The influence of Keruing sawdust on the geotechnical properties of expansive Soils

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Abstract. Expansive soils have proved to be problematic to most Civil Engineering structures. Several researchers have tried to look for different materials which can alter the properties of these poor soils, and among them are lime and cement, which are expensive. Considering the vast quantities of sawdust produced in woodwork departments, they can be used as a secondary stabilizer, thus leading to sustainable technologies. Sawdust not only acts as a cheap stabilizer but also reduces the problem of environmental pollution caused by its poor disposal. This paper examines the geotechnical properties of expansive soil stabilised by Keruing sawdust. The sawdust used as partial replacement of soil in the ratio of 0, 3, 5, and 7% by the dry soil weight. The investigation was done by conducting laboratory tests on both stabilised and non-stabilized soils. The results showed that Keruing sawdust significantly improved the geotechnical properties of the soil by reducing the plasticity index from 64% to 36% at 0% and 7% sawdust, respectively. The unconfined compressive strength and California Bearing Ratio also improved at 3% sawdust + 97% soil. It concluded that Keruing sawdust, a waste material could be used as a cheap additive to the expansive grounds.

1. Introduction

The problems associated with expansive soils have been reported all over the world. Extensive grounds have been a threat to Civil Engineering projects such as roads, buildings, bridges, dams, and many others [1]. Particular issues associated with constructions over expansive soils are mostly the seasonal volumetric change in these soils rather than their low bearing strength since expansive soils are often relatively stable at equilibrium water content[2]. Deformations like cracking, corrugations, and soil pumping mainly occur due to poor subgrade soils[3], [4]. Expansive soils tend to shrink or to break during the dry season and swell during the wet season. The shrink and swell behaviours caused by the presence of montmorillonite, a clay mineral that has an expanding lattice [5], [6]. Many scientists found that when the clay soil exposed to water, the montmorillonite clay mineral enlarges. It is believed that montmorillonite clay minerals are found in many parts of the world, more especially the arid and semi-arid regions[7]. The effect of the clay mineral is that it allows water to penetrate the soil, thus leading to cracks, hence causing considerable expansion. The result is pavement deterioration since the expansion, and the subsequent heave is never uniform. The only way to
minimize this problem is to conduct more research on possible ways of improving the properties of these weak soils. In ancient times, the more fragile soils were replaced by the quality of borrow soil, but this option is costly because it involves excavation and transportation costs[8]–[10]. Other techniques were introduced to solve the challenge of costs, such as stabilization to modify the properties of the subgrade soils. The engineers have attempted to treat the cohesive soils with different methods of subgrade stabilization. Subgrade stabilization is a common technique used to improve soil properties such as shear strength, bearing capacity, and other features. These techniques help to reduce lateral and vertical deformations and settlement under transmitted loads from the structures. Stabilization is done in the form of chemical and mechanical means.

According to Ogunribido[11], lime, cement, and fly ash have been widely used as a stabilizing agent, but their prices are relatively high hence making the cost of road construction expensive. These increasing prices remain a problem for developing countries like Indonesia, thus forcing researchers to look for alternative stabilizers. The re-use of wastes is environmentally friendly, thus becoming the best option to replace traditional methods[12], [13]. The vast quantities of sawdust produced from lumbering industries can be used as secondary stabilizers [14]. According to Horisawa, dry wood consists of compounds with cementitious properties, thus increasing the strength of the soil[4], [15], [16]. This study aims at using Keruing sawdust (SD) to improve on the geotechnical properties of expansive soils.

2. Materials and Methods

2.1 Materials

The materials used in this study were expansive clay soil and sawdust.

2.1.1. Expansive Clay Soil.

The expansive clay soil used in this study was collected along Semarang-Purwodadi road (Godong sub-district, Grobogan District, Central Java Province) at STA 49 Km. The samples were picked within the coordinates 7°02'04.2"S 110°48'09.3"E (See Figure 1). The visual inspection showed that the soil is greyish black and highly plastic. The soil samples were picked at a reasonable depth to avoid the inclusion of organic matter using both disturbed and undisturbed sampling method and then packed in the sacks. They were then transported to the Soil Mechanics Laboratory, Universitas Diponegoro. The samples were air-dried before use.

