
Kaolin	25.8	16.5	35.9	38.9
Composite Soil	28.2	14.3	49.1	

The results from the composite soil are particularly noteworthy, indicating that a minor addition of clay does not significantly impact the φ^0. The increase in the internal friction angle is observed only at a 75% clay content. Moreover, a mixture of clay and sand shows a substantial decrease in the internal friction angle as the clay content increases, corroborating findings by Dafalla [41].

5.8. Test Mechanics and Results Justifications

This study assessed the shear strength parameters using the DST and the DSST. Each of these methods utilises distinct approaches and serves various purposes. The DST is preferred for its simplicity but has limitations related to stress inhomogeneity. To address these limitations, DSST was developed. In the direct shear test, the shearing action is applied at a predetermined specimen centre, which might not correspond to the weakest plane within the soil sample.

The Direct Shear Test (DST) and the Direct Simple Shear Test (DSST) are extensively utilised in geotechnical engineering to determine soil shear strength. Despite serving similar purposes, these tests differ significantly in mechanics, conditions, and outcomes. In the DST, the soil sample is placed within a shear box split into two parts. A vertical standard load is applied, followed by a horizontal force that induces shear along the interface of the box halves. This test captures the shear stress at the failure point and the corresponding normal stress. On the other hand, the DSST involves lateral confinement of the sample, often achieved using a series of rings or a wire-reinforced membrane. Shear force is applied uniformly across the sample, simulating a plane strain scenario. This method ensures that the vertical stress varies, but the volume of the sample remains constant, simulating undrained conditions.

This distinction explains the variation in results between DST and DSST testing conditions. Using the Mohr-Coulomb failure criterion, the DST determines shear strength parameters (c, φ^0). However, the results are affected by the predefined shear plane, which may not correspond to the soil's weakest plane. Additionally, the DST assumes constant vertical stress during shearing, allowing sample height and volume changes. In contrast, the DSST evaluates shear strength under constant volume (undrained) conditions, better simulating real-world conditions for saturated soils.

Furthermore, the DSST does not restrict the failure plane, enabling the soil to fail along its weakest path. The DSST results are also influenced by principal stress rotation, a factor not considered in DST. It is essential to evaluate the DST and DSST results in this study. In DST, the stress distribution is uneven due to the rigid boundaries of the shear box, which can result in stress concentrations and an overestimation of shear strength. In contrast, DSST achieves a more uniform stress distribution, providing a closer representation of real-world conditions where stress rotation and redistribution occur. Regarding failure planes, DST constrains failure to a predefined aircraft, which may not align with the soil's natural weakest plane, potentially leading to higher shear strength measurements. Conversely, DSST enables failure along the soil's weakest path, offering a more accurate representation of shear strength.

The results were also assessed based on volume changes during testing. DST allows volume alterations during shearing, which can impact pore pressure and shear strength, especially in saturated soils. On the other hand, DSST maintains constant volume throughout the test, simulating undrained conditions and providing valuable insights into the behaviour of saturated soils under rapid loading. Additionally, DST does not account for the rotation of principal stresses, an essential factor in many geotechnical scenarios. By incorporating principal stress rotation, DSST proves more effective for analysing complex stress conditions, such as those beneath embankments or around pile foundations. Overall, the DST tends to mobilize higher shear strengths than DSST, largely due to its prescribed shear plane and boundary conditions. Reliance solely on DST- derived parameters could lead to underestimated safety factors and non-conservative designs. It is assertined that the DSST better captures fabric anisotropy and realistic shear surface development, providing more conservative strength estimates. Therefore, applying an empirical reduction factor to DST strengths based on corresponding DSST results to ensure adequate safety margins is recommended.

6. Conclusion

The findings indicate that the DST yields higher strength estimates than the DSST. The comparative evaluation of shear strength across the tested soils revealed a remarkably consistent difference between the two methods. This uniformity was especially pronounced in specimens compacted at the optimum moisture content, although a slight scatter of values appeared in the wetter samples. The divergence in strength outcomes stems directly from the contrasting shear modes: in the conventional direct shear test (DST), shearing is confined to the lower half of the specimen, concentrating stresses and yielding higher peak strengths; in the direct simple shear test (DSST), the entire soil column undergoes uniform shearing, distributing deformation more evenly but producing lower strength readings. Because the DSST mobilises shear resistance throughout the full height of the sample, it better replicates field-like stress paths and strain distributions, thereby offering a more realistic assessment of in situ shear behaviour. Consequently, deploying the DSST protocol is likely to generate shear strength parameters that more accurately reflect the performance of soils under actual loading conditions.

