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# Mechanical Performance of Expansive Clay Subgrades Under Heavy Compaction and Soaked Loading Conditions

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***Abstract: Expansive clay subgrades are highly moisture-sensitive, and their mechanical response under saturated conditions often governs the performance of flexible pavements and thickness design. This study experimentally evaluates the short-term mechanical performance of an untreated, remoulded expansive clay compacted with high compaction energy and tested under soaked loading conditions. Laboratory testing was carried out according to relevant Indian Standards and comprised of Modified Proctor compaction (IS 2720 Part 8), soaked California Bearing Ratio CBR testing after soaking for 96 h at 2.5 kg surcharge (IS 2720 Part 16), and unconsolidated-undrained UU direct shear testing as three runs to add normal stress. The results of the Modified Proctor tests showed a well-defined compaction peak: Maximum Dry Density (MDD) = 1.780 g/cm3 at Optimum Moisture Content (OMC) = 17.80%. The soil under saturated conditions showed low penetration resistance, with CBR = 4.96% at 2.5 mm and CBR = 3.86% at 5.0 mm, indicating that the saturated bearing capacity is limited under stable, ductile load penetration. UU direct shear response showed progressive mobilisation of shear resistance with increasing confinement, with peak shear stress increasing from approx 0.54 kg/cm2 (Run 1) to 0.80 kg/cm2 (Run 2) and 0.94 kg/cm2 (Run 3). The Mohr-Coulomb failure envelope was linear in the stress range tested, yielding shear. strength c = 0.36 kg/cm2, ph = 21.9deg (shear strength parameters given in kg/cm2 consistent with laboratory output). Overall, the sum of the results suggests that the short-term shear behaviour of the soil is predictable, while the soaked bearing resistance is mechanically limiting the soil and thus strength-based parameters of the soil (soaked CBR and shear parameters) may be used to select between pavement thickening and subgrade improvement/stabilisation in moisture-prone service conditions.***

***Keywords: Performance evaluation; Expansive soil subgrade; Moisture-induced strength degradation; Soaked CBR response; Shear strength behaviour; Bearing capacity assessment***

## I. INTRODUCTION

In pavement engineering, the serviceability of flexible pavements depends not only on the quality of the surface and base layers but also, critically, on the strength, stiffness, and deformation response of the subgrade. This dependence is more serious when pavements are built over wide clays, where seasonal variation in soil moisture changes the soil's stress state, fabric, and pore-water regime, causing significant instability under traffic loading. Field and laboratory evidence consistently indicate that expansive soils undergo progressive deterioration of mechanical properties during wet seasons, transitioning from relatively stiff, load-bearing states to softening with low-strength characteristics, during which rutting, cracking, and a decrease in ride quality are likely (Murmu et al., 2019; Safi & Singh, 2022). Beyond simple seasonal trends, the soil's structure also governs expansive clay response and how it might evolve during the loading, wetting, and consolidation stages. Specific experimental studies have shown that both undisturbed and compacted expansive clays can retain "memory" of long-term wetting and drying cycles, and the swelling due to inundation can be as disruptive to the structure of the soil as the laboratory preparation process and can therefore influence strength and deformation characteristics in a way that is of direct importance for pavement performance (Gaspar et al., 2022).

A major concern in the use of expansive soils as pavement subgrades is their rapid loss of bearing capacity under continued saturation. The California Bearing Ratio (CBR) is still a widely used design parameter for determining the thickness of flexible pavement, and a reduction in CBR under soaked conditions is a direct indicator of subgrade weakening and the need to increase pavement thickness. Review-based pavement design analysis confirms that subgrade CBR improvement can significantly reduce pavement layer thickness and alter design adequacy when considering IRC-based approaches as well as mechanistic tools such as IITPAVE (Mukherjee & Ghosh, 2021). However, expansive soils often have soaked CBR values that are below acceptable limits; therefore, the evaluation of soaked performance is a critical step in design realism and durability planning.

 

Experimental stabilisation investigations show that expansive soils are characterised by low unconfined strength coupled with high swelling potential in their untreated state. Improvements in the soaked CBR coupled with improvements in strength are more dependably translated to pavement-level improvement compared with changes in index properties alone (Rabab'ah [et al.], 2023; Zhao [et al], 2025). In this sense, soaked bearing assessment is not just a classification task but an operational measure of performance related to pavement distress and risk (riersicher) in terms of pavement age and damage cost-benefit (community tax vs cost: pavement thickening vs subgrade enhancement).

In addition to bearing capacity indicators, shear strength behaviour is fundamental to the stability of subgrades because traffic loading induces repeated shear stresses. In contrast, moisture ingress will reduce the effective stress and the ability of interparticle bonding. Direct shear and similar tests of shear strength are therefore still required to determine parameters such as cohesion and the friction angle, which control failure and deformation processes in pavement foundations. Stabilisation-basedStabilisation-based experimental work on expansive soils shows a steady increase in strength (UCS and shear resistance) and stiffness, with denser, more bonded soil fabric, whereas untreated expansive soil is usually vulnerable to softening and loss of shear resistance (Murmu et al., 2019; Nadeem et al., 2024). Moreover, thermal treatment studies focus on considerably modifying expansive soil behaviour through mineralogical and microstructural transformations, with swelling pressure reduction, as well as alterations in compaction and strength responses above threshold temperatures, reinstating the sensitivity of expansive subgrades to moisture, microstructure, and strength-response coupling (Tamiru et al., 2025). Collectively, these findings support the argument that performance-based subgrade evaluation should include strength parameters because they reflect the governing response under loading rather than just the soil's properties at the classification level.

