Advances in Civil Engineering

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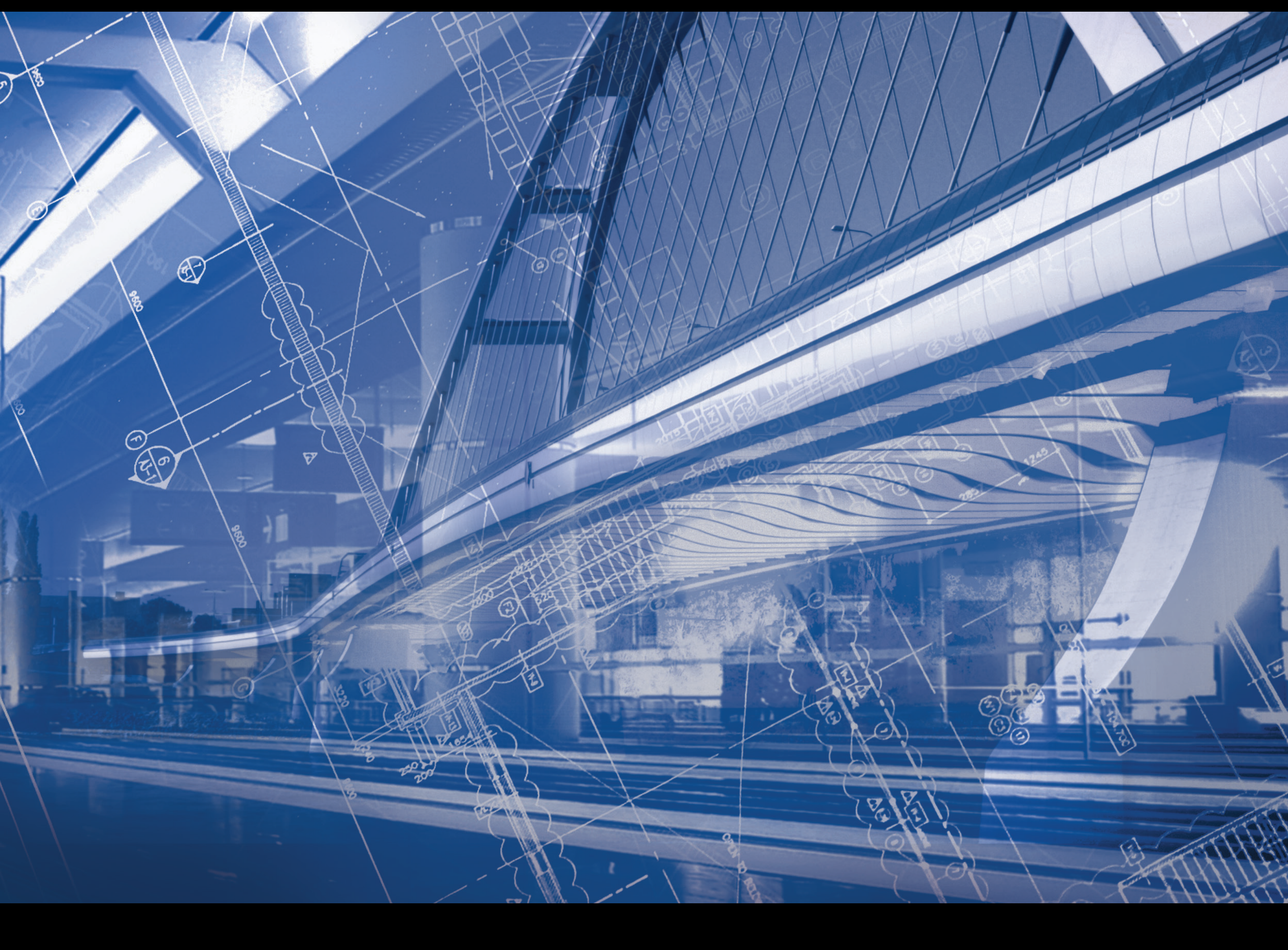

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Research Article

Comparative Study of Swelling Pressure in Expansive Soils considering Different Initial Water Contents and BOFS Stabilization

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In terms of geotechnical engineering, swelling soils are among the most important soil groups whose characteristics should be determined in detail before design studies. These types of soils cause significant damage to engineering structures. For this reason, it is expected that the swelling behavior of the soils will be known in advance to minimize the damage that may occur in the structures. Within the scope of this study, the swelling pressures of bentonite clay with 10 different water content were determined by keeping all conditions the same to reveal the effect of water content on soil swelling behavior. In this context, bentonite-type (montmorillonite content) clay, which has a very swelling property when it comes in contact with water, was used in the experiments. The fixed volume swelling pressure test method was used in the experiments and all samples were compressed at the same rate and placed in the swelling test device. In all samples left to swell with pure water, measurements were made for 10 days and the effects of swelling pressures on the initial water content were discussed. Thereafter, another swelling soil was stabilized using basic oxygen furnace slag (BOFS) during different curing times, and after performing the swelling pressure test, the results were compared with the findings obtained from different initial water contents. According to the results, while the swelling pressures increase in the regions close to optimum water content, significant decreases are observed in swelling pressure values in wetter and drier regions than in optimum water content. Finally, the results indicated that the application of BOFS, albeit small, after the proper curing time can significantly affect the swelling behavior of bentonite, even more than changing the initial water content.

1. Introduction

Determining the swelling properties of soils is of great importance in terms of explaining soil behavior. Drnevich et al. [1] described swelling pressure as the pressure required to keep the soil volume constant when water is added. It can be listed as the expansion caused by the elastic stretching of crystals because of unloading and the swelling caused by the pressure in the compressed air during the progress of wetting in the soil [2–5]. As these soils increase in volume as they get wet, their volume decreases as they dry [6, 7]. Clay soils with swelling properties cause deformations in engineering structures depending on the change in stress conditions [8–11]. Estimates in the literature that the annual cost of damages caused by swelling soil could reach billions of dollars worldwide [12–15]. While settlements occur on the soils due to the increase in stress conditions, swelling may be observed as a result of the decrease in stress. The changes in the volume of the soil mass that occur due to the settlement and swelling properties are the most effective factors in the design of the projects related to the soil.

For this reason, it is extremely important to determine the swelling characteristics of the soil in the buildings to be built on it and to make the necessary precautions by stabilizing before the application phase [14, 16]. Therefore, one of the geotechnical engineering fields of study is knowing how the soil and structure will be affected by different loading and environmental conditions of the soils [17]. One of the most important stages of these evaluations is to determine the swelling properties of clay soils, where large volume changes may occur. In this context, it is necessary to determine and predict the swelling potential of the clayey soil, the maximum pressure level that will occur because of swelling, and the amount of swelling that will occur on the soil surface. When the interaction of clays with water is examined in detail, the predicted problems can be controlled by taking necessary measures [18, 19].

Most engineering structures built on clay soils cause moisture changes in the clay due to the stresses they apply, degrading the natural water content in the clay [20, 21]. Clay swelling occurs as a result of balancing the interaction forces between the clay surface, ions, and water [22, 23]. Due to the increase in the water content of dry soil, the clay grains move away from each other and increase in volume, and as a result, the thrust force occurs [24, 25].

Many factors such as clay percentage, the mineralogical structure of clay, dry unit weight, stress conditions, external loads, water content, and climate conditions can affect the swelling of clay soils [26–30]. Although clays tend to swell more at lower water content than optimum water content, the amount of swelling is negligibly small at water content above optimum [31]. David and Komornik [32] suggested that the swelling pressure increased as the water content decreased. Chen [33] stated in his studies that volumetric swelling decreased due to the increase in water content. Warkentin [34] stated that clayey soils have a significant effect on swelling behavior, and swelling decreases due to the increase in water content. David Suits et al. [35] investigated the effect of curing time on the swelling behavior of soil with a liquid limit of 100%. Samples were kept for 7, 15, 30, and 90 days in glass desiccators with different concentrations of sulfuric acid to maintain different relative humidity conditions. One-dimensional oedometer swelling tests were performed on these samples. They observed that the increase in the waiting time caused a decrease in the swelling potential. In the study, it is stated that the initial saturation degree and water content affect aging. It was emphasized that the aging effect increases as a result of the increase in water content for the same dry unit weight, and the effect becomes more important with the increase in the degree of saturation at constant water content. McCormack and Wilding [36] examined the relationship between soil swelling and clay content by keeping all other parameters constant and emphasized that clay percentage is a parameter that affects the swelling potential in illite-dominated soils. Schafer and Singer [37] explained that the change in the swelling potential of the soil is related to the percentage of swelling clay.

The swelling properties of expansive soils are generally determined by two different techniques. The first of these is the estimation method, which is made by using some soil parameters such as swelling potential, density, Atterberg limits, and clay fraction, known as the indirect method. The other is the method that quantitatively determines the swelling potential of the soil by performing the oedometer test, free swelling test, or swelling index test. Determining the swelling characteristics of the expansive soils directly takes a long time and is costly due to complex laboratory studies [34]. Indirect assessment of swelling requires only a few simple routine laboratory tests. Therefore, it is a usual practice to give a preliminary estimate for the swelling potential using mostly indirect methods, and further tests such as odometers are preferred after problems [38].

There are studies in the literature that examine the effect of cyclic wetting and drying on the swelling behavior of soils [35, 39–47]. In addition, Nordquist and Bauman [48], Obermeier [49], and Popescu [50] emphasized that the swelling ability increases with the number of drying and wetting repeats. Some researchers [42, 51, 52] stated that the swelling potential will decrease if soils are repeatedly subjected to swelling and then allowed to dry (partial shrinkage) to the initial water content [40]. Osipov [39] showed that the potential for swelling increases after the first cycle when the stabilized swelling soil is allowed to dry completely to or below the shrinkage limit (full shrinkage).

On the other hand, the cycling effect on the swelling potential of lime-stabilized soils has been reported to increase the swelling potential when the lime-stabilized soil is subjected to a wetting and drying cycle. Basma et al., Chaney et al., and Alonso et al. [42–44] stated that the drying method has a significant effect on the swelling properties of the swelling soil. Chen et al. [53] stated that swelling pressure is exponentially dependent on dry density but independent of the initial water content of the clay. He found that the swelling capacity is mainly affected by the vertical load at which saturation occurs and it increases with the initial dry density but decreases as the initial water content increases. Besides the water content, the chemistry of the pore water is important in the swelling behavior of soils. In general, as the salinity of the pore water increases, the swelling pressure of bentonite decreases. Also, Di Maio et al. [54] and Castellanos et al. [55] stated that chemical conditions and initial stress state play a very important role in the swelling behavior of compressed bentonite.

In this study, the effect of water content on the swelling property of soils was assessed. In the literature, the effect of wetting conditions of compacted bentonite and bentonite-aggregate mixtures on swelling pressure has been investigated by various researchers [45–47]. However, as in this study, no discussion was made regarding the optimum water content based on wet conditions. For this purpose, clay which has a high swelling capacity, the industrial name of which is bentonite, was used. Within the scope of the study, the Atterberg limit values of bentonite clay were first determined, and then the compaction test was performed to determine the optimum water content. Bentonite clay with optimum water content was prepared at different water contents determined below and above the optimum and the effect of initial water content on swelling pressure was investigated by measuring swelling pressures for 10 days for each sample. In the next step of this study, for comparison, another type of bentonite was stabilized using base oxygen furnace slag (BOFS) and swelling pressure tests were performed on the samples after up to 90 days of curing times, and finally, the results were compared with the previous findings.