Despite the systematic strength offset between DST and DSST results, the two methods exhibit a strong proportional relationship. Yet their divergent mechanisms and fundamental assumptions mandate the development of a robust correlation framework to translate parameters from one test into the context of the other. For instance, the internal friction angle measured in the DST can serve as a reference point for interpreting DSST outputs, but deriving a precise conversion factor remains a complex challenge that warrants further experimental and analytical study. Ultimately, this work emphasises that method selection must align with project objectives: while the DST offers simplicity and wide historical precedent, the DSST provides a more comprehensive shear profile. By integrating insights from both approaches, geotechnical engineers can enhance the reliability of designs and tailor test strategies to the nuanced demands of varied soil conditions.

7. Declarations

7.1. Author Contributions

Conceptualization, D.K. and Z.B.; methodology, F.A. and L.S.B.; software, Z.B.; validation, D.K., F.A., and F.C.; formal analysis, D.K.; investigation, Z.B.; resources, D.K.; data curation, Z.B.; writing—original draft preparation, L.S.B.; writing—review and editing, F.A.; visualization, L.S.B.; supervision, D.K.; project administration, F.C.; funding acquisition, D.K. All authors have read and agreed to the published version of the manuscript.

7.2. Data Availability Statement

The data presented in this study are available in the article.

7.3. Funding and Acknowledgements

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7.4. Conflicts of Interest

The authors declare no conflict of interest.

8. References

[1] Aneke, F. I., Onyelowe, K. C., Ebid, A. M., Nwobia, L. I., & Adu, J. T. (2022). Predictive models of swelling stress—a comparative study between BP- and GRG-ANN. Arabian Journal of Geosciences, 15(17), 1438. doi:10.1007/s12517-022-10706-1.

[2] Ukkurthi, V. R., Prasad, K. N., Reddy, B. S., & Babu, J. S. (2023). Prediction of undrained shear strength characteristics of compacted residual soils. AIP Conference Proceedings, 2754(1), 0161126. doi:10.1063/5.0161126.

[3] Aneke, F. I., Mostafa, M. M. H., & El Kamash, W. (2021). Pre-compression and capillarity effect of treated expansive subgrade subjected to compressive and tensile loadings. Case Studies in Construction Materials, 15, 575. doi:10.1016/j.cscm.2021.e00575.

[4] Ikechukwu, A. F., Hassan, M. M., & Moubarak, A. (2021). Swelling stress effects on shear strength resistance of subgrades. International Journal of Geotechnical Engineering, 15(8), 939–949. doi:10.1080/19386362.2019.1656445.

[5] Amšiejus, J., Dirgeliene, N., Norkus, A., & Skuodis, Š. (2014). Comparison of sandy soil shear strength parameters obtained by various construction direct shear apparatuses. Archives of Civil and Mechanical Engineering, 14(2), 327–334. doi:10.1016/j.acme.2013.11.004.

[6] Aneke, I. F., Shamshad, A., Denis, K., & Siddiqua, S. (2024). Enhancing Dynamic Shear Resistance and Efficient Micro-Void Reduction of Expansive Soil Using Activated Nano-Desilicated Fly Ash. International Journal of Geosynthetics and Ground Engineering, 10(4), 72. doi:10.1007/s40891-024-00582-y.

[7] Grognet, M. (2011). The Boundary Conditions in Direct Simple Shear Tests, Development for Peat Testing at Low Vertical Stress. Master Thesis, Delft University of Technology, Delft, Netherlands.

[8] Kjellman, W. (1951). Testing the shear strength of clay in Sweden. Geotechnique, 2(3), 225–232. doi:10.1680/geot.1951.2.3.225.

[9] Zeraati-Shamsabadi, M., & Sadrekarimi, A. (2025). A DEM study on the effects of specimen and particle sizes on direct simple shear tests. Granular Matter, 27(2), 36. doi:10.1007/s10035-025-01513-y.

[10] Ahmad, M., Al Zubi, M., Almujibah, H., Sabri Sabri, M. M., Mustafvi, J. B., Haq, S., Ouahbi, T., & Alzlfawi, A. (2025). Improved prediction of soil shear strength using machine learning algorithms: interpretability analysis using SHapley Additive exPlanations. Frontiers in Earth Science, 13, 1542291. doi:10.3389/feart.2025.1542291.