A very important mechanism underlying the strength loss and risk of deformation in expansive subgrades is hydromechanical coupling, which occurs during unsaturated-to-saturated transitions. In expansive soils, the rate of infiltration modifies suction stress and the effective stress regime, which in turn dictates the development of swelling pressure and the mechanical response. Advanced analytical modelling reveals that lateral swelling pressure in constrained unsaturated expansive soils can be explained within the framework of infiltration-induced variation in suction stress, linking soil-water retention behaviour, hydraulic conductivity evolution, and swelling-shrinkage characteristics to a coupled framework (Abdollahi & Vahedifard, 2021). Complementing this, experimental and conceptual efforts show expansive soils to display dual porosity behaviour, in which microstructural pore systems govern the reversible expansion and shrinkage behaviour, and macropore structure makes a contribution to plastic volumetric strain until equilibrium is reached, providing a mechanistic explanation for moisture-driven change in deformation and strength of interest for pavement subgrades (Al-Dakheeli & Bulut, 2019). Importantly, laboratory measurements of the soil-water retention behaviour are also influenced by rate of drying and wetting, with accelerated moisture paths yielding suction deviations associated with reduced permeability at lower degrees of saturation, resulting in a conclusion that moisture sensitivity is not only a function of "how wet" the soil becomes, but also "how fast" the moisture regime changes (Azizi et al., 2020). These insights together build the case for pavement subgrade evaluation frameworks that take into account moisture sensitivity and its effects on strength and bearing performance.

While stabilisation approaches using binders, waste, and alternative sustainable additives continue to expand, several studies highlight that the value of any improvement strategy depends on the extent and nature of untreated soil deficiencies. Broad reviews and applied discussions indicate that subgrade problems, the choice of additive, stabilisation technique, and economic valuations should be based on the natural soil's starting performance limitations (Daud et al., 2019; Zimar et al., 2022). Recent experimental studies on expansive soil treatment with recycled and waste-derived materials also further confirm the fact that pavement performance improvements are mainly mediated by increases in resilient modulus, CBR, and compressive strength, while also by reduction in swelling-shrinkage potential that is, in many cases, also accompanied by microstructural observations (Yaghoubi et al., 2021; Perera et al., 2022; Zhu et al., 2022; Priyadarshi & Sharma, 2025). Similarly, industrial solid waste additives and geopolymers systems have been shown to improve strength, shear response and CBR by densification with cementitious product formation, with the emphasis given on mechanical performance (practical target variable for subgrade readiness) (Sahoo & Singh, 2022; Nadeem et al., 2024; Marathe et al., 2025). Reinforcement-based field observations also reveal that geogrid inclusion can counter longitudinal cracking caused by expansive clay movement, strengthening the case for incorporating mechanical performance mechanisms to manage expansive subgrade movement rather than relying solely on classification metrics (Zornberg & Gupta, 2009).

In this regard, the current study aims to evaluate the performance of expansive soil proposed for pavement subgrade applications under criteria based on its strength and bearing capacity in moisture-sensitive conditions.

The experimental program's emphasis is rightly on heavy compaction (modified Proctor), soaked CBR response, and direct shear strength behaviour to capture the limitations of mechanical strength in untreated expansive subgrade under service-relevant wetting.

The goals are to (i) determine soaked bearing capacity using CBR (a pavement design relevant parameter), (ii) to define (shear strength parameter governing subgrade stability and deformation resistance) parameters, and (iii) to develop a set of practical performance benchmarks leading to the rational decision making for subgrade improvement or stabilisation for pavement applications. By grounding the analysis in measured performance indicators while interpreting results in the context of moisture-related mechanisms of strength degradation incidents reported in the latest literature, this study provides a practical baseline recorded information for engineering evaluation and design decisions for expansive soil regions (Mukherjee & Ghosh, 2021; Abdollahi & Vahedifard, 2021; Gaspar et al., 2022; Rabab'ah et al., 2023; Tamiru et al., 2025).

## II. MATERIALS AND METHODS

### A. Material description and sample identification

The experimental programme consisted of two laboratory investigations conducted on a sample of remoulded soil identified in the laboratory records as "Soil, Lime and Coal." The testing programme included: (i) Heavy compaction (Modified Proctor) of specimens prepared for soil CBR soaked, and (ii) direct shear tests under U (unconsolidated - undrained) (UU) condition. All tests were carried out in accordance with the corresponding Indian Standard (IS) procedure, as indicated in the laboratory sheets.

### B. Heavy compaction test setup (Modified Proctor)

Heavy compaction testing was done as per IS 2720 (Part 8): 1983 (RA, 2020). The soil size fraction used for specimen preparation was smaller than a 37.5 mm sieve, as per the test record. The heavy compaction setup consisted of a 150mm diameter mould with a 1000 cc volume, applied in 5 layers using a rammer at 4.9kg, fall of 450mm. The details of the identification and setup of the test heavy compaction test set are documented in Table 1.

Table 1. Test Report – Sample and Compaction Details (Heavy Compaction / Modified Proctor)

| Parameter | Details |
|---|---|
| URL | TC148982500000001F |
| Report No. | TP/2025/11/08 |
| Reporting Date | 08-11-2025 |
| Sample Type (Visual) | Soil, Lime and Coal |
| No. of Layers | 5 layers |
| Compaction Type | Heavy |
| Diameter of Mould | 150 mm |
| Weight of Rammer | 4.9 kg |
| Height of Mould | 127.3 mm |
| Fall of Rammer | 450 mm |
| Sample Passing Through | 37.5 mm |
| Volume of Mould | 1000 cc |
| Weight of Empty Mould + Base Plate | 5355 g |
| Date of Test | 03/11/2025 to 07/11/2025 |
| Standard | IS 2720 (Part 8): 1983, RA 2020 |

### C. Soaked CBR test conditions and specimen preparation details

The soaked CBR test was carried out in accordance with IS 2720 (Part 16): 1987 (RA, 2021), using a remoulded specimen prepared under heavy compaction conditions. The laboratory record showed that the specimen preparation was performed using dynamic compaction, and the testing was performed under saturated conditions after 96 hours of soaking, with a surcharge weight of 2.5 kg used during soaking and testing. The record further mentions that the percentage of replacement of the soil fraction larger than 37.5 mm was Nil, and the loading device constant used for interpretation was 1 division = 27.8N. The complete CBR test condition record is given in Table 2.