2. Materials and Methods

Bentonite is part of the montmorillonite family and is a clay mineral with a liquid limit value of 500% or higher. They are formed as a result of chemical decomposition or degradation of volcanic ash, tuff, and lava rich in aluminum and magnesium content. In commercial terms, any clay with advanced liquid absorbent and colloidal properties is called bentonite [56]. Bentonites swell more or less when they come in contact with water. Bentonites can be classified into three groups according to their sodium-calcium ions, as they are divided into over, medium, and low swelling bentonites according to their swelling ability. The geological features of these bentonites differ in their formation. Among these, sodium bentonite is commercially important. However, sodium bentonite has little reserves in nature. Therefore, calcium and sodium-calcium bentonites that do not show much swelling feature are converted to sodium bentonite by various chemical methods [57].

The bentonite samples used in the experiments were obtained from KarBen Inc in Tokat (Turkey) (B1) and Naeen city in Isfahan (Iran) (B2). These natural bentonites were used in the experiments by sieving under the 40 sieves to release the lumps before starting the experiments. The physical and chemical properties of bentonite soils used in the study are listed in Table 1. The consistency limits of raw bentonites (B1 and B2) were determined as liquid limit (312 and 350.1%) and plastic limit (67 and 38.6%) according to ASTM D4318 (Table 1, Figure 1(a)). To determine the optimum water content, with the compaction tests as per ASTM D698, the optimum water contents of the clay samples for B1 and B2 were found to be 43% and 45.5%, respectively (Figures 1(b) and 2). In this study, basic oxygen furnace slag (BOFS) as a stabilizer was prepared by Iran Ferroalloys Industries Co. to enhance the swelling behavior of bentonite (B2).

After determining the optimum water content, in the case of B1, the soil samples were prepared from 0% to 100% water content (with an interval of 10%), and the swelling pressure of the samples was measured according to ASTM D4546 for 10 days with the swelling pressure test setup consisting of devices S type load cell and oedometer cell (Figure 3). This 4-channel data collection unit consists of a computer, and it can measure instantaneous swelling pressure with the software. The pressure that prevents the volume change that will occur as a result of the increase in the water content of swollen clay soil is called swelling pressure. Within the scope of this study, the pressure reached when the swelling did not occur with the device detailed above was found. In the case of B2, the soil was mixed with different amounts of BOFS (0, 2.5, 5, 10, 15, 20, and 30%) and tested after curing times of 1, 3, 7, 28, 45, and 90 days at a temperature of 25°C and with a relative humidity of 85%. In this study, the range of 0 to 30% was considered for BOFS in line with previous studies [58, 59] because this amount of BOFS can effectively change the engineering characteristics of the soil, thereby it is an acceptable range. The results of the XRF analysis and also a view of the BOFS used in this study are presented in Table 1 and Figure 4, respectively. It is worth noting that all tests were repeated twice and average values were measured to minimize changes.

3. Results and Discussion

3.1. Effect of Initial Water Contents on the Swelling Potential. Figure 5 shows that the swelling pressures of bentonite clay with 0% and 10% water content increase with time. In both conditions, the values showed a continuous increase, and it was seen that the maximum swelling pressure value was obtained within 10 days. It can be stated that the swelling pressure curve of bentonite clay with 10% water content has a sharper rise compared to that of 0% water content. In general, percentage changes depending on the time between two water contents are given in Table 2. It is seen that the swelling value of 10% water content at the last moment shows an increase of 57% compared to 0% water content.

The maximum swelling pressure value of bentonite clay kept at 20% water content for 10 days is higher than that of one kept at 10% water content. Based on this, it can be stated that the swelling in 20% water content is more than 0% water content. While the curve for 10% water content shows a steady increase, the curve at 20% water content was fixed after the 10000th minute as shown in Figure 6. As can be seen in Table 2, the percentage changes increased to about 50% by 8400 minutes, after which the rate decreased until the difference finally reached 16%.

Considering the swelling pressures of bentonite with 20% and 30% water content, it can be seen that both of them increase up to the 10000th minute depending on the time, and stabilize after this minute. It can be said that the swelling pressure for a 30% water content value is more than 20% water content. The maximum swelling pressure values for both occurred at the end of the experiment as shown in Figure 7. As can be seen in Table 2, the percentage change in the two water contents made a rapid decrease up to 37% until the 8400th minute, and then it was fixed at 23% as very little changes.

The optimum water content of bentonite clay used in the experiment is 40%, and the swelling pressure in this water content has the highest value when compared to the swelling pressure in other water contents. Swelling pressure values at 30% water content are close to the values at optimum water content. Considering Figure 8 and Table 2, it is seen that the curves approach each other, and the percentage change decreases with time.

As can be seen in Figure 9, the swelling pressure decreases in the water contents higher than the optimum value. At the beginning of the experiment, the swelling in clay with 50% water content shows a decrease of over 100% compared to 40% water content (Table 2). The maximum swelling

Table 1: Physical and chemical properties of raw bentonite.

Definition	B1	B2	BOFS
>75 μm	2.5% (by weight)	0% (by weight)	0% (by weight)
E (Methylene blue concentration (0.01 N))	310 ml	—	—
Montmorillonite content	75%	77%	—
SiO_2	61.28%	70.4%	15.6%
Al_2O_3	17.79%	12.1%	8.2%
Fe2O_3	3.01%	1.6%	20.9%
CaO	4.54%	2.2%	48%
MgO	2.10%	2.1%	3.4%
K_2O	1.24%	1.1%	<0.1%
Na_2O	2.70%	0.6%	0.2%
Liquid limit	312%	350.1%	—
Plastic limit	67%	38.6%	—

(a) (b)

Figure 1: (a) Liquid limit test. (b) Compaction test.

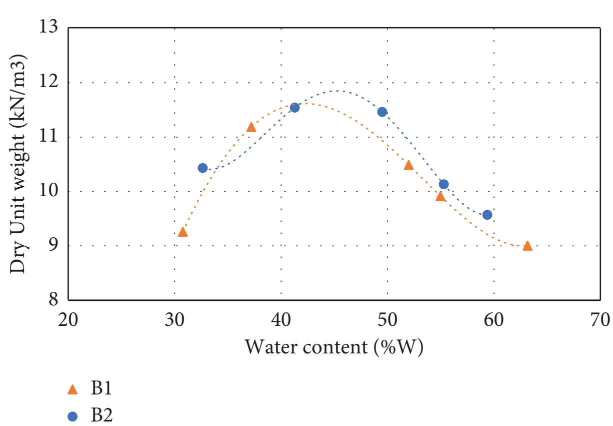

Figure 2: Compaction test results.

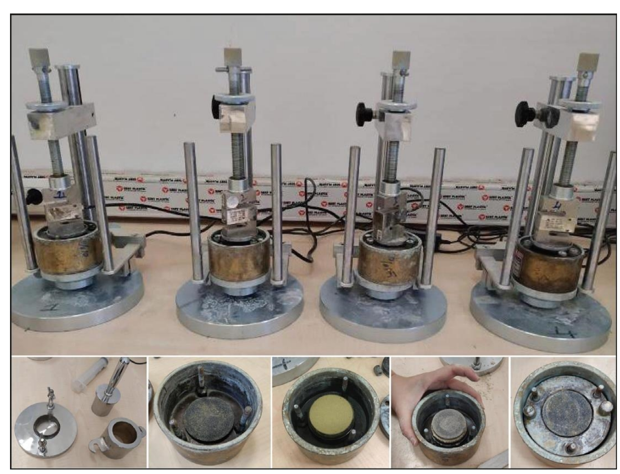

Figure 3: Stages of the swelling pressure test.

pressure value was obtained on the last day of the experiment (i.e., after the 10000th minute).

Comparing the swelling pressure values of bentonite clay with 50% and 60% water content, the swelling value at 60% water content is higher until the average 7000th minute, and then, the swelling value at 50% water content begins to increase compared to 60% water content. While the swelling pressure value of the clay with 60% water content shows steady progress after the 7000th minute, a constantly increasing curve is observed at 50% water content (Figure 10). It can be stated that the maximum swelling pressure values occur at the end of the 10th day.

According to Figure 11, similar results were obtained for the swelling behavior of bentonite clay with 60% and 70% water content. It can be seen that the curves continuously increase up to the 6000th minute, and the changes after this period progress at a minimum and reach the maximum inflation pressure value. The change in percentage after the 3600th minute was fixed by the remaining 24% as shown in Table 2.

Figure 4: A view of BOFS used in this study.

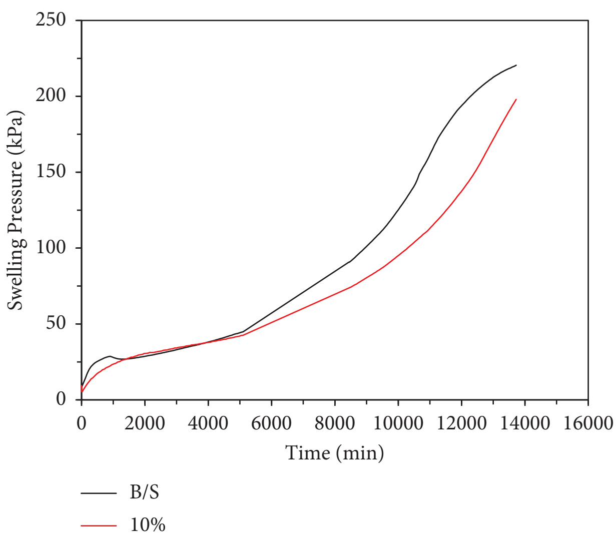

Figure 5: Comparison of swelling pressures of bentonite with 0% and 10% water content.

Considering Figure 12, it was observed that the swelling pressure of bentonite clay with 70% and 80% water content had similar changes to the swelling pressure of 60% and 70%. Generally, it can be stated that bentonite clay exhibits similar behavior in water contents greater than its optimum water content. It can be seen that the swelling pressure increased rapidly until the 6000th minute and then reached a constant value. Based on Table 2, the swelling pressure is 30% less than the 80% water content compared to the 70% water content, but this percentage change is fixed by coming to 11% depending on the time. When the previous conditions are compared with the current conditions, it is seen that the percentage change in 60% and 70% water content remained constant at 24% after the 4800th minute, and the percentage change in 70% and 80% water content continued for a while and stabilized at 11%. These indicate the importance of the percentage change in each water content in the studied soil.

Based on Figure 13, it is possible to say that after 80% water content, the soils can no longer show more swelling behavior and reach the highest swelling pressure they can show. In this context, it can be seen that the swelling pressure of the sample with 90% water content compared with samples prepared under different water content conditions reached the maximum value in a shorter time. While the clay with 80% water content reached a constant value after the 6000th minute, the swelling pressure of bentonite clay with 90% water content was fixed after the 4000th minute. As per Table 2, the percentage difference has reached a very high value, such as 41%, since the swelling pressure value is less at 90% water content.

In Figure 14, when the bentonite with 100% water content is compared with the bentonite with 90% water content, it was seen that they have similar changes and their swelling pressure values are one of the lowest values. As shown in Table 2, an average change of 20% was observed up to the 6000th minute and after that, it remained at around 13%.

Within the scope of this research, the swelling behaviors of bentonite clay depending on each 10% increase in water content were examined separately, and the swelling pressure changes for each condition were considered as a whole in the graphic given in Figure 15. Based on this, it can be stated that the most swelling pressure is at 40% water content, followed by 30% and 50% water content, respectively. It is observed that there is a continuous decrease in swelling values from 60% water content to 100% water content due to the increase in water content. The lowest swelling pressure values are found at 0% and 10% water content. In all cases, it is found that there is not much change in values after the 10000th minute.