Figure 1. The study location.
2.1.2. Keruing sawdust.

Keruing is a kind of tree found in Central Java, Indonesia. The sawdust from the Keruing tree (see Figure 2) used in this study was obtained from the Woodwork Department, Politeknik Negeri Semarang (POLINES), Tembalang. Afterwards, it was oven-dried at a temperature of 60°C. Then a selected sample was taken to Integrated Laboratory, Universitas Diponegoro to determine the chemical composition and the morphology of the sample. The test was done using the scanning electron microscope (SEM). After knowing the chemical properties of the sawdust, more samples were taken to the Soil Mechanics Laboratory, Diponegoro University, and sieved using Sieve No.20 (diameter 0.841 mm).

Figure 2. Keruing sawdust.

2.2. Methods

Samples of expansive soil and sawdust-stabilized expansive soils were tested to determine the index properties, particle-size distribution, soil classification, specific gravity, compaction characteristics, UCS, and CBR according to procedures outlined in ASTM codes.

2.2.1. Grain Size Distribution.

The test consists of both sieve analysis and hydrometer test. Hydrometer test was done in accordance with ASTM D422-63 procedures[17] and sieve analysis was done in accordance with ASTM D 6913-04 procedures[18]. Hydrometer test is done in accordance to ASTM D 422-63 procedures determine the particle-size distribution of soils whose diameter is less than 0.075 mm, while sieve analysis is used to determine the particle distribution larger than 0.075 mm.

2.2.2. Atterberg limits test.

The consistency limit tests were done in accordance with ASTM D4318 procedures [19]. These tests help in determining the swelling characteristics of the clay soils hence ensuring that the soil has the correct amount of shear strength and exhibit minimal volume change as it expands and shrinks with different moisture contents [20]. Casagrande’s method was used in the current study to determine the liquid limit of both stabilised and non-stabilized expansive soil. The plasticity index (PI) was calculated from the liquid limit (LL) and plastic limit (PL), as shown in Equation 1.

\[ PI = LL - PL \] (1)

2.2.3. Compaction test.

The Preparation of both stabilised and non-stabilized soil samples for Proctor's standard compaction test was done by ASTM D698[21]. Proctor's analysis helps in determining the maximum dry density (MDD) and optimum moisture content (OMC). The MDD and OMC were determined from the graph of dry density against moisture content. The optimum moisture content was taken as the moisture
content at which the maximum dry density was attained. The bulk density and dry density were obtained with the expression shown in Equations (2) and (3), respectively.

\[ \gamma = \frac{W}{V} \quad (2) \]

\[ \gamma_d = \left[ \frac{\gamma}{1 + \frac{m}{100}} \right] \quad (3) \]

Where \( \gamma \) = Bulk density, \( W \) = Weight of wet soil, \( V \) = Volume of wet soil, \( \gamma_d \) = Dry density of soil, and \( m \) = Moisture content of the soil.

2.2.4 Unconfined Compression Strength (UCS).

The Unconfined Compression Test determines relative undrained shear strengths due to the slightly relaxed in situ pressures of the sample. The test was determined according to the standard of ASTM D-2166\[22\]. According to Das [23], clayey soils whose UCS ranges from 0 to 24 kN/m\(^2\) falls under very soft category, 24 to 48kN/m\(^2\) soft, 48 to 96kN/m\(^2\) medium soft, 96 to 192kN/m\(^2\) stiff, 192 to 383kN/m\(^2\) very thick, and greater than 383 kN/m\(^2\) hard.