Table 2. CBR Test Details at Heavy Compaction (Soaked Condition)

| Parameter | Details |
|---|---|
| Method used for the preparation of the specimen | Remoulded specimen |
| Type of compaction for the remould specimen | Dynamic |

| | |
|---|---|
| Initial Dial Gauge Reading | 0.00 |
| Final Dial Gauge Reading | 0.00 |
| Expansion Ratio | 0.00 |
| Condition of specimen at test | Soaked |
| Surcharge Weight Used | 2.5 kg |
| Period of Soaking | 96 hours |
| Per cent of soil fraction above 37.5 mm replaced | Nil |
| Loading Device Constant | 1 division = 27.8 N |
| Bulk Density | 2.096 g/cm³ |
| Density of specimen (γd) | 1.780 g/cm³ |
| Water Content | 17.80 % |

### *D. Direct shear test under unconsolidated–undrained (UU) condition*

Direct shear testing was carried out on remoulded specimens using an unconsolidated-undrained (UU) condition in which the specimen was sheared without allowing consolidation and without permitting drainage during shearing. Three shear runs were made with conditions of different normal stresses (seen in the laboratory plots and observation sheets). In each run, a horizontal displacement was applied, and proving ring readings were recorded to determine the shear force and the resultant shear stress (based on the corrected area). Vertical dial readings and specimen thickness were measured simultaneously to assess compression/volume-change tendencies in the specimen's geometry during shearing. The results and discussion section of the report contains the full observation tables (Runs 1-3) as well as the derived shear strength parameters

## III. RESULTS AND DISCUSSION

### *A. Heavy Compaction Characteristics (Modified Proctor Test)*

The heavy compaction behaviour of the remoulded soil sample was measured using the Modified Proctor test as per IS 2720 (Part 8): 1983 (RA, 2020). The objective of this test was to determine the relationship between moisture content and dry density under high compactive effort, which represents field compaction conditions for pavement subgrade layers. The experimental observations, having detailed trial-wise data obtained directly from the laboratory records, are presented in Table 3.

Table 3. Heavy Compaction Test Results

| Trial | Weight of Mould + Soil (g) | Weight of Soil (g) | Water Content, Wc (%) | Bulk density, γm (g/cm³) | Dry Density, γd (g/cm³) |
|---|---|---|---|---|---|
| 1 | 7288 | 1933 | 11.55 | 1.933 | 1.733 |
| 2 | 7347 | 1992 | 13.92 | 1.992 | 1.749 |
| 3 | 7401 | 2046 | 15.88 | 2.046 | 1.766 |
| 4 | 7457 | 2102 | 18.20 | 2.102 | 1.778 |
| 5 | 7418 | 2063 | 19.97 | 2.063 | 1.720 |

A progressive increase in dry density is shown with an increase in moisture content from 11.55% (Trial 1) to 18.20% (Trial 4). Over this moisture range, the dry density increases from 1.733 g/cm3 to 1.778 g/cm3, indicating better packing of the soil particles as moisture availability increases during compaction. This increase indicates that the additional water that was duplicated helped bring the particles closer together and provided extra power (or compactive energy) for transfer.Beyond this range, a decrease in dry density is observed at a moisture content of 19.97% (Trial 5), reaching 1.720 g/cm3. This reduction suggests that adding water beyond the optimum level adversely affects compaction efficiency, as excess moisture fills pore spaces and reduces interparticle contact in the soil under the applied compactive effort.Based on the peak dry density results obtained from the experimental trials, the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) were determined and are summarised in Table 4.

Table 4. Optimum Moisture Content and Maximum Dry Density

| Parameter | Value |
|---|---|
| Optimum Moisture Content (OMC) | 17.80 % |
| Maximum Dry Density (γd max) | 1.780 g/cm³ |

The identified OMC of 17.80% falls within the moisture content range observed between Trials 3 and 4, when the dry density is close to its maximum. The 1.780 g/cm3 MDD is the maximum dry density attainable under the given specifications for the tested soil's heavy compaction characteristics. The good agreement between trial-wise observations and the OMC-MDDs so derived confirms the internal consistency and reliability of the laboratory measurements.The variation of dry density with moisture content is shown graphically in Figure 1, which shows the characteristic compaction curve obtained from the Modified Proctor test.