[11] Baxter, C. D. P., Bradshaw, A. S., Ochoa-Lavergne, M., & Hankour, R. (2010). DSS Test Results Using Wire-Reinforced Membranes and Stacked Rings. Geo Florida, 600–607. doi:10.1061/41095(365)57.

[12] Onyelowe, K. C., Ebid, A. M., Aneke, F. I., & Nwobia, L. I. (2023). Different AI Predictive Models for Pavement Subgrade Stiffness and Resilient Deformation of Geopolymer Cement-Treated Lateritic Soil with Ordinary Cement Addition. International Journal of Pavement Research and Technology, 16(5), 1113–1134. doi:10.1007/s42947-022-00185-8.

[13] Zhang, H. Y., Li, M., Wells, R. R., & Liu, Q. J. (2019). Effect of soil water content on soil detachment capacity for coarse-and fine-grained soils. Soil Science Society of America Journal, 83(3), 697-706. doi:10.2136/sssaj2018.05.0208.

[14] Aneke, I., Mostafa Hassan, M., & Moubarak, A. (2019). Shear strength behavior of stabilized unsaturated expansive subgrade soils for highway backfill. Bituminous Mixtures and Pavements VII, 111–122, CRC Press, CRC Press, United States. doi:10.1201/9781351063265-18.

[15] Zhang, H., Hu, J., Li, Z., Zheng, B., Jin, H., Chou, Y., Li, H., Lu, M., & Yang, S. (2025). Measurement and modeling of excess pore-water pressure in warm saturated frozen soil based on dynamic loading effect. Alexandria Engineering Journal, 110, 132–144. doi:10.1016/j.aej.2024.10.007.

[16] Carraro, J. A. H. (2017). Analysis of simple shear tests with cell pressure confinement. Geomechanics and Geoengineering, 12(3), 169–180. doi:10.1080/17486025.2016.1193635.

[17] Dabeet, A., Wijewickreme, D., & Byrne, P. (2015). Evaluation of stress strain non-uniformities in the laboratory direct simple shear test specimens using 3D discrete element analysis. Geomechanics and Geoengineering, 10(4), 249–260. doi:10.1080/17486025.2014.979889.

[18] Wroth, C. P. (1984). The interpretation of in situ soil tests. Géotechnique, 34(4), 449–489. doi:10.1680/geot.1984.34.4.449.

[19] Li, Y., Chan, L. S., Yeung, A. T., & Xiang, X. (2013). Effects of test conditions on shear behaviour of composite soil. Proceedings of the Institution of Civil Engineers - Geotechnical Engineering, 166(3), 310–320. doi:10.1680/geng.11.00013.

[20] S. Gundersen, A., C. Hansen, R., Lunne, T., L'Heureux, J.-S., & O. Strandvik, S. (2019). Characterization and engineering properties of the NGTS Onsøy soft clay site. AIMS Geosciences, 5(3), 665–703. doi:10.3934/geosci.2019.3.665.

[21] Hassona, F., Abu Bakr, A. M., Abdelmalak, R. I., & Abbas, A. A. (2025). Estimation of Shear Strength Parameters For C-Phi Soils. Journal of Advanced Engineering Trends, 44(1), 74-81.

[22] ASTM D1140-00. (2017). Standard Test Methods for Amount of Material in Soils Finer Than the No. 200 (75-um) Sieve. ASTM International, Pennsylvania, United States. doi:10.1520/D1140-00.

[23] Mitchell, J. K., & Soga, K. (2005). Fundamentals of soil behavior (3rd Ed.). John Wiley & Sons Inc., Hoboken, United States.

[24] ASTM D3080/D3080M-11. (2020). Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions. ASTM International, Pennsylvania, United States. doi:10.1520/D3080_D3080M-11.

[25] ASTM D6528-07. (2017). Standard Test Method for Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils. ASTM International, Pennsylvania, United States. doi:10.1520/D6528-07.

[26] Ikechukwu, A. F., Hassan, M. M., & Moubarak, A. (2021). Resilient modulus and microstructure of unsaturated expansive subgrade stabilized with activated fly ash. International Journal of Geotechnical Engineering, 15(8), 915–938. doi:10.1080/19386362.2019.1656919.

[27] Hidayat, R. F., Kiyota, T., Umar, M., Rosiyani, & Nawir, H. (2022). Flow deformation characteristics of sandy soils under constant shear stress with water inflow. Soils and Foundations, 62(3), 101128. doi:10.1016/j.sandf.2022.101128.