The unconfined compression strength is significantly altered for many naturally deposited clayey soils when the soil is tested after remoulding, and therefore, it is better to calculate the sensitivity. Sensitivity is the ratio of the unconfined compression strength from the undisturbed sample to the unconfined strength from the remoulded example (Equation 4). The description of the magnitude of sensitivity (\( S_t \)) is shown in Table 1[24].

\[ S_t = \frac{q_{u(undisturbed)}}{q_{u(remolded)}} \quad (4) \]

Table 1. Description of the magnitude of sensitivity (Das, 2002)

| Sensitivity, \( S_t \) | Description |
|------------------------|-------------|
| 1-2                    | Slightly sensitive |
| 2-4                    | Medium sensitive |
| 4-8                    | Very sensitive |
| 8-16                   | Slightly quick |
| 16-32                  | Medium quick |
| 32-64                  | Very quick |
| >64                    | Extra quick |

2.2.5 California Bearing Ratio (CBR).

The CBR test was carried out on both stabilised and non-stabilised samples by ASTM D-1883 [25]. The CBR value was determined as the ratio of the unit load required to penetrate the specimen of 0.1 or 0.2 inches to the unit load required to penetrate a standard material of well-graded crushed stone (See Equation 3). According to the National Cooperative Highway Research Program [26], CBR values less than 5% are bad, between 5-10 good, 10-20 very good for engineering purposes.

\[ CBR = \frac{F}{F_s} \times 100\% \quad (5) \]

Where; \( F \) is the force required for the sample, and \( F_s \) is the force required for similar penetration into a standard sample (usually 3000 and 4500 lbf at the penetration of 0.1 and 0.2 inches, respectively).
3. Results and Discussion

3.1 Properties of natural soil

The features that were tested on native soil presented in Table 2. The visual inspection shows that the soil is greyish black and belongs to expansive clay. Results for Grain-size distribution were obtained from both hydrometer and sieve analysis. As shown in Figure 3 in the particle size distribution curve, almost 98.64% of the soil is passing through sieve No. 200; it exhibits a liquid limit of 94.51%, a plastic limit of 30.55% and plasticity index of 63.96%. According to Chen [20], plasticity index between 0-15% signifies low swelling potential, between 10% and 35% medium, between 20% and 55% high and greater than 35% very high. Therefore, the value for the soil under consideration is 63.96%, which is very high. Based on the USCS soil classification system, the soil is CH (high plastic clay). According to AASHTO soil classification, the soil falls under the A-7-6 soil class. Soils under this class are generally classified as a material of weak engineering property to be used as subgrade material. Results that are related to swelling characteristics of the soil also indicate that the soil is highly expansive clay with a free swell of about 130%. The soil has a maximum dry density of 1.36 g/cm³ and optimum moisture content of 31.5%.

Table 2. Properties of non-stabilised soil.

| No. | Property                           | Quantity |
|-----|------------------------------------|----------|
| 1   | Percentage passing No. 200         | 98.64    |
| 2   | Liquid limit %                     | 94.51    |
| 3   | Plastic limit %                    | 30.55    |
| 4   | Plasticity index %                 | 63.96    |
| 5   | Specific gravity                   | 2.68     |
| 6   | AASHTO soil classification          | A-7-6    |
| 7   | USCS                               | CH       |
| 8   | Natural moisture content %         | 56.68    |
| 9   | Specific gravity                   | 2.68     |
| 10  | Free swell %                       | 130      |
| 11  | Maximum dry density g/cm³          | 1.36     |
| 12  | Optimum moisture content %         | 31.5     |
| 13  | Remolded UCS (kg/cm²)              | 4.576    |
| 14  | Undisturbed UCS (kg/cm²)           | 0.760    |
| 15  | CBR at 95% compaction (%)          | 10       |
| 16  | CBR at 100% compaction             | 11       |