3.2. Comparison of the Effect of Initial Water Content and BOFS Stabilization on Swelling Potential. Figure 16 shows the results obtained from the swelling pressure test for two expansive soils under different initial water content as well as stabilized with different amounts of BOFS after different curing times. For a better comparison between the results, by defining the $\Delta P/P_0$ as a dimensionless parameter, a reasonable comparison was made in which the ΔP is equal to the difference in swelling pressure in each sample with that of the bentonite soil sample (P_0). As shown in Figure 16(a), the $\Delta P/P_0$ for bentonite soil under different water contents initially had an upward trend up to its optimum water content range (~42%-see Figure 2) and then decreased with increasing water content. It should be noted that positive values for the $\Delta P/P_0$ parameter mean that the swelling pressure is higher than the bentonite soil sample and negative values for $\Delta P/P_0$ indicates lower swelling pressure than the bentonite soil sample so that if in a sample the pressure is equal to −1, it indicates complete control of the swelling potential (swelling potential equal to zero).

As previous studies have reported [60], the engineering parameters of compacted clay soils on the dry and wet sides are significantly different from each other. Clay soils compacted on the dry side have a random fabric, while compaction on the wet side of optimum moisture content (OMC) leads to more particle orientation, resulting in more thoroughly developed double-layer water films.

In general, at a certain amount of energy for the compaction process, on the dry side, the specimens are flocculated and large voids are formed due to their random

Table 2: Percentage change of swelling pressures of bentonite with water content.

Water content/time (min) (%)	0 (%)	1200 (%)	2400 (%)	3600 (%)	4800 (%)	6000 (%)	7200 (%)	8400 (%)	9600 (%)	10800 (%)	12000 (%)	13200 (%)
0–10	−25	10	16	19	22	2	31	34	41	45	49	57
10–20	−50	−15	−11	5	26	43	49	57	55	47	36	16
20–30	66	71	75	73	69	60	51	37	26	24	23	23
30–40	40	55	51	46	33	19	13	8	5	4	3	3
40–50	−180	−204	−142	−103	−71	−49	−37	−23	−17	−13	−11	−10
50–60	12	44	34	23	16	10	1	−12	−16	−21	−23	−25
60–70	−18	−46	−38	−29	−25	−24	−23	−23	−23	−24	−24	−24
70–80	5	−35	−29	−27	−22	−13	−12	−12	−11	−11	−11	−11
80–90	−8	12	4	−6	−17	−34	−37	−40	−40	−41	−41	−41
90–100	−10	−27	−28	−24	−23	−20	−19	−12	−12	−13	−13	−15

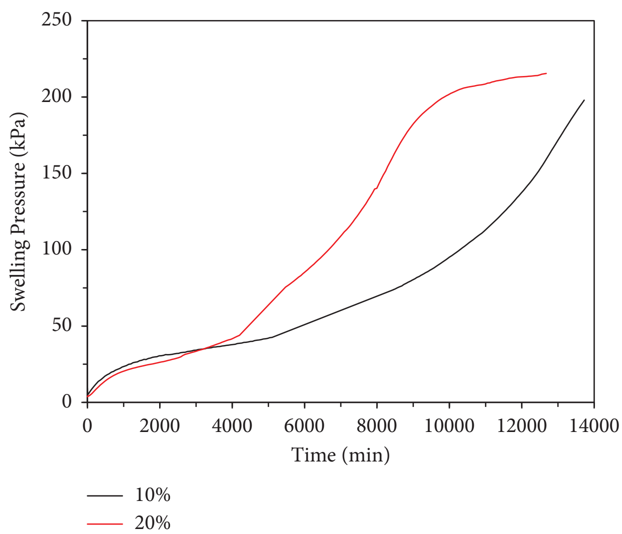

Figure 6: Comparison of swelling pressures of bentonite with 10% and 20% water content.

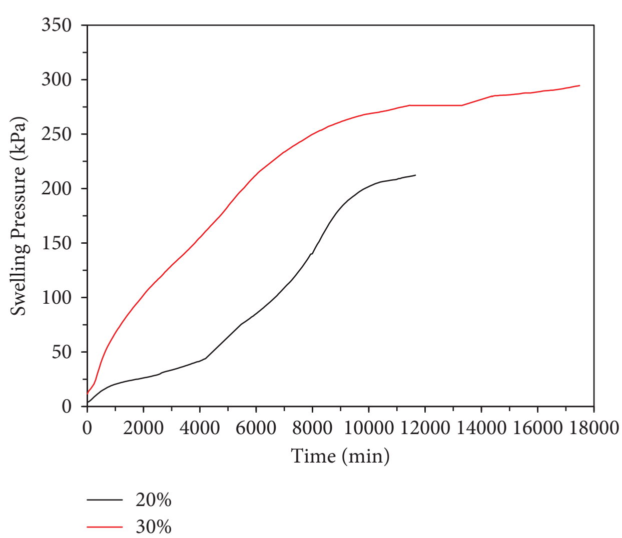

Figure 7: Comparison of swelling pressures of bentonite with 20% and 30% water content.

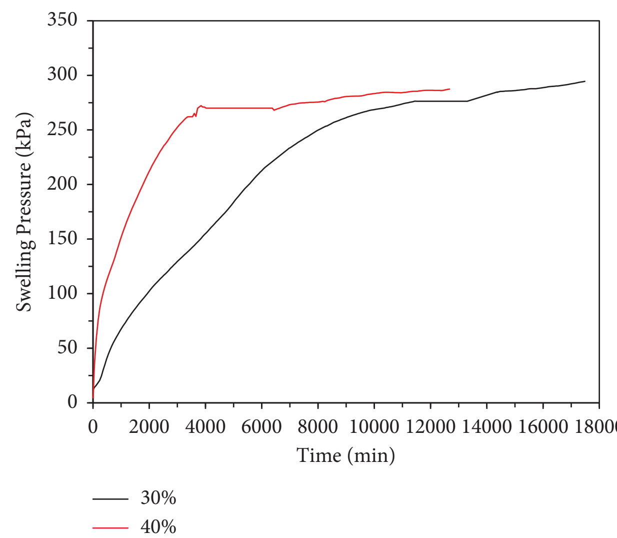

Figure 8: Comparison of swelling pressures of bentonite with 30% and 40% water content.

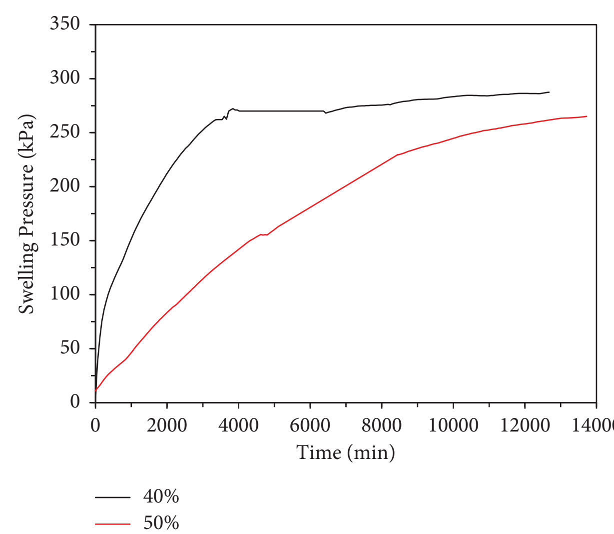

Figure 9: Comparison of swelling pressures of bentonite with 40% and 50% water content.

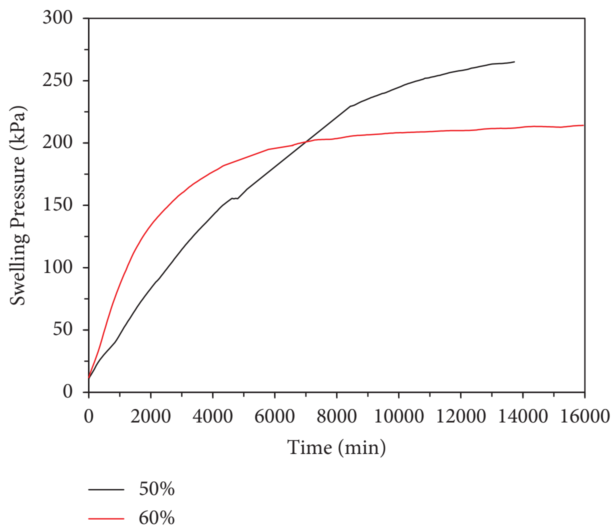

FIGURE 10: Comparison of swelling pressures of bentonite with 50% and 60% water content.

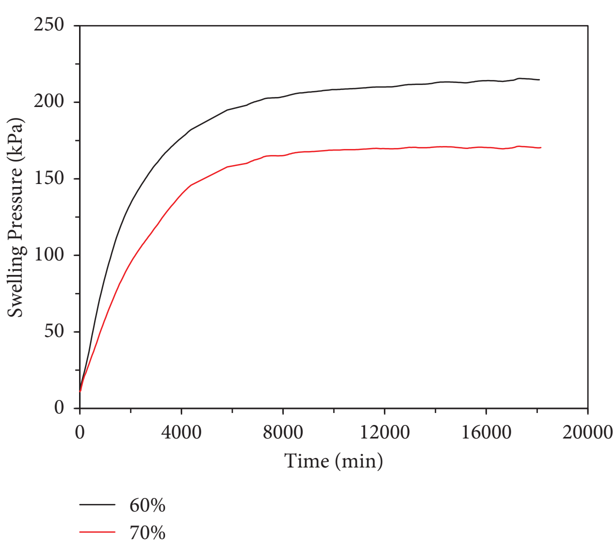

FIGURE 11: Comparison of swelling pressures of bentonite with 60% and 70% water content.

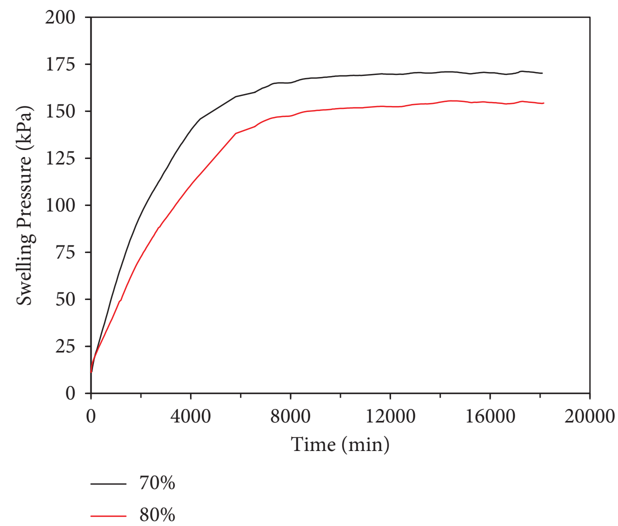

FIGURE 12: Comparison of swelling pressures of bentonite with 70% and 80% water content.

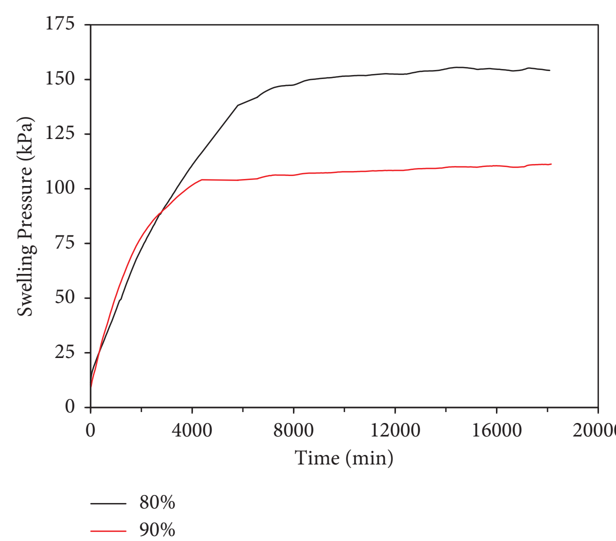

FIGURE 13: Comparison of swelling pressures of bentonite with 80% and 90% water content.

orientations, which usually have edge-to-edge or edge-to-face contacts. However, on the wet side of OMC, the orientation of soil particles is much higher than that of the dry side, and therefore, the quantity of face-to-face contact also increases. As shown in Figure 16(a), the $\Delta P/P_0$ value initially increased with increasing water content up to the optimum content range which is in good agreement with previous studies [61]. According to the earlier explanations, the reason for this can be attributed to the state of particle flocculation with the edge-to-face contact and large voids between particles. In this case, more water content can be placed between the particles. However, by adding the initial water content higher than the optimum range, the $\Delta P/P_0$ rate and swelling pressure have been reduced due to the different structures of clay particles, their oriented patterns, and face-to-face contact. It should be noted that under the specified dry density, samples with a lower initial water content have larger macro voids and therefore the interior space is largely sufficient to allow the soil to swell. As the initial water content increases, the amount of these voids decreases, and as a result, the swelling pressure increases due to the limited space (between the soil particles) to swell during the wetting procedure. According to Figure 16(a), with the addition of the initial water content, there was a threshold value for which the $\Delta P/P_0$ began to decrease. The reason for such a decrease in swelling pressure, as well as $\Delta P/P_0$, is that soil swelling with higher initial water content increases during the sampling process, and therefore, the $\Delta P/P_0$ decreases during the test. Hence, the maximum swelling pressure was obtained with the optimum moisture content range under the same dry density of samples.