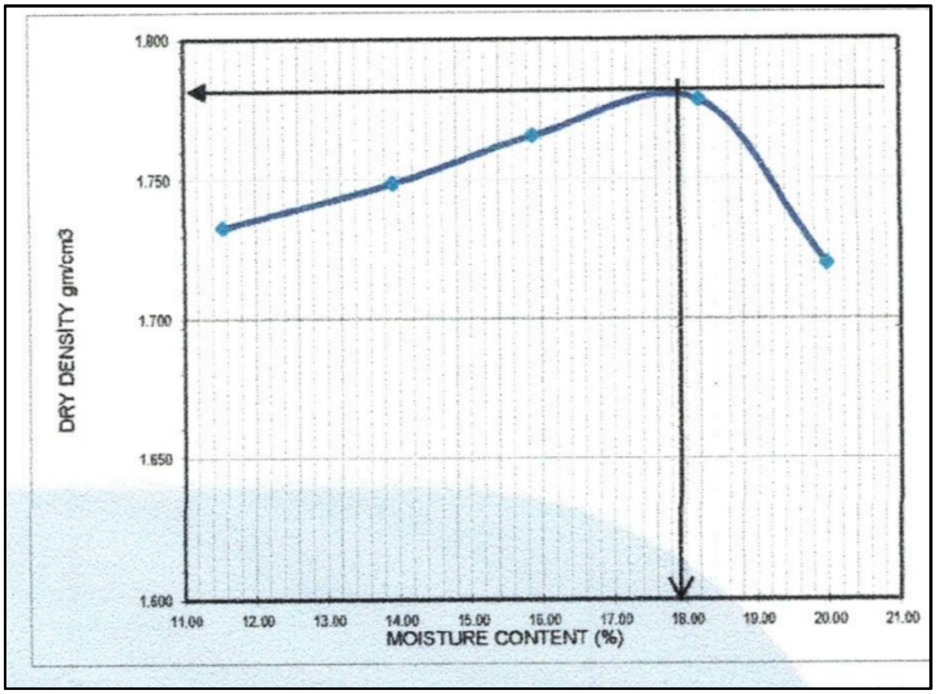


Fig. 1. Variation of dry Density with moisture content obtained from the heavy compaction (Modified Proctor) test.

As shown in Figure 1, the compaction curve shows a gradual increase in dry density with increasing moisture content in the ascending zone of the optimum zone, followed by a descending zone beyond the OMC. The well-defined peak at about 17.80% moisture content confirms the experimentally found optimum condition. This behaviour indicates that the soil exhibits a well-defined compaction response under high compaction energy; hence, OMC and MDD can serve as suitable reference conditions for the subsequent soaked CBR testing and shear strength evaluation performed in this study.

### *B. Soaked California Bearing Ratio (CBR) Behaviour*

The soaked California Bearing Ratio (CBR) behaviour of the soil compacted at the previously determined Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) were evaluated in accordance IS 2720 (Part 16): 1987 (RA 2021). The test was performed on a remoulded specimen that was soaked before loading, and the load-penetration response was measured throughout the test. The detailed experimental observations obtained from the laboratory are given in Table 5.

Table 5. CBR Load–Penetration Test Results

| Penetration (mm) | Load Division | Load (N) |
|---|---|---|
| 0.00 | 0 | 0.00 |
| 0.50 | 7 | 194.60 |
| 1.00 | 11 | 305.80 |
| 1.50 | 15 | 417.00 |
| 2.00 | 18 | 500.40 |
| 2.50 | 24 | 667.20 |
| 3.00 | 25 | 695.00 |
| 4.00 | 27 | 750.60 |
| 5.00 | 28 | 778.40 |

| | | |
|---|---|---|
| 7.50 | 30 | 834.00 |
| 10.00 | 32 | 889.60 |
| 12.50 | 33 | 917.40 |

The load-penetration behaviour obtained from the soaked CBR test reveals a systematic, continuous increase in applied load with increasing penetration depth, as presented in Table 5. At low levels of penetration (up to about 2.0 mm), the load increases rapidly; this represents the initial mobilisation of the resistance within the compacted soil matrix. This response is the additive contribution of particle interlocking and apparent cohesion under soaked undrained conditions.

Beyond this initial stage, the rate of load increase relative to penetration is more gradual. From 2.5 mm onwards, the load-penetration curve shows a more gradual progression, indicating progressive deformation and stress redistribution in the soil mass rather than sudden failure. The load values increase steadily until reaching the maximum recorded penetration of 12.5 mm, demonstrating stable resistance behaviour under sustained penetration.The absence of abrupt drops and irregularities in the load response indicates that the soil exhibits ductile deformation under soaked conditions, characteristic of fine-grained soils compacted to optimum moisture content. This behaviour indicates that the measured load values are reliable for CBR computation and subsequent determination of the governing soaked CBR value at standard penetration depths.The California Bearing Ratio values at standard penetration depths of 2.5 mm and 5.0 mm were calculated from the recorded load-penetration data, and the results are summarised in Table 6.

Table 6. CBR Values of Specimen

| Parameter | Value |
|---|---|
| CBR at OMC | 17.80 % |
| CBR at 2.5 mm penetration | 4.96 % |
| CBR at 5.0 mm penetration | 3.86 % |

The C b r value at 2.5 mm penetration (4.96%) is higher than the value for 5.0 mm penetration (3.86%). As per standard practice, the higher of the two values is considered for design and evaluation purposes. Accordingly, the ratio of the CBR of the soil compacted at OMC and MDD is controlled by the 2.5 mm penetration value of CBR, being called the soaked CBR. This indicates a moderate level of resistance to penetration under saturated conditions for the soil sample tested.The load-penetration behaviour observed during the soaking CBR test is shown in Figure 4.