3.2 Atterberg limits

The liquid limit (LL) and plastic limit (PL) tests were carried out with different percentages of soil and sawdust mixtures. The effects of sawdust content on the liquid limit, plastic limit, and plasticity index (PI) for both the stabilised and non-stabilised soil samples are seen in Table 3 and plotted in Figure 4.
In this study, the liquid limit decreases while the plastic limit increases with increased SD content, thus leading to an overall decrease in plasticity index of soil. The same conclusion was derived from the previous study by Yadav et al. [13] in stabilizing the alluvial soils of India using different kinds of ashes. The decrease in the liquid limit is an indicator of a reduction in the compressibility and swelling potentials. The increased partial replacement of soil with SD, with some cementitious properties, increases the binding ability of the mixtures and its capacity to retain moisture. According to the previous study done in Uganda, Jjuuko et al. [12] found that blends having liquid limit more than 50% are indicative of high plasticity soils, and therefore not suitable for road construction. The decrease in liquid limit may signify the improvement in the behaviour of expansive soil, hence improved workability [13].

Table 3. The variation in consistency limit for stabilised and non-stabilised soils.

|                | 0% SD+100% soil | 3% SD+97% soil | 5% SD+95% soil | 7% SD+93% soil |
|----------------|-----------------|---------------|---------------|---------------|
| LL             | 94.51           | 86.61         | 80.59         | 74.98         |
| PL             | 30.55           | 36.00         | 39.49         | 38.33         |
| PI             | 63.96           | 50.61         | 41.10         | 36.65         |

3.3. Compaction characteristics

The dry densities for the different compositions were plotted against the corresponding moisture contents, as shown in Figure 5. It was observed that the maximum dry density (MDD) decreases, while the optimum moisture content (OMC) increases with the increase in the proportion of sawdust.
The reduction in MDD of the soil sample with increased sawdust content is an indication of improvement of the compaction characteristics of the soil-SD blends. This phenomenon could be attributed to the addition of SD with lower specific gravity than the one of the soil used. Some other researchers used different materials with lower specific gravity than the soil and observed the same phenomenon; for example, Al- soudany et al. [3] used activated carbon to stabilize clay soil, and Yadav et al. [13] also noticed it while using different kinds of ashes to stabilize the alluvial soils. According to Osinubi et al. [14], the expansive soil occupies larger spaces than the stabilizers used, thus increasing their volume and consequently decreasing their dry densities. On the other hand, the reason behind the increase in OMC with growth in the different proportion of sawdust and decrease in the proportion of the soil in the mix is that the SD causes absorption of a high amount of water which takes away the clay particles, thus more water is needed to occupy the place for the clay particles.

Table 4. Optimum moisture content and maximum dry density

| Composition         | OMC (%) | MDD (g/cm³) |
|---------------------|---------|-------------|
| 0%SD+100%soil       | 31.50   | 1.360       |
| 3%SD+97%soil        | 32.00   | 1.259       |
| 5%SD+95%soil        | 35.00   | 1.236       |
| 7%SD+93%soil        | 36.50   | 1.196       |

Figure 5. Compaction characteristics of natural and SD treated soil

3.4. Unconfined compressive strength (UCS)

Figure 6 depicts the variation in unconfined compressive strength of both stabilised and non-stabilised soil. It has been observed that the UCS from the undisturbed specimen (UDS) is 0.760 kg/cm², while the remoulded UCS of the non-stabilised soil from the disturbed example (DS) is 4.576 kg/cm² (see Table 5). This shows that there is a sensitivity of 6.0. According to Das [24], the soil under study is susceptible because the magnitude of the sensitivity lies between 4-8. The UCS values after replacing the soil with 3%, 5%, and 7% are relatively higher than the UCS value of natural soil (4.576 kg/cm²), especially at 3% SD and 97% soil, where the UCS value is 6.387 kg/cm²; however, replacing the soil with 5% and 7% SD reduces the UCS though still higher than the one of the non-stabilised soil. The occurrence is similar to the one reported in the clay silt stabilized with sawdust [10]. The sawdust mainly consists of cellulose, lignin, and hemicellulose [16], and the main component of cellulose is carbon units linked together by a covalent bond. The carbon unit bond is vital for sustaining loads generated to the soil samples, thus the increase in the UCS. Also, the growth could be attributed to the formation of cementitious compounds between the calcium hydroxide present in the soil and the
pozzolans present in the sawdust. A slight decrease in the UCS values at the composition of 5% SD and 95% soil, and 7% SD and 93% soil may be due to the excess SD introduced to the soil, hence forming weak bonds between the soil and the cementitious compounds formed [13].