Figure 16(b) shows the changes in the dimensionless $\Delta P/P_0$ parameter for different amounts of BOFS at up to 90 days of curing times. As can be seen, the $\Delta P/P_0$ decreased with

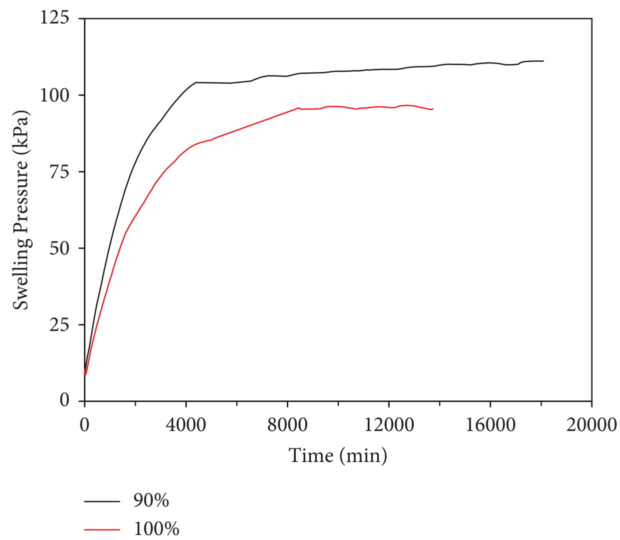

Figure 14: Comparison of swelling pressures of bentonite with 90% and 100% water content.

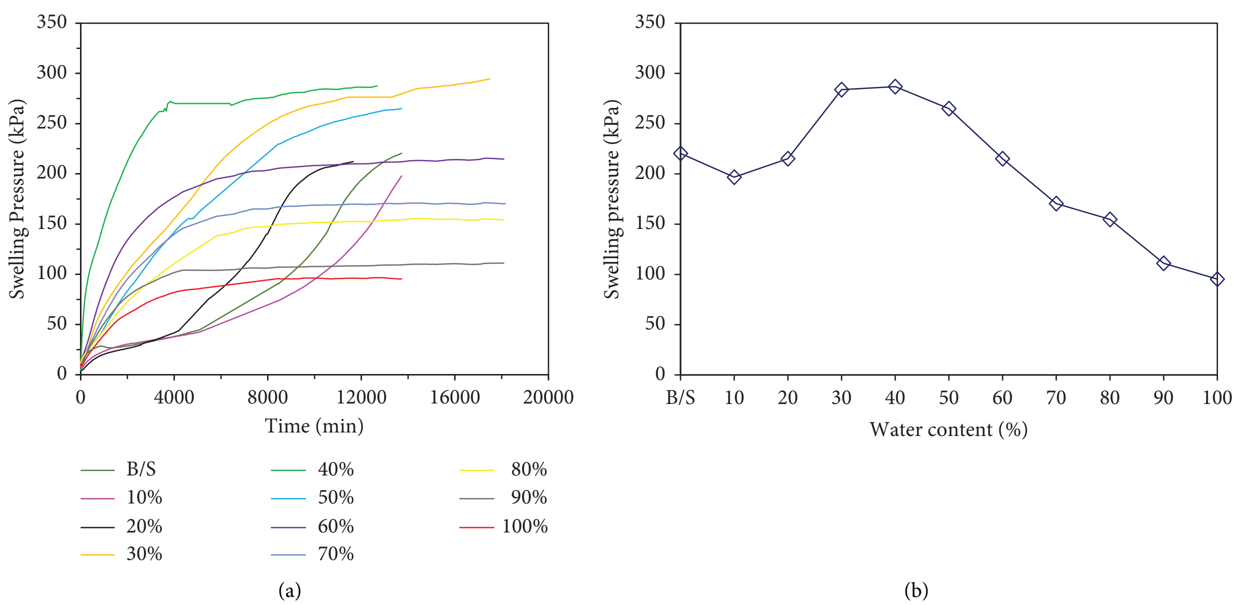

Figure 15: Swelling pressures of bentonite for all water contents: (a) in detail and (b) final swelling pressure.

increasing BOFS, which can be attributed to the positive effect of chemical additives in the stabilization of bentonite (B1). So, the reason for the decrease in $\Delta P/P_0$ is the short-term and long-term reactions between BOFS and soil particles. Comparing these two diagrams, it can be seen that 2.5 and 5% BOFS after 90 and 45 days of curing time, respectively, can greatly reduce the ΔP/P0, which is much lower than that of the initial moisture content of 100%. This indicates that the use of such an additive, albeit small, at the proper curing time can have a significant effect on the swelling pressure, even more than changing the initial water content. Therefore, in projects where complete control of

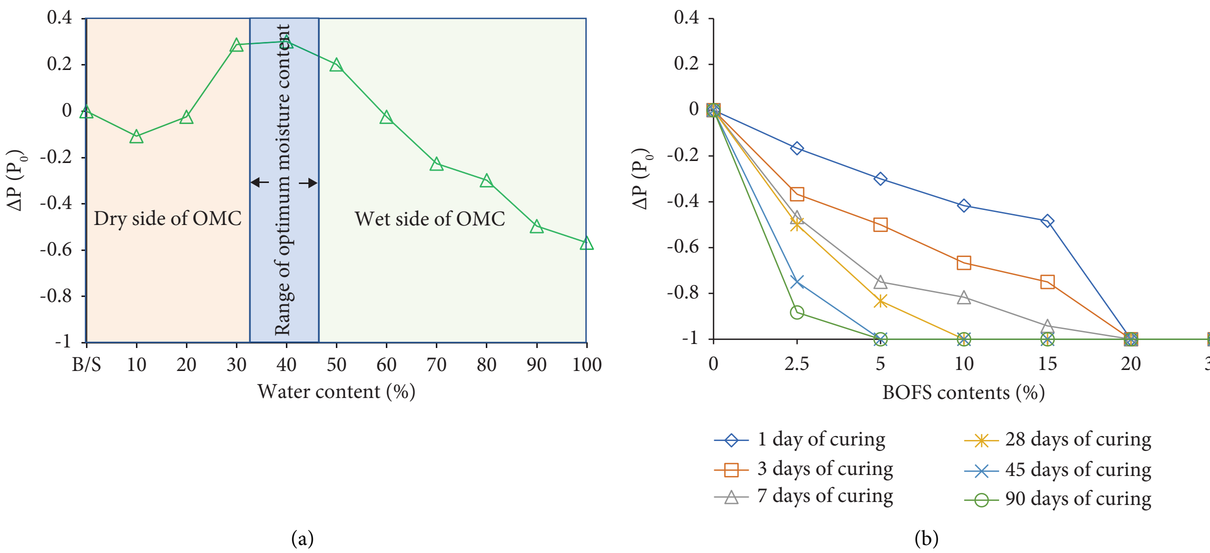

Figure 16: Swelling pressures of bentonite samples for different (a) initial water contents and (b) BOFS contents.

swelling pressure is considered, the use of additives such as BOFS can be very useful because it is not possible to achieve this goal by changing the initial moisture content.

4. Conclusions

Within the scope of the study, swelling pressure changes of bentonite clay depending on the changes in water content were investigated. Unlike the studies in the literature, it has been experimentally revealed how the swelling pressure will change for each 10% increase in water content. If a general evaluation is made on the swelling pressure graphs of bentonite clay prepared in different water contents, it can be summarized that the data obtained support the previous studies in the literature up to the optimum water content, but there is a relative decrease in swelling pressures after optimum water content.

Experimental studies have examined the change in swelling pressure of bentonite clay at different rates depending on each 10% increase in water content. Based on this, it can be stated that the most change occurs between 40% and 50% water content. The biggest change under optimum water content was observed with an average 75% increase in the transition from 20% to 30%. Above optimum, the maximum swelling pressure change occurred with a 40% increase between 80% and 90% water content. It was found that the swelling pressure values of the bentonite samples generally have been stabilized from the 5th and 6th days. Based on this data, it can be noted that bentonite shows the swelling property for a certain period, and then the swelling feature stops.

With this study, the swelling potential of the swelling clays in the soil improvements to be made in the geotechnical field was examined, depending on the water content, and it was evaluated that the results obtained would be effective in reducing swelling damages by reflecting on the application. However, the use of BOFS in soil stabilization significantly improved the swelling pressure of bentonites. Comparing these two techniques showed that despite the positive effect of initial water content on soil properties, it is impossible to fully control the swelling pressure. Therefore, in order to achieve the complete elimination of swelling and the resulting swelling pressure, it is necessary to include chemical stabilization methods in the projects.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

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Research Article

Effectiveness of Liquid Antistripping Additive for Emulsion-Treated Base Layer Using Reclaimed Asphalt Pavement Material

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In the new global economy, getting natural aggregates (NA) has become a central issue for constructing flexible pavements due to the scarcity of aggregates and the ban on mining in various states in India. This research is an attempt to achieve sustainability by using a liquid antistripping additive for emulsion-treated base layer to improve the performance of Reclaimed Asphalt Pavement Material (RAPM) inclusive aggregates. RAPM was evaluated, with inclusion percentages of 50 and 70 percent, whereas, the control mix was prepared using 100 percent natural aggregate (NA). The effect of inclusion of liquid antistripping additive (ASA) with different RAPM percentages on various properties of ETB mixes, such as maximum dry density, indirect tensile strength, moisture resistance and resilient modulus, was studied. Furthermore, when compared to RAP-ETB mixes without ASA, RAP-ETB mixes with ASA were found to preserve many of their qualities. The present study aimed to propose the laboratory design of optimum bitumen emulsion content (OBEC) for ETB in a simpler manner. For 50 RAP, obtained OBEC was at 4.4%, whereas for 70 RAP, OBEC was obtained at 3.8%. However, for 100 % NA, calculated OBEC was 7.0% as there was 0% RAP in it, hence binder absorption was more. The strength parameter was assessed using the Indirect Tensile Strength (ITS) test. At the same time, the pavement response was measured in terms of Resilient Modulus (MR). MR of 70 RAP mixes was higher than that of 50 RAP mixes, and 100 NA mixes with antistripping additive. Test results were encouraging, and significant improvement in strength was caused by cement filler and antistripping additive.