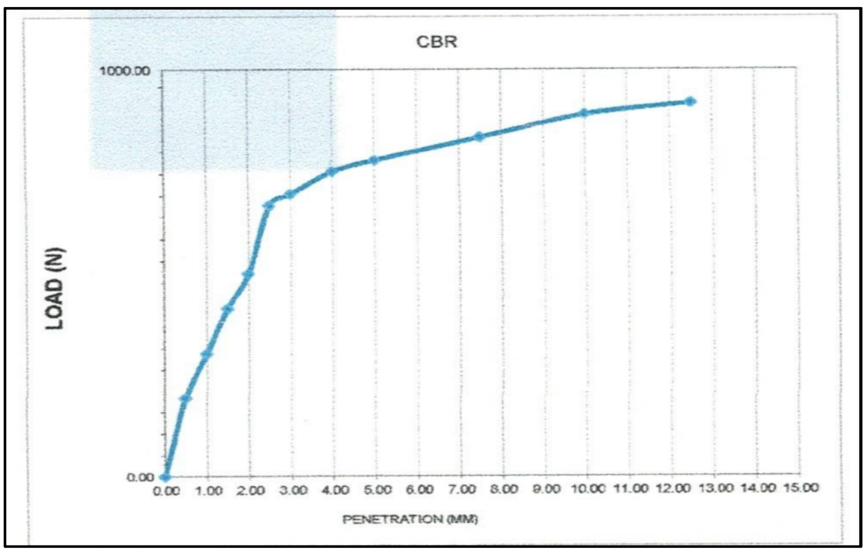


Fig. 2. Loadpenetration curve obtained from the soaked CBR test at heavy compaction.

As shown in Figure 4, initially, for small penetration values, the curve shows a steep increase in load, then becomes smoother and gradually flatter at higher penetrations. This shape represents the progressive mobilisation of the resistance in the soil mass under a compacted shape at saturation.

### C. *Direct Shear Strength Behaviour under Unconsolidated Undrained (UU) Condition*

The shear strength behaviour of the remoulded black clay soil was studied by the direct shear tests performed under unconsolidated undrained (UU) conditions. Three different tests were carried out under a progressive increase in normal stress to quantify the progressive mobilisation of shear resistance and identify the soil's short-term shear strength parameters.The detailed shear response for each of the test conditions is given in Tables 7 to 9, which summarise the variation of shear force, shear stress, specimen thickness and displacement during shearing.

Table 7. Direct Shear Test – Run-1

| Time (min) | Displacement (cm) | Corrected area ($cm^2$) | Proving Ring Div (PR) | Shear Force (kg) | Shear Stress ($kg/cm^2$) | Vertical Dial Reading | Thickness of Specimen (mm) |
|---|---|---|---|---|---|---|---|
| 0 | 0.00000 | 0.000 | 0 | 0.00 | 0.00 | 0.00 | 25.00 |
| 0.25 | 0.03125 | 35.813 | 10 | 3.83 | 0.11 | 0.04 | 24.96 |
| 0.50 | 0.06250 | 35.625 | 15 | 5.75 | 0.16 | 0.13 | 24.87 |
| 0.75 | 0.09375 | 35.438 | 17 | 6.51 | 0.18 | 0.18 | 24.82 |
| 1.00 | 0.12500 | 35.250 | 21 | 8.05 | 0.23 | 0.22 | 24.78 |
| 1.50 | 0.18750 | 34.875 | 25 | 9.58 | 0.27 | 0.31 | 24.69 |
| 2.00 | 0.25000 | 34.500 | 27 | 10.35 | 0.30 | 0.36 | 24.64 |
| 2.50 | 0.31250 | 34.125 | 30 | 11.50 | 0.34 | 0.42 | 24.58 |
| 3.00 | 0.37500 | 33.750 | 32 | 12.26 | 0.36 | 0.46 | 24.54 |
| 3.50 | 0.43750 | 33.375 | 35 | 13.41 | 0.40 | 0.50 | 24.50 |
| 4.00 | 0.50000 | 33.000 | 35 | 13.41 | 0.41 | 0.53 | 24.47 |
| 4.50 | 0.56250 | 32.625 | 36 | 13.80 | 0.42 | 0.55 | 24.45 |
| 5.00 | 0.62500 | 32.250 | 37 | 14.18 | 0.44 | 0.57 | 24.43 |
| 5.50 | 0.68750 | 31.875 | 38 | 14.56 | 0.46 | 0.59 | 24.41 |
| 6.00 | 0.75000 | 31.500 | 40 | 15.33 | 0.49 | 0.60 | 24.40 |
| 6.50 | 0.81250 | 31.125 | 40 | 15.33 | 0.49 | 0.61 | 24.39 |
| 7.00 | 0.87500 | 30.750 | 40 | 15.33 | 0.50 | 0.62 | 24.38 |
| 7.50 | 0.93750 | 30.375 | 40 | 15.33 | 0.50 | 0.63 | 24.37 |
| 8.00 | 1.00000 | 30.000 | 40 | 15.33 | 0.51 | 0.64 | 24.36 |
| 8.50 | 1.06250 | 29.625 | 40 | 15.33 | 0.52 | 0.65 | 24.35 |
| 9.00 | 1.12500 | 29.250 | 40 | 15.33 | 0.52 | 0.66 | 24.34 |
| 9.50 | 1.18750 | 28.875 | 40 | 15.33 | 0.53 | 0.67 | 24.33 |
| 10.00 | 1.25000 | 28.500 | 40 | 15.33 | 0.538 | 0.68 | 24.32 |

Table 7 is the expressed shear response of remoulded soil at the lowest applied normal stress condition. The results show a gradual and continuous increase in shear stress with horizontal displacement; the shear stress reaches a maximum of approximately 0.54 kg/cm2 at higher displacements. The lack of a sharp peak and the subsequent sudden drop suggest progressive mobilisation of shear resistance, which is typical of cohesive soils tested under UU direct shear conditions.A small but consistent decrease in specimen thickness is observed with shearing, indicating that low compression is exhibited during loading and that the restrained drainage behaviour is consistent with the intended UU condition.