**Table 5. Recapitulation of UCS and Shear strength for the Soil-SD mixture**

| Composition                  | UCS (kg/cm²) | Shear Strength (kg/cm²) |
|------------------------------|--------------|-------------------------|
| 0%SD+100%soil (UDS)         | 0.760        | 0.380                   |
| 0%SD+100%soil (DS)          | 4.576        | 2.288                   |
| 3%SD+97%soil                | 6.387        | 3.194                   |
| 5%SD+95%soil                | 5.985        | 2.993                   |
| 7%SD+93%soil                | 5.924        | 2.962                   |

Based on the analysis of the results, UCS of the natural soil from the disturbed sample is 4.576 kg/cm² (449kN/m²). After replacing the soil with 3% sawdust, the UCS increased to 6.387kg/cm² (626kN/m²). Both the natural soil and 3% sawdust treated soil falls under the category of hard clay [23].

![Figure 6. Variation in unconfined compressive strength of natural and SD treated soils](image)

3.5. California Bearing Ratio (CBR)

As seen in Table 6, sawdust has shown a significant increase in the CBR value of expansive soil, especially on the unsoaked soil samples. The peak unsoaked and soaked CBR values obtained at 56 blows compaction are 20.53%, and 7.16% at a composition of 3% SD + 97% Soil, while the lowest unsoaked and soaked CBR value are 12.05%, and 4.32% which is for the natural soil (0% SD + 100% Soil). The CBR for the composition 7%SD + 93% soil was not determined in this study because of the decline that was noticed in UCS test and also the decline in CBR at 5%SD + 95% soil. It was observed that all the CBR values for the soaked samples are relatively smaller than the unsoaked samples (see Table 7), this is attributed to the presence of water in the saturated samples that do not have shear strength for engineering purposes. Based on the results obtained from the macerated samples, the CBR
for a non-stabilized sample at 56 blows of compaction is 4.32%. After substituting the soil with 3% sawdust, at the same compaction effort (56 blows), the soaked CBR value increased to 7.16%. According to National Cooperative Highway Research Program [26], the un-stabilized soil falls into bad category, while the soil treated with 3% SD + 97% soil falls under the moderate category. The phenomena signify that sawdust added some strength to the soil.

Table 6. CBR values for the unsoaked samples

| No. Blows | 0% SD+100% Soil | 3% SD+97% Soil | 5% SD+95% Soil |
|-----------|-----------------|----------------|----------------|
| 56        | 12              | 21             | 20             |
| 25        | 11              | 17             | 17             |
| 10        | 7               | 9              | 6              |

Table 7. CBR values for the soaked samples

| No. Blows | 0% SD+100% Soil | 3% SD+97% Soil | 5% SD+95% Soil |
|-----------|-----------------|----------------|----------------|
| 56        | 4.32            | 7.16           | 6.58           |
| 25        | 2.90            | 5.22           | 3.68           |
| 10        | 1.45            | 3.03           | 2.64           |

4. Conclusion

Based on the results of the current study, it is indicated that the soil is not suitable as a subgrade material in its natural state as its properties are weak, hence the need to be stabilized. The replacement of soil with sawdust reduced plasticity index; subsequently, the workability of the soil was improved. The unconfined compressive strength and shear strength increased by 1.4 times one of the un-stabilized soil. The increase occurred after the soil was replaced with the sawdust at 3% and 5% SD of the dry unit weight of the soil, and then decreases after the soil was replaced with 7% SD. It can be concluded that small percentages of Keruing sawdust can be used as a cheap additive to expansive soils to improve on its engineering properties. Also, using sawdust minimizes on the environmental problems that are caused by its poor disposal.

Acknowledgement

The authors are grateful to Diponegoro University, Soil Mechanics laboratory, Department of Civil Engineering for availing to them all the laboratory equipment used in this research.

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