1. Introduction

The increasing cost of binder and environmental concerns attract Government agencies and the construction industry to use other alternatives for pavement construction [1–6]. Moreover, getting a natural aggregate is becoming more challenging due to the ban and restriction in mining. This is equally responsible for shifting to other alternatives of construction [4, 7–9]. To overcome this issue, the use of reclaimed asphalt pavement (RAP) seems a promising alternative. When the mixture of bitumen and aggregate is removed from the flexible pavement by controlled milling up to the desired depth, the collected material is termed RAP material. Moreover, utilization of RAP for the construction of emulsion-treated base (ETB) by cold in-place recycling (CIPR) not only reduces the cost of virgin aggregates but also provides a stiffer base course and reduces the problem of stockpiling of RAP material due to which its properties get degraded [1, 10–12]. According to Federal Highway Administration (FHWA) reports, up to 33 million metric tons (36 million tons) of excess asphalt concrete is currently being used as a portion of recycled hot mix asphalt, in cold mixes, or as aggregate in granular or stabilized base materials, accounting for 80 to 85 percent of all asphalt concrete currently produced. The quantity of surplus asphalt concrete that needs to be disposed of is expected to be less than 20% of the total amount of RAP produced each year.

Extensive research has shown that [13–19] fumes from hot bituminous mixes are a health concern for construction workers, which needs to be eliminated to provide a safe and healthy environment for construction. In the current study, the mix design of ETB was done using cold mix technology (CMT) over conventional hot mix technology (HMT) as it has various environmental merits and lower production plant emissions [5, 13–20]. In HMT, all the mix ingredients such as bitumen, coarse aggregate, fine aggregate, and filler (if any) are heated. However, in the case of CMT, no heating is required to make the mix. Moreover, there is no need to stockpile the material in advance as the whole recycling process can be carried out using the cold in-place recycling (CIPR) technique. There are inherent benefits to this technology; from reducing greenhouse gas emissions to lesser number of truck trips to the construction site as the requirement of virgin aggregates is significantly less as compared to the conventional construction technologies, and one can achieve an overall economy in the project due to lesser fuel consumption during construction [4, 5, 16–19, 21, 22].

Fluid plays a vital role during the mix design of ETB. If the material is dry, then emulsion might break prematurely during mixing. Also, if there is too much fluid content, the mix will prematurely break due to the detachment of bitumen film from the aggregate; the phenomenon is known as stripping of aggregate. It is hypothesized that the presence of moisture may be one of the reasons for lower early strength in ETB. Therefore, in the present research work, to improve the resistance against moisture damage or to avoid stripping of aggregates in ETB, the thought of using liquid anti-stripping additive might reap advantages. Even in the presence of moisture, using a little amount of antistripping additive allows the binder to coat the aggregate properly, resulting in improved moisture resistance properties [13, 14, 23]. It has been previously observed that on adding antistripping additives in the mix, moisture resistance was increased [15].

This study set out to investigate the usefulness of using liquid antistripping additives for ETB using RAP for constructing a sustainable strong sub-base layer. Efforts had also been made to utilize the maximum percentage of RAP without compromising the performance of the mix to achieve economic and environmental benefits. Researchers have investigated various approaches to utilizing RAP in surface layers and binder layers [24–31]. However, the Indian Roads Congress (IRC) recommends only up to 30 percent RAP to achieve the desired strength characteristics without compromising its performance [16]. Moreover, the effect of using more percentage of RAP material with antistripping additive and cement had not been studied and reported in detail for a base layer. Therefore, the current paper explores how the maximum amount of the RAP material could be utilized by identifying the optimum dosage of bitumen emulsion and antistripping additive. 100% Natural Aggregate (NA), 50% RAP, and 70% RAP were used for the design of the ETB mix in the current study. The amount of RAP percentage was decided to target the mid-value gradation requirements and to satisfy the gradation limits. The optimum bitumen emulsion content (OBEC) was based on the indirect tensile strength (ITS) test, where the peak load was calculated for both dry and wet Marshall specimens. The mix were evaluated for estimating the cracking resistance and extent of moisture damage in the mix. Moreover, the present study also investigates the effectiveness of the use of liquid antistripping additive with mix for ETB layer. The paper also ascertains the performance of liquid anti-stripping additive Levasil by studying and comparing its rheological properties with residual emulsion. For the design purpose of the current pavement section, guidelines for the design of flexible pavement were used [17].

2. Materials and Methods

2.1. Materials

2.1.1. Aggregate and Gradation. Natural aggregates (NAs) and RAP were used to cast and test emulsion-treated base layer mix in the laboratory. NAs were collected from the local stone quarry, whereas RAP was collected from NH-344 using a controlled milling technique. Residual asphalt content (RAC) in the milled material was extracted in accordance with ASTM D2172-11, Standard Test Methods for Quantitative Extraction of Asphalt Binder from Asphalt Mixtures [18]. It can be seen from the data of Table 1 that the RAP material reported significantly more water absorption value as the RAP was collected through milling, and material from layers beneath might have collected. Another reason for such high-water absorption may be the presence of dust particles as the RAP material was collected and stored in the open bins behind the Indian Institute of Technology (IIT) Roorkee laboratory for testing and research purposes. Other physical properties of NAs and RAP aggregates are shown in Table 1. Both fine and coarse RAP were used in the study after gradation analysis and blending. The blending proportions were determined to satisfy the specification in Table IX-1 of IRC: 37-2012 [19] to produce a mix for emulsion-treated base layer using RAP [19]. Since the RAP was recovered by milling technique, 100% RAP cannot be used as it did not satisfy the aggregate gradation requirement. Based on the blending exercise, the selected RAP percentages for the current study were selected as 50% RAP and 70% RAP (50 RAP, 70 RAP, and 100 NA, respectively) for the mix preparation, and the rest were NAs satisfying the mid-value aggregate gradation requirements as per the Indian Roads Congress (IRC) specifications. The adopted gradation curve is shown in Figure 1.

2.1.2. Bitumen Emulsion and Antistripping Additive. In this study, a cationic slow setting bitumen emulsion (SS_2) was collected from Total Bitumen, Jodhpur, India, which was used as a base binder. The specific reason for using slow setting bitumen emulsion as a base binder and not using rapid setting (RS) grade bitumen emulsion was the setting time of bitumen emulsion because RS sets faster compared to SS_2. As a result, the bitumen emulsion may be unable to

Table 1: Physical properties of NA and RAP.

Description	Standard specification	Specified limits	Test results	
			Natural aggregate	RAP aggregate
Combined FI & EI	IS:2386 part I	Max 35%	12%	7%
Aggregate impact value	IS:2386 part IV	Max 27%	15.82%	21%
Aggregate crushing value	IS:2386 part IV	Max 30%	13%	15.57%
Los Angles abrasion value	IS:2386 part IV	30	18%	24.59%
Specific gravity of coarse aggregate	ASTM C127	—	2.619	2.559
Water absorption of coarse aggregates	ASTM C127	—	0.629	1.54
Stripping test value	IS:6241	Min Retained coating 95%	98%	—
Plasticity index (%)	IS:2720 part V	Min 6	—	Non-plastic (N.P.)
Residual asphalt content (RAC) (%)	ASTM D2172	—	—	3.50%

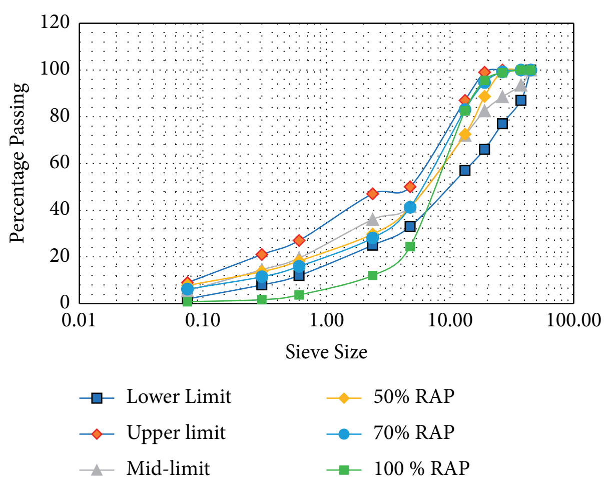


Figure 1: Adopted aggregate gradation after blending.

coat the larger particle in the case of RS due to its fast-setting tendency. However, SS_2 aims to coat the coarser aggregates and delay setting time, which helps in proper mixing and compaction of material by using the cold in-place recycling (CIPR) method for laying ETB. To study the rheological properties of bitumen emulsion, the residue was obtained using oven method as per IS 8887, *Indian Standard for Bitumen Emulsion for Roads (Cationic Type)–Specifications, Third Revision* [21]. The residual binder and liquid antistripping additive-modified binder was tested for rheological tests on residual binder which includes, penetration, ductility, residue by evaporation tests in accordance with ASTM D2397–20, *Standard Specification for Cationic Emulsified Asphalt* [32] and IS 8887, *Indian Standard for Bitumen Emulsion for Roads (Cationic Type)–Specifications, Third Revision* [21]. Apart from basic rheological tests, the residual binder was also tested for determining the softening point of the base binder and modified binder, which the authors believe is one of the important tests for predicting the quality control of the binder. The addition of a liquid antistripping additive improves the softening point of the base binder. The liquid antistripping additive Levasil was used and it was in the liquid form. It is believed that the small addition of this antistripping additive helped in enhanced aggregate coating and improved moisture resistance [24, 25, 33]. The properties of the base binder and antistripping additive are presented in Table 2.

2.1.3. Cement. Commercially available ordinary Portland cement, Grade 43 (OPC 43) conforming to IS: 8112, *Ordinary Portland Cement 43 Grade–Specification, Second Revision* [26], was used in this study as a filler substituting the stone dust filler. Cement was collected from a local vendor in Roorkee, India. Considering the literature [27], cement was used at a fixed dosage of 1 percent by weight of aggregate to improve the adhesion between binder and aggregate. An amount greater than 1 percent might make the ETB brittle, and it will act more as a cement-treated base (CTB) layer [27].

2.2. Testing Methods. As mentioned in the preceding paragraph, different fractions of RAP were used to determine the maximum amount of RAP for ETB mixtures, and the methodology used is shown in Figure 2. Based on many performance and durability criteria, the maximum amount of RAPM for cold ETB mixture was determined.

2.2.1. Maximum Dry Density and Optimum Fluid Content. To compact the RAP bitumen emulsion-treated mix to its maximum density, optimum fluid content (OFC) is required [19, 28]. OFC is the sum of water present in the emulsion, aggregate moisture content, and additional water added to the mix. Here, the moisture content of the aggregate is zero, as the aggregates were taken after being oven-dried in a hot air oven at least for 24 hours before making the mix to remove the excess moisture content from the RAP material. However, this OFC can be increased or decreased at the site depending on the weather conditions. In the case of ETB, two fluids are present; one is the fluid that is added while preparing the bitumen emulsion, the other is the fluid that is used as water content for compaction of the mix. Higher water content may result in deformation of the surface during compaction [27]. Hence, optimum fluid content has to be determined in the laboratory for a proper mix design procedure. In the current study, the maximum dry density (MDD) and optimum fluid content for the mix were determined as per IS 2720 (Part VIII), *Determination of Water content–Dry Density Relation Using Heavy Compaction* [29]. The mix was prepared using a standard proctor test employing 150 mm diameter mold, of volume 2250 cm^3; the material was compacted into five layers, giving 55 blows to each layer using a rammer of weight 4.89 kg having a free-

Table 2: Properties of residual emulsion and modified residual emulsion.