[28] Gylland, A. S., Jostad, H. P., & Nordal, S. (2014). Experimental study of strain localization in sensitive clays. Acta Geotechnica, 9(2), 227–240. doi:10.1007/s11440-013-0217-8.

[29] Acharya, B., & Airey, D. (2017). Effect of Specimen Confinement Method on Simple Shear Test of Clay. Advances in Laboratory Testing and Modelling of Soils and Shales (ATMSS), ATMSS 2017, Springer Series in Geomechanics and Geoengineering, Springer, Cham, Switzerland. doi:10.1007/978-3-319-52773-4_28.

[30] Huang, D., & Zhu, T. (2019). Experimental study on the shear mechanical behavior of sandstone under normal tensile stress using a new double-shear testing device. Rock Mechanics and Rock Engineering, 52(9), 3467-3474. doi:10.1007/s00603-019-01762-3.

[31] Wang, J. P., Ling, H. I., & Mohri, Y. (2007). Stress-Strain Behavior of a Compacted Sand-Clay Mixture. Soil Stress-Strain Behavior: Measurement, Modeling and Analysis. Solid Mechanics and Its Applications, 146. Springer, Dordrecht, Netherlands. doi:10.1007/978-1-4020-6146-2_31.

[32] Zhao, Y. R., Xie, Q., Wang, G. L., Zhang, Y. J., Zhang, Y. X., & Su, W. (2014). A study of shear strength properties of municipal solid waste in Chongqing landfill, China. Environmental Science and Pollution Research, 21(22), 12605–12615. doi:10.1007/s11356-014-3183-2.

[33] Baig, A. A., Baig, N. A., & Baig, S. A. (2023). The Effect of Dry Density on Shear Strength Parameters of Ravi, Chenab and Lawrencepur Sand in Dry and Saturated Conditions. SSRN Electronic Journal, 4644744. doi:10.2139/ssrn.4644744.

[34] Natarajan, K., Siva Sankara Reddy, D.V. (2022). Influence of Density and Degree of Saturation on the Shear Strength Characteristics of Marine Sands. Ground Characterization and Foundations. Lecture Notes in Civil Engineering, Springer, Singapore. doi:10.1007/978-981-16-3383-6_6.

[35] Narayan, S., Maharaj, R. R., Kumar, R., Kishore, T., Salahuddin, M., & Mamun, K. (2024). Comparative Evaluation of Compressive Strength in Earth Blocks Enhanced with Natural Fibers. Civil Engineering Journal, 10(10), 3322–3338. doi:10.28991/CEJ-2024-010-10-013.

[36] Skuodis, S., & Norkus, A. (2014). Influence of test quantity on loose sand shearing strength parameters. Proceedings of the International Conference on Innovative Materials, Structures and Technologies, 167. doi:10.7250/iscconstrs.2014.28.

[37] Onyelowe, K. C., Eidgahee, D. R., Jahangir, H., Aneke, F. I., & Nwobia, L. I. (2023). Forecasting Shear Parameters, and Sensitivity and Error Analyses of Treated Subgrade Soil. Transportation Infrastructure Geotechnology, 10(3), 448–473. doi:10.1007/s40515-022-00225-7.

[38] Vithana, S. B., Nakamura, S., Gibo, S., Yoshinaga, A., & Kimura, S. (2012). Correlation of large displacement drained shear strength of landslide soils measured by direct shear and ring shear devices. Landslides, 9(3), 305–314. doi:10.1007/s10346-011-0301-9.

[39] Kong, Z., Guo, Y., Mao, S., & Zhang, W. (2023). Experimental Study on Shear Strength Parameters of Round Gravel Soils in Plateau Alluvial-Lacustrine Deposits and Its Application. Sustainability (Switzerland), 15(5), 3954. doi:10.3390/su15053954.

[40] Aneke, F. I., & Onyelowe, K. C. (2022). Applications of preloading pressure on expansive subgrade treated with nano-geopolymer binder for cyclic crack resistance. Nanotechnology for Environmental Engineering, 7(3), 593–607. doi:10.1007/s41204-022-00250-4.

[41] Dafalla, M. A. (2013). Effects of clay and moisture content on direct shear tests for clay-sand mixtures. Advances in Materials Science and Engineering, 2013. doi:10.1155/2013/562726.