Table 8. Direct Shear Test – Run 2

| Proving Ring Div (PR) | Shear Force (kg) | Shear Stress ($kg/cm^2$) | Vertical Dial Reading | Thickness of Specimen (mm) |
|---|---|---|---|---|
| 0 | 0.00 | 0.00 | 0.00 | 25.00 |
| 15 | 5.75 | 0.16 | 0.05 | 24.95 |
| 20 | 7.66 | 0.22 | 0.08 | 24.92 |
| 22 | 8.43 | 0.24 | 0.12 | 24.88 |
| 25 | 9.58 | 0.27 | 0.14 | 24.86 |
| 30 | 11.50 | 0.33 | 0.19 | 24.81 |
| 36 | 13.80 | 0.40 | 0.21 | 24.79 |

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| | | | | |
|---|---|---|---|---|
| 40 | 15.33 | 0.45 | 0.25 | 24.75 |
| 45 | 17.24 | 0.51 | 0.27 | 24.73 |
| 47 | 18.01 | 0.54 | 0.29 | 24.71 |
| 50 | 19.16 | 0.58 | 0.30 | 24.70 |
| 52 | 19.93 | 0.61 | 0.32 | 24.68 |
| 55 | 21.08 | 0.65 | 0.33 | 24.67 |
| 57 | 21.84 | 0.69 | 0.34 | 24.66 |
| 60 | 22.99 | 0.73 | 0.35 | 24.65 |
| 60 | 22.99 | 0.74 | 0.35 | 24.65 |
| 60 | 22.99 | 0.75 | 0.36 | 24.64 |
| 60 | 22.99 | 0.76 | 0.36 | 24.64 |
| 60 | 22.99 | 0.77 | 0.37 | 24.63 |
| 60 | 22.99 | 0.78 | 0.37 | 24.63 |
| 60 | 22.99 | 0.79 | 0.38 | 24.62 |
| 60 | 22.99 | 0.80 | 0.38 | 24.62 |
| 60 | 22.99 | 0.81 | 0.38 | 24.62 |

Table 8 enables the comparison of the shear response under higher applied normal stress to that of Run-1. For similar shearing stages, the mobilised shear stress increases, reaching values close to 0.80 kg/cm2, demonstrating the influence of confinement on shear resistance. The shear stress increases smoothly and approaches a steady state as the proving ring divisions increase, suggesting ductile behaviour and strain hardening rather than brittle failure. Thickness reduction is also still small and gradual, which resembles controlled compression with no abrupt structural collapse during the shearing process.

Table 9. Direct Shear Test – Run 3

| Proving Ring Div (PR) | Shear Force (kg) | Shear Stress (kg/cm²) | Vertical Dial Reading | Thickness of Specimen (mm) |
|---|---|---|---|---|
| 0 | 0.00 | 0.00 | 0.00 | 25.00 |
| 10 | 3.83 | 0.11 | 0.05 | 24.95 |
| 25 | 9.58 | 0.27 | 0.12 | 24.88 |
| 30 | 11.50 | 0.32 | 0.18 | 24.82 |
| 40 | 15.33 | 0.43 | 0.21 | 24.79 |
| 45 | 17.24 | 0.49 | 0.25 | 24.75 |
| 50 | 19.16 | 0.56 | 0.28 | 24.72 |
| 55 | 21.08 | 0.62 | 0.30 | 24.70 |
| 60 | 22.99 | 0.68 | 0.33 | 24.67 |
| 62 | 23.76 | 0.71 | 0.35 | 24.65 |
| 65 | 24.91 | 0.75 | 0.38 | 24.62 |
| 68 | 26.06 | 0.80 | 0.40 | 24.60 |
| 70 | 26.82 | 0.83 | 0.42 | 24.58 |
| 70 | 26.82 | 0.84 | 0.43 | 24.57 |
| 70 | 26.82 | 0.85 | 0.44 | 24.56 |
| 70 | 26.82 | 0.86 | 0.45 | 24.55 |
| 70 | 26.82 | 0.87 | 0.45 | 24.55 |
| 70 | 26.82 | 0.88 | 0.45 | 24.55 |
| 70 | 26.82 | 0.89 | 0.45 | 24.55 |
| 70 | 26.82 | 0.91 | 0.45 | 24.55 |
| 70 | 26.82 | 0.92 | 0.45 | 24.55 |
| 70 | 26.82 | 0.93 | 0.45 | 24.55 |
| 70 | 26.82 | 0.94 | 0.45 | 24.55 |

Table 9 presents the applied normal stress condition, in which the maximum shear resistance occurs among the three tests, with peak shear stress values around 0.94 kg/cm2. The initial increase in shear stress, followed by a plateau, represents the mobilisation of peak/steady resistance under higher confinement. The repeating values of stress at greater divisions of the proving ring indicate reaching a stable condition in which further displacement entails limited further resistance. Specimen thickness change remains small, corresponding with the restrained volume change under the test condition. One can see that the peak shear stress increases systematically with increasing normal stress, supporting the Mohr-Coulomb-type shear strength response of the remoulded soil under test. The progressive mobilisation of shear resistance and the absence of brittle post-peak collapse indicate stable short-term shear behaviour within the laboratory stress range.The shear stress-displacement behaviour for various normal stresses is shown in Figure 5.