Test parameters	Standard specification	Emulsion (E)	$E + 0.3\%$ additive	$E + 0.4\%$ additive	$E + 0.5\%$ additive
Residue by evaporation (%)	ASTM D7497	62.08	—	—	—
Stability to mixing with cement (coagulation), %	ASTM D244	1.62	—	—	—
Specific gravity	ASTM D70	1.03	—	—	—
Penetration of residue at 25°C, 100 g, 5 s, 0.1 mm	ASTM D5	90	86	84	79
Softening point (ring and ball), °C	ASTM D36	44.1	45.1	45.7	46.4
Ductility of residue (cm)	ASTM D113	67	65	63	59

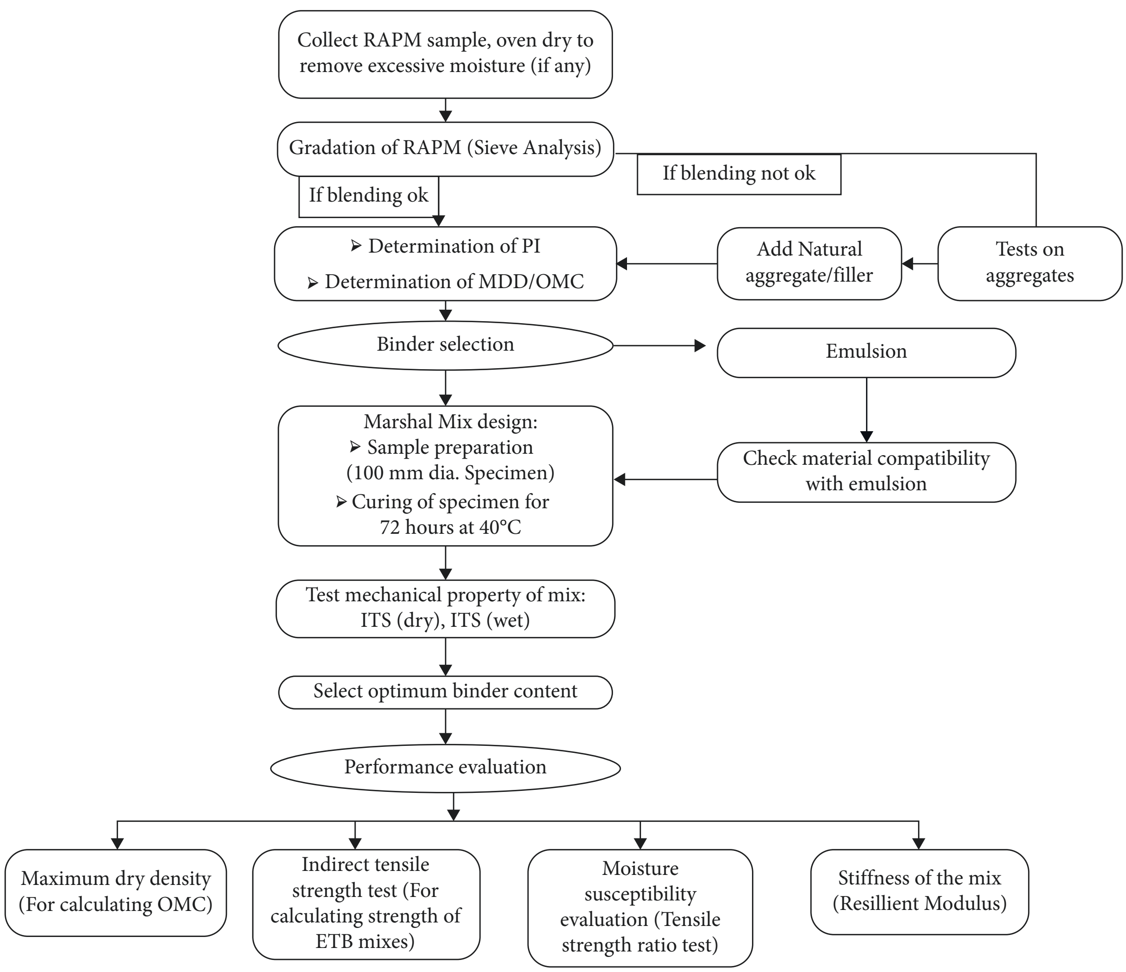


Figure 2: Work methodology for the experimental program.

falling drop of 450 mm height. In order to provide higher interlocking among the compacted layers and to minimize the cracks inside, each layer was scarified before adding the subsequent compacted layer [2, 30]. A blend of emulsion and water by volume was prepared in a 1 : 1 ratio; this blend is known as "total fluid." The amount of emulsion was kept constant, while fluid content was increased by 1% increment, and three samples were cast at each fluid content of 4, 5, 6, 7, and 8 (by weight of total mix). The mix was then transferred to a standard 100 mm diameter Marshall Mold and compacted at 75 blows on each side. To ensure homogenous mixing and uniform coating of the RAP and virgin aggregates, the mixes were prepared using a pugmill mixer for three minutes. The fluid content of the specimens was determined by drying the specimens in a hot air oven for 24 hours at a specified 100°C temperature, and the dry density of the specimens was calculated by equation [19].

$$D_{dd} = \frac{(D_{bulk})}{(1 + FC)}, \quad (1)$$

where D_{dd} = dry density in gm/cm^3, D_{bulk} = Bulk density in g/cm^3, FC = Fluid content by dry weight of aggregates in decimal

2.2.2. Indirect Tensile Strength. The optimum bitumen emulsion content (OBEC) for ETB was determined by using Marshall specimens. The indirect tensile strength test of ETB mixtures was performed in accordance with ASTM D 6931, *Standard Test Method for Indirect Tensile (IDT) Strength of Asphalt Mixtures* [34, 35]. Mix specimens were prepared using Marshall molds, targeting 100 mm diameter and 63 mm height by taking approximately 1200 grams weight of each sample. A total of six samples (three conditioned and three unconditioned) were casted with the help of a pugmill mixture at each emulsion content, as shown in Figure 3(a), for three RAP contents (50 RAP, 70 RAP, and 100 NA) compacted with 75 blows on each side were casted starting from 3.2% to 4.7% bitumen emulsion with an increment of 0.3% percent by weight of the total mix. A total of 108 samples [(6 $ITS_{Dry+Wet}$) ∗ (6 Emulsion contents) ∗ (3 RAP contents) = 108)] were cast for finding OBEC and 54 samples [(6 $ITS_{Dry+Wet}$) ∗ (3 Anti-stripping additive content) ∗ (3 RAP contents) = 54)] for determining the effective liquid antistripping additive dosage. The specimens were cured at room temperature for 24 hours in molds and then extracted to cure it further cured for 72 hours at 40°C in a hot air oven (Figure 3(d)), as the sample contains compaction water and water in emulsion [16].

Marshall loading frame was used, which applied a load of 50.8 mm per minute to perform the Indirect Tensile Strength Test at 25°C, an illustration of the test assembly is shown in Figures 3(b) and 3(c) [34]. Three samples were tested for ITS_{dry} at 25°C, and the rest three samples were kept in a water bath for the next 24 hours and then tested for ITS_{wet} at 25°C (Figure 3(e)). ITS value of each briquette mold was calculated using the below-mentioned equation.

$$ITS = \frac{2 * P}{\pi * d * h} * 1000, \tag{2}$$

where ITS = Indirect tensile strength, kPa. P = maximum load, N, d = diameter, mm, h = height, mm.

2.2.3. Moisture Susceptibility Evaluation. The ratio of ITS_{wet} and ITS_{dry} is termed as Tensile Strength Ratio (TSR), expressed as a percentage, which is the measure of evaluation of moisture susceptibility [14, 36, 37]. Also, for Bitumen stabilized materials (BSM), if ITS_{dry}>400 kPa and TSR <50%, it indicates the requirement of active filler as the material confirms the presence of a clayey particle in it [1]. TSR is an important parameter to evaluate the stripping or the detachment of bond between the aggregate and bitumen emulsion during their service life; it should be greater than 80. Six samples were prepared at each bitumen emulsion content, and the samples were kept in a hot air oven at a controlled temperature of 40°C for 72 hours. During the process, the samples reached a constant mass, as the moisture inside the samples dried out. Out of these six samples, three samples were tested for ITS_{dry} and the rest three samples were tested for ITS_{wet} by further keeping the samples for next 24 hours in water bath at 25°C. The TSR of samples can be calculated using equation.

$$TSR = \frac{Avg\,.\,Wet\,Tensile\,Strength}{Avg\,.\,Dry\,Tensile\,Strength} * 100. \tag{3}$$

2.2.4. Resilient Modulus (MR). Generally, to evaluate the quality of materials, the resilient modulus value can be used. The elastic modulus based on the recoverable strain under repeated loads is called the Resilient Modulus (MR). Resilient modulus correlates stress-strain for rapidly applied load, with a loading duration of 0.1 sec and rest of 0.9 sec. A cylindrical specimen of diameter 100 mm of ETB mix is loaded vertically. MR is the measure of the stiffness of the mixture within its linear elastic region. MR is the nondestructive testing performed by using a Universal testing machine (UTM) as per ASTM D 4123 [38] as shown in Figure 4. The test was conducted at 35°C as per Indian conditions, as the average annual average temperature range is around 35°C. Before conducting the test, the test specimen was conditioned at least for 24 hours at the specified load temperature.

3. Results and Discussion

3.1. Maximum Dry Density and Optimum Fluid Content. Fluid plays a vital role during the mix design of ETB. If the material is dry, then emulsion might break prematurely during mixing. Therefore, attention must be given while mix the design process in the laboratory. Figure 5 shows the variation of maximum dry density (MDD) and fluid content (FC), from which the corresponding optimum fluid content (OFC) was calculated for different percentages of RAP mixes. OFC is required to compact the RAP bitumen emulsion-treated mix to its maximum density. Corresponding to MDD, OFC for 100 NA, 50 RAP, and 70 RAP was found to be 5.5%, 6.0%, and 7.0%, respectively. Moving on to MDD results, 100 NA mix had the highest MDD (2.280 g/cm^3) with lowest fluid content (5.5%). The addition of 50% RAP to the 100 NA mix increased the fluid content value and decreased the MDD value considerably. At the same time, 70 RAP mixes showed a further decrease of MDD by 3.07% and 1.81% compared to 100 NA mixes and 50 RAP mixes, respectively. Together, these results suggest that there is an association between MDD, OFC, and different percentages of RAP. It was found that the addition of RAP leads to the reduction of MDD and an increase in fluid content. The reduction in MDD and increase in fluid content can be due to the poor internal friction between the RAP material. Also, the decrease in MDD in RAP mixes was owing to the lower specific gravity of RAP aggregates as compared to natural aggregates. Although the presence of dust from the lower layer in the RAP mixes results in higher water absorption of RAP, aggregates may be held responsible for increased OFC in the RAP mixes [11, 39].

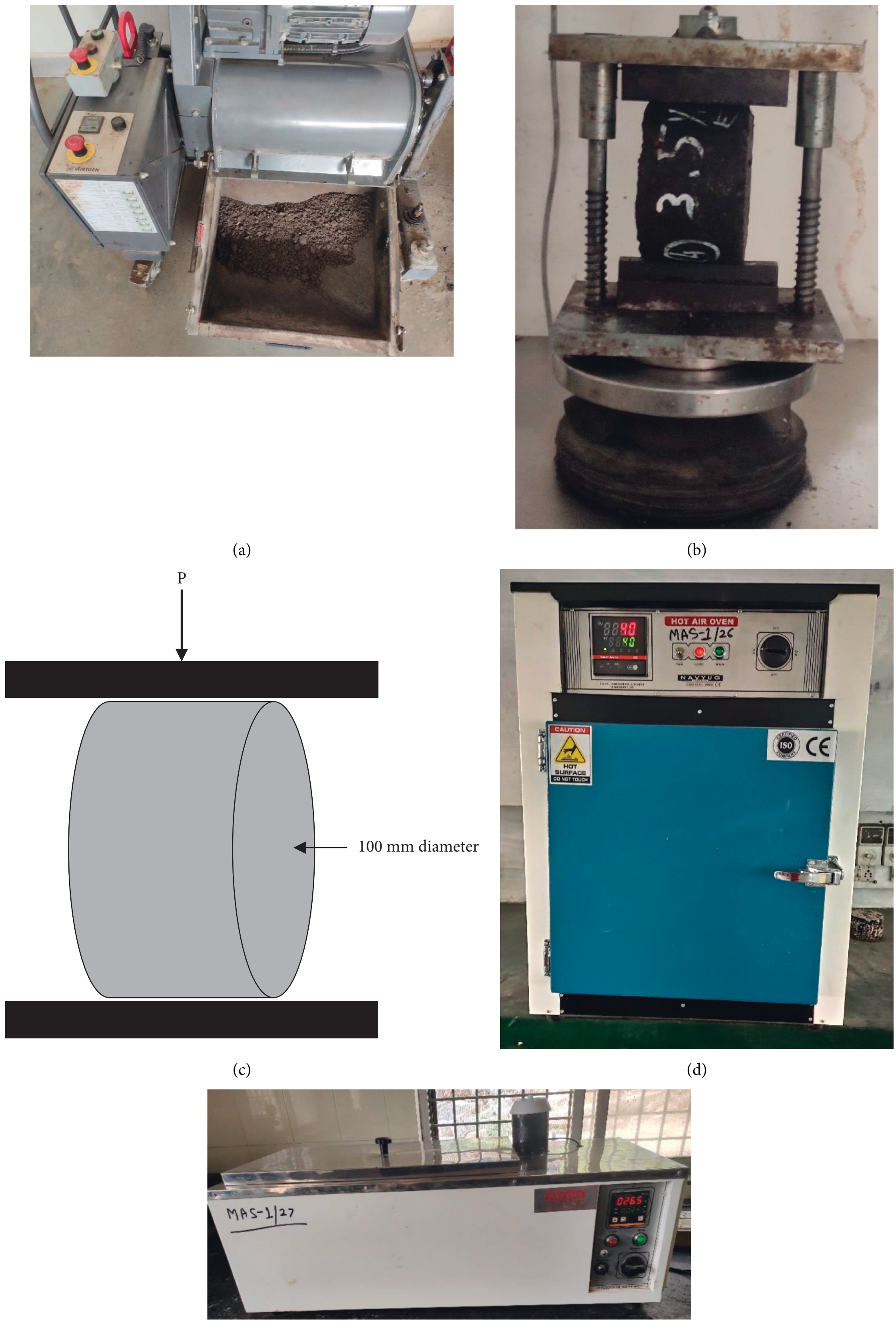


Figure 3: (a) Pugmill mixture (b) Indirect tensile strength test. (c) Loading assembly illustration. (d) Test specimens at 40°C for 72 hours in hot air oven. (e) Water bath for ITS_{wet}.

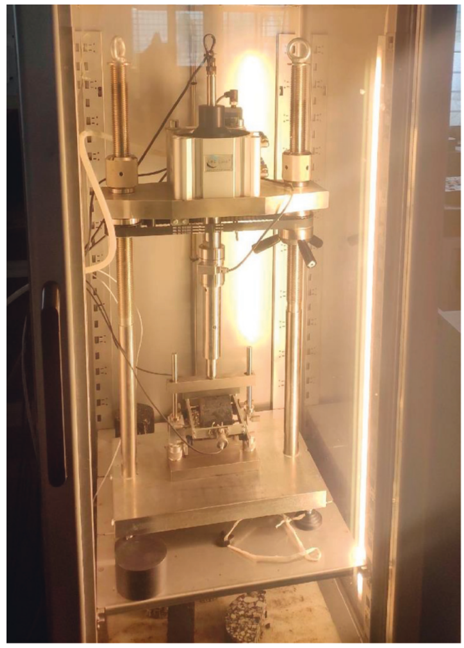


Figure 4: Resilient modulus test set-up.

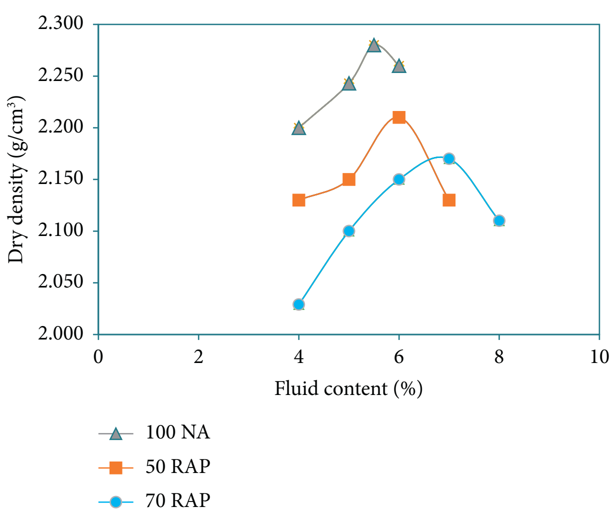


Figure 5: Relationship between fluid content and dry density.

3.2. Indirect Tensile Strength. Optimum bitumen emulsion content (OBEC) was calculated by Indirect Tensile Strength (ITS) of both conditioned and unconditioned samples before adding a liquid antistripping additive. A total of $6*6*3=108$ Marshall samples were cast and tested for finding OBEC. All these samples contain 1% cement as the use of cement provides better stiffness to the mix and helps in gaining early strength, whereas the purpose of the antistripping additive was to keep the bond of aggregate and bitumen emulsion safe from the OFC, which was added for the preparation of ETB mix. For 50 RAP, 4.40% was the OBEC, similarly for 70 RAP, OBEC was obtained at 3.80% whereas, for 100 NA, OBEC was at 7%. The reason for a higher amount of Bitumen emulsion required for 100 NA was due to the use of fresh aggregates as there was no RAP bitumen present on the surface of the aggregate, which increased the requirement of fresh bitumen emulsion due to increased surface area. However, for 50 RAP and 70 RAP, the presence of stiff aged binder and dust from the lower layer around the aggregate surface can be the possible reason for lower emulsion content, as it is difficult to bind them with RAP aggregate. This is in agreement with some previous studies [10, 15, 24, 40].

Based on OBEC, liquid antistripping additive Levasil was added, and $6 * 3 * 3=54$ mixes were prepared and further tested for ITS_{dry} and ITS_{wet} to determine the optimum dosage of liquid antistripping additive. It can be seen from the data in Figure 6 that 100 NA reported significantly more average ITS values, with and without using liquid antistripping additive, than the other two groups, namely 50 RAP and 70 RAP. Figure 7(a) depicts the ITS machine used for testing, Figure 7(b) for sample conditions after the testing and Figure 7(c) for RAP material shown after splitting the sample into two halves. This can be related to lower impact, crushing, and abrasion values of natural aggregates as compared to those with aged RAP aggregates. It was noted that 70 RAP with 0.4% RS generates average ITS_{dry} and ITS_{wet} values of 496 kPa and 464 kPa, whereas 50 RAP with 0.4% RS was able to get average ITS_{dry} and ITS_{wet} values of 410 kPa and 352 kPa, respectively, which were 17% and 24% more than 50 RAP. The results clearly demonstrate the higher ITS values of liquid antistripping additive Levasil samples as compared to the samples without Levasil irrespective of the RAP percentages that clarify better adhesion of bitumen emulsion with aggregates. However, on closely observing the values (average of three Marshall molds) from Figure 6, it can be observed that ITS_{dry} and ITS_{wet}values kept on increasing upto a certain limit by adding liquid antistripping additive , then it decreased. The main reason

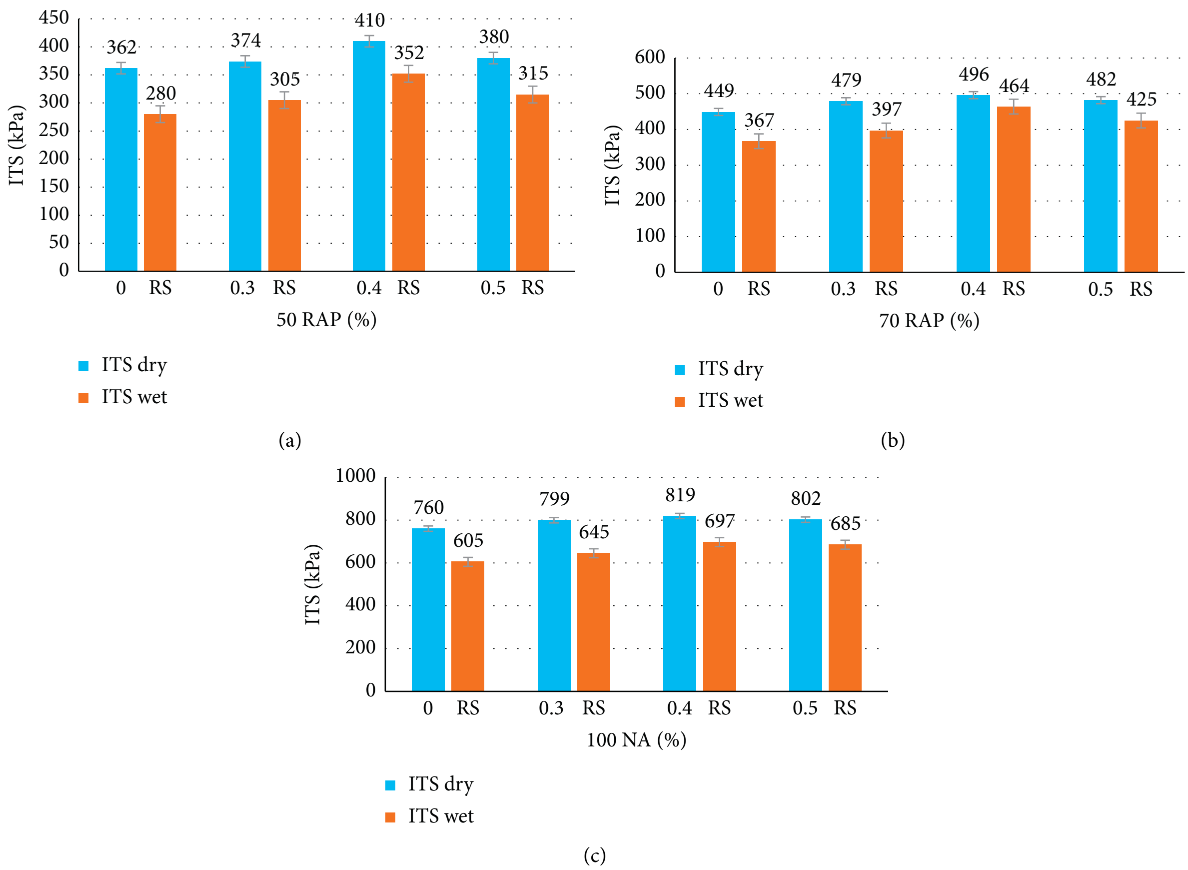


Figure 6: ITS values for (a) 50 RAP. (b) 70 RAP, and (c) 100 NA.

behind this is due to the stiff nature of bitumen emulsion by the addition of Levasil in it. The same behavior was observed during binder testing as described above in softening point and ductility test results. This trend of results is in agreement with few previous studies [41, 42].

3.3. Moisture Susceptibility Evaluation. The moisture susceptibility of compacted specimens was evaluated in terms of TSR value. The presence of moisture results in loss of adhesion between the bitumen emulsion and aggregates [37]. The samples of ITS_{wet} were further kept in a water bath for the next 24 hours curing at 25 °C for ITS_{wet}. The specimens were then put on the drainboard for 30–40 minutes to remove excess moisture from it before testing the indirect tensile strength. From Table 3, it was observed that samples with Levasil showed higher TSR values as compared to those without adding Levasil. In general, 70 RAP exhibited a higher TSR value as compared to 50 RAP and 100 NA mixes. Higher TSR values indicate better moisture resistance against moisture damage which is important in the case of ETB as it contains OFC also in its mixture. The TSR values of RAP mixtures showed better resistance against moisture susceptibility than the 100 NA mixtures since RAP mixtures were already stiff due to aging during its service life.