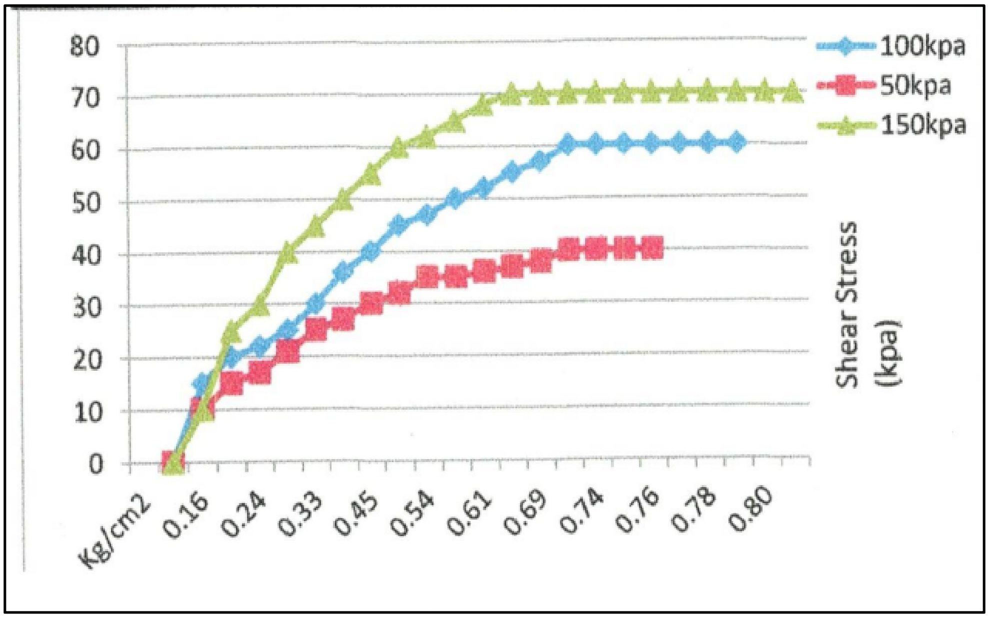


Fig.3. Shear stress vs Displacement response of black clay soil subjected to various normal stresses (50 kPa, 100 kPa, and 150 kPa) obtained via direct shear testing

Figure 5 shows the shear stress against displacement response of the remoulded soil for three increasing values of the normal stress (indicated in the legend of the figure). All curves exhibit a rapid increase in shear stress at small displacements, indicating early mobilisation of resistance through particle rearrangement and bonding. With increasing displacement, the curves separate distinctly: increasing normal stress always mobilises increasing shear stress, demonstrating the confinement dependence of shear resistance. At larger displacement, a reduction in the rate of increase in stress and a tendency towards a near-steady condition indicate progressive mobilisation, which can lead to an abrupt brittle collapse. Overall, the figure confirms a systematic improvement in shear resistance under increased confinement. Naphtali, those with higher confinement would also exhibit lower Mohr-Coulomb-type behaviour within the stress range tested.

Shear Strength Parameters

The maximum shear stress values presented by the 3 unconsolidated undrained direct shear tests (TEST-1, TEST-2, and TEST-3) were applied to define the shear strength characteristics of the remoulded black clay soil. These peak values correspond to the maximum mobilised resistance under increasing normal stresses and were used to determine the Mohr-Coulomb failure envelope, as shown in Figure 6. The derived shear strength parameters, i.e. cohesion and angle of internal friction, are summarised in Table 10.

Table 10. Shear Strength Parameters Calculated from Direct Shear Test

| Parameter | Value |
|---|---|
| Cohesion, *c* | 0.36 kg/cm² |
| Angle of internal friction, $\varphi$ | 21.9° |

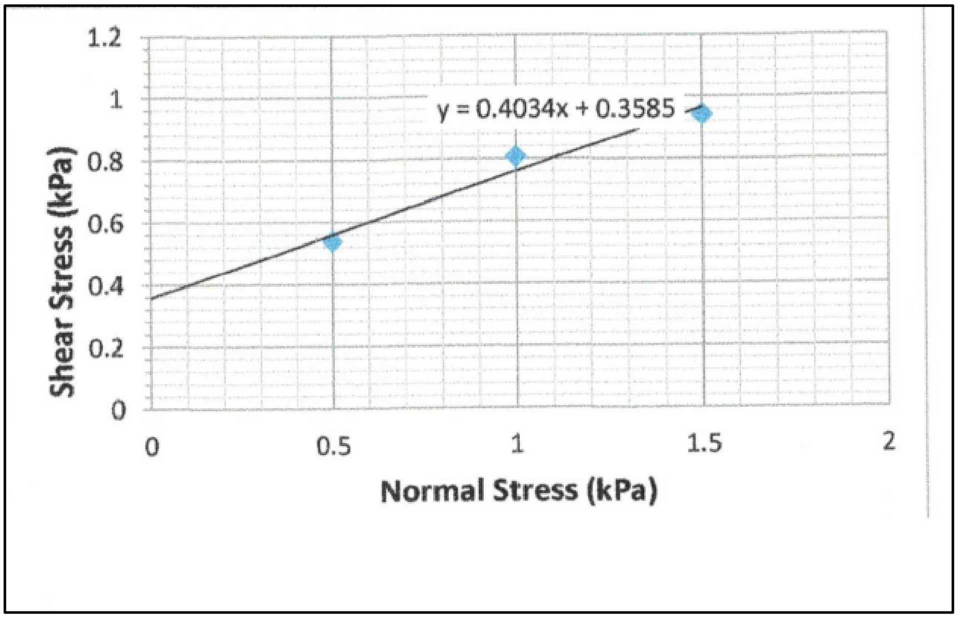


Fig. 4 showing the Mohr-Coulomb failure envelope derived from the experimental data

The results of the maximum shear stress tested obtained from TEST-1, TEST-2 and TEST-3 were used for the construction of the Mohr-Coulomb failure envelope (Figure 6) and the determination of shear strength parameters (Table 10). The linear trend between peak shear stress and applied normal stress indicates that the Mohr-Coulomb criterion is a good representation of the soil response within the stress range tested. The value of cohesion (c = 0.36 kg/cm2) is the apparent intercept obtained at the failure envelope for the condition of remoulded specimens. At the same time, the angle of friction (ph = 21.9 °) is the confinement-dependent contribution to the shear resistance.In the context of the pavement subgrade, these parameters provide useful short-term strength inputs for stability assessment under rapid loading; however, the observed low soaked CBR value for the same soil indicates that saturated bearing resistance is also a major limitation affecting pavement design under wet conditions.

## IV. CONCLUSION

This study was conducted to experimentally evaluate the mechanical performance of a remoulded expansive clay subgrade under heavy compaction and soaked loading, using Modified Proctor compaction, soaked CBR test, and UU direct shear test, based on relevant IS procedures. The Modified Proctor results yielded a well-related compaction curve with a maximum dry density (MDD) of 1.780 g/cm3 and an OMC of 17.80%, indicating a Good compaction response for preparing repeatable laboratory specimens under heavy compaction conditions for subgrade construction.

Under soaked conditions, the soil showed low CBR, controlled by the 2.5 mm penetration value of 4.96% (higher values at 3.86% for 5.0 mm). The load-penetration response revealed a stable, ductile increase in resistance without sudden failure. However, the magnitude of the CBR implies that bearing capacity is limited under saturated conditions, which has direct implications for increasing pavement thickness requirements or the need for subgrade enhancement when a soaked condition is anticipated.

The UU direct shear results showed progressive mobilisation of shear resistance with displacement and definite dependence on applied normal stress. The Mohr-Coulomb envelope was observed to be linear over the tested stress range, and the derived parameters were c = 0.36 kg/cm2 and ph = 21.9 degrees, consistent with a soil exhibiting a certain amount of apparent cohesion and a moderate frictional contribution. Collectively, the results of the soaked CBR and shear strength tests confirm that this soil is moisture-sensitive and that its untreated soaked behaviour is mechanically limiting for subgrade service conditions in pavements. Therefore, for practical pavement engineering, the study supports considering strength-based performance as decision variables (soaked CBR and shear parameters in this case) to rationalise whether to proceed with subgrade improvement/stabilisation or design with increased pavement layer thickness to manage wet-season performance risks.

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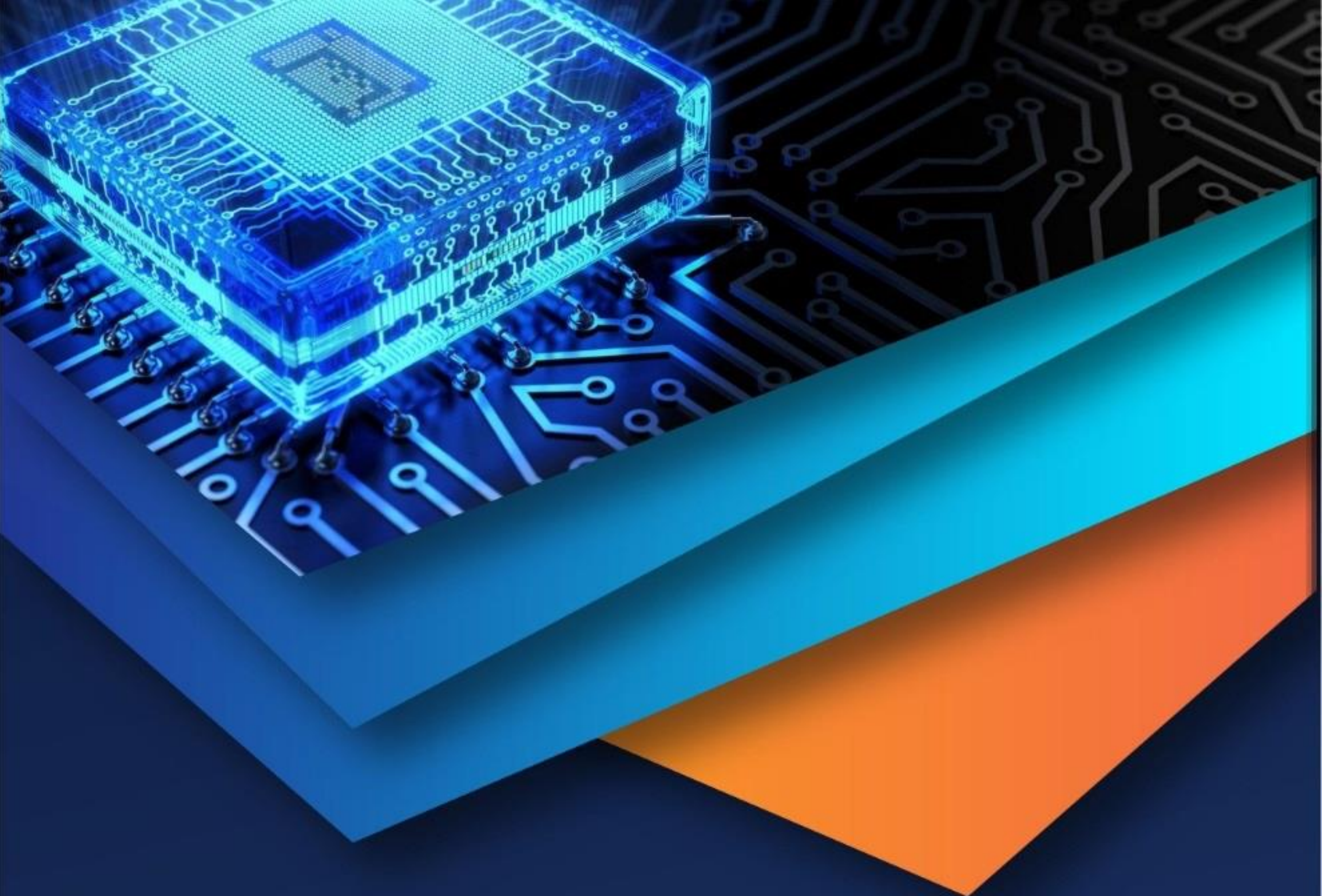


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