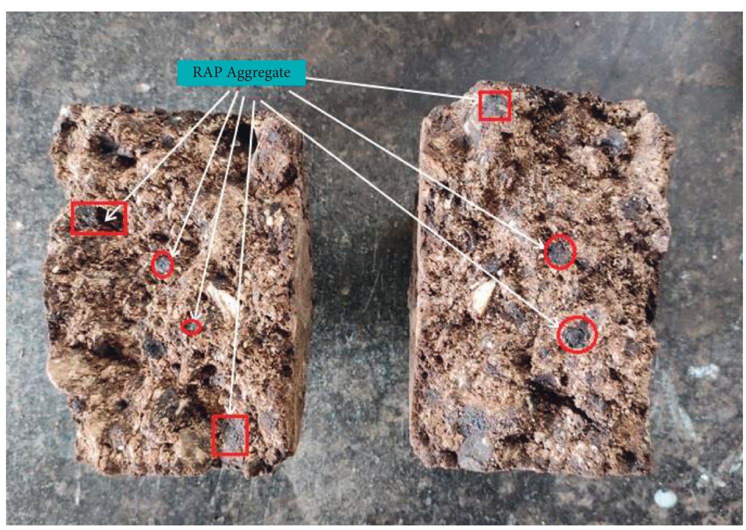
3.4. Stability Loss in Water. An effort has also been made to determine the loss of stability (%), which was calculated for specimens of optimum binder and additive content after curing at 25°C in water. Figure 8(a) demonstrated that even after keeping the samples submerged in water for 24 hours, samples showed improved stability of both RAP mixes. A similar trend can be seen for mixes prepared with 100 NA. Usage of cement and Levasil can be combined reasons for the lesser loss of stability in the RAP and NA mixes. Stability loss in water has not been reported in the past literature, but the authors believe that it can help the researchers in identifying the mixture behavior after keeping the samples immersed underwater, as shown in Figure 8(b). Also, stability loss in water indicates the durability of the sample, and it highlights the impact on the sustainability of RAP materials.

3.5. Resilient Modulus (MR). Pavement response to loading can be measured by resilient modulus (MR). The resilient modulus test values were found by taking an average of three values from each sample of optimum emulsion content at RAP content, i.e., 50 RAP, 70 RAP, and 100 NA, and, with a combination of 0% RS and 0.4% (optimum dosage of antistripping additive), respectively. Since the resilient modulus test was nondestructive, the same sample can be used to take three readings from each sample. The test was conducted at 35°C, and the results are depicted in Figure 9.

From Figure 9, it can be observed that for the mixes prepared with RAP content and antistripping agent, MR

(a)

(b)



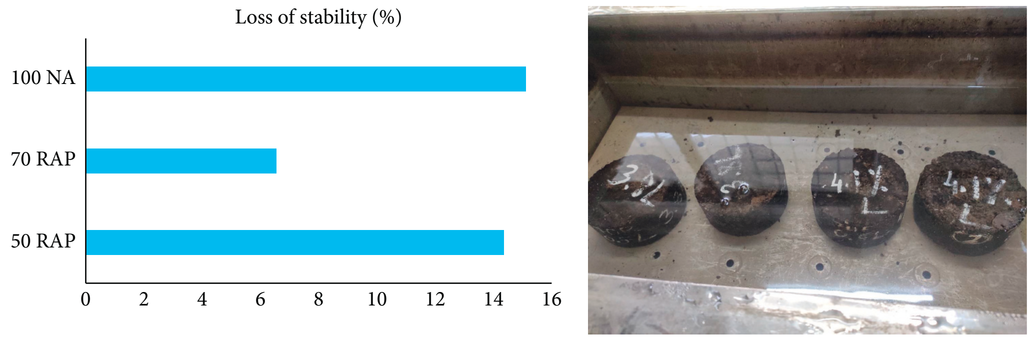
(c)

Figure 7: (a) ITS testing machine. (b) Sample after testing. (c) Sample after breaking.

Table 3: Tensile strength ratio values and coefficient of variation (COV) for 50 RAP, 70 RAP, and 100 NA.

Mix notation	50 RAP		70 RAP		100 NA	
	Average TSR (%)	COV (%)	Average TSR (%)	COV (%)	Average TSR (%)	COV (%)
0% RS	77	3	82	3	80	4
0.3% RS	82	4	83	2	81	3
0.4% RS	86	2	94	1	85	2
0.5% RS	83	3	88	2	85	2

values were higher than that of natural aggregate mixes. As the RAP percentage increased, resilient modulus values also increased, indicating that the presence of RAP makes the mix stiffer. This behavior can be attributed to the stiff nature of RAP-modified mixes, the same was confirmed by different rheological test results discussed in the above sections. From all types of mixes, 100 RAP showed lesser MR values as compared to 50 RAP and 70 RAP, reason being it having no



(a) (b)

Figure 8: (a) Los of Stability (%). (b) Samples submerged in a water bath.

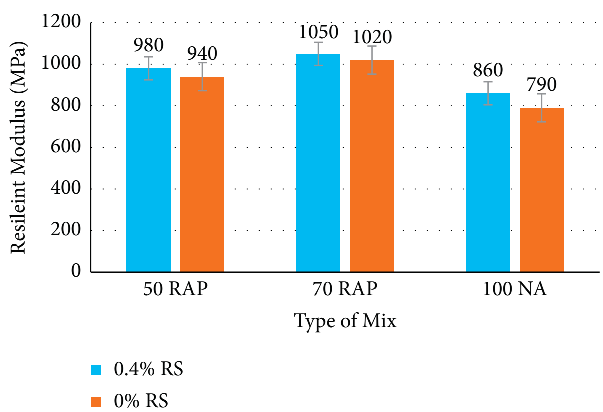


Figure 9: Resilient modulus of emulsion-treated base mixes at 35°C.

RAP percentage. However, the effect of different percentages of bitumen emulsion on MR value needs to be studied further.

4. Conclusions

The use of liquid antistripping additives with RAP inclusive mixtures was studied using different RAP percentages for ETB mix design aiming the sustainability aspect during research work. The final mix design includes an optimum percentage of additive, RAP, and bitumen emulsion with a constant percentage of cement as a filler material. Hence, based on aggregate gradation and blending exercise, 50 RAP and 70 RAP were considered along with 100 NA as a control mix for the mix design procedure. As gradation plays a vital role while designing ETB mix and can affect the mix performance adversely, that is why 100 RAP was discarded as it did not meet the aggregate gradation requirements. Binder properties were evaluated in detail, including conventional tests studying the effect of liquid antistripping additives on bitumen emulsion residual binder. Maximum dry density and optimum fluid content, indirect tensile strength test, tensile strength ratio, and resilient modulus tests were conducted to predict the performance and for the mix design of ETB in the laboratory. An attempt was also made to predict the stability loss (in percentage) due to the curing of conditioned samples, which can be a new parameter added to predict the strength and performance of the ETB samples prepared in the laboratory. Following are the conclusions drawn, based on the various test results:

(i) Properties of 50 RAP and 70 RAP aggregates were satisfying as per the specifications, but 100 RAP was rejected for the current study as it did not meet desired gradation specifications. However, water absorption of RAP aggregates was higher on comparing it with the NA, which is the main reason for the increased OFC in RAP mixes.

(ii) Observed OFC of RAP mixes was more as compared to 100 NA mixes due to increased water absorption behavior of RAP mixes due to poor internal friction of RAP mixes.

(iii) Although 100 NA mixtures performed better in terms of ITS and TSR than 50 RAP and 70 RAP, using RAP considerably helps reduce the project's cost and saves natural resources resulting in ETB as a sustainable construction technique for the years to come. Also, 100 NA showed more stiffness that can change the material behavior, and performance might be significantly compromised as the mixture may lose its flexibility.

(iv) The addition of liquid antistripping additive Levasil improved the performance of ETB mixes which shows better resistance to moisture susceptibility and increased compatibility of ETB mixes.

(v) Improved stability was observed when the mixes were kept in a water bath for 24 hours, indicating the mix's improved performance. It shows better adhesion property of the ETB mix.

(vi) Resilient modulus of the RAP mixes was more as compared to natural aggregate mix, which clearly indicated that due to the presence of RAP aggregate, the mix became stiffer. MR values for 70 RAP were highest among all the ETB mixes.

Overall, this study strengthens the idea that while using RAP and liquid antistripping additives together, the performance of ETB may improve significantly, and the use of natural aggregates can be restricted, resulting in considerable savings in the project and saving natural resources. Hence, the current laboratory mix design and construction method by CIPR is recommended for the construction of the ETB layer as a sustainable way of construction.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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Research Article

A Novel Technique to Utilize Second Waste of Plastic Bottle as Soil Reinforcement: A Comparative Study on Mechanical Properties with Natural Black Cotton Soil

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Black cotton soils, which are expansive, are present in abundance in Ethiopia. This type of soil possesses expansion when saturated with water and contraction during hot seasons, due to which it is labelled as "weak soil." They may remain a threat to the structures if they are constructed over them without precautions. The quality of such soils can be improved by treating them with suitable stabilizers or soil reinforcers. This paper discusses the chances of using the second waste of plastic bottles as a reinforcer to strengthen weak black cotton soils in Ethiopia. Second, plastic bottle waste was added at 1%, 2%, 3%, 5%, 7%, and 9% to the soil, and numerous trials were conducted to ensure the reliability of the results. The effects were analyzed based on the results from the Atterberg limit tests, compaction tests, unconfined compression strength (UCS) tests, and the California bearing ratio test (CBR) for soaked and unsoaked conditions. The results were compared against the natural soil results, and the optimum usage percentage of second waste plastic required to reinforce the soil was reported. The results indicate that among the various properties used, the mix with 2% second waste plastic is effective with numerous trials being conducted to ensure the reliability of the results and decreased values of OMC by 18.5%, increased MDD by 1.9%, increased CBR by 50.9%, and increased UCS by 10.1%. Thus, the research provides a novel technique to recycle plastic waste once again as soil reinforcement, thereby saving the environment from dumped waste.

1. Introduction

Soil particles containing clay particles possess severe strength loss, especially during the rainy season, and will show signs of shrinkage in summer. Black cotton soil is an excellent example, with the characteristics mentioned above. In Ethiopia, the major portion of the country's roadways, lightly loaded residential and commercial buildings, airfields, and planned railway systems, as well as a significant portion of the country's newly planned railway systems, are built on expansive soils. Uba [1] reported that expansive soils are the major cause of the destruction in Ethiopia as they are not suitable to carry heavy loads. They cause large volume changes, causing severe slope and foundation damage [2]. These types of soils can be made suitable to bear heavy loads by performing certain techniques such as stabilization and soil reinforcement involving the use of admixtures, waste materials, or with natural plant fibers such as coir waste, which can improve the physical characteristics of the soil [3, 4].

1.1. Literature Review

1.1.1. General Materials Used for Soil Stabilization. Many minerals and supplementary cementitious materials are used for soil stabilisation, such as lime [5], Bagasse flies ash [6], bottom ash [7], concrete slurry [8], and stabilizers like ordinary Portland cement, which is frequently used as an individual or in combination with any supplementary cementitious material like Metakaolin, as it leads to forming hydration products essential for soil stabilization effect [9–11]. However, the use of such alternative cementitious materials may be required in larger quantities and, depending on the circumstances, may be uneconomical. This can be made economical by blending weak soils with industrial waste and other possible waste materials that will be used in minimal quantity.

1.1.2. The Use of Waste Materials such as Soil Stabilizers. Solid waste reuse has gained popularity in recent years as a method of addressing long-term waste management. Soil stabilisation with recycled byproducts of manufacturing operations has a lower environmental impact and reduces the cost of reinforcing the soil before construction. E-waste, glass, and plastic waste are among those wastes which are great sources of land pollution, causing severe dumping problems globally. They are less likely to harm the environment when used as a soil stabiliser or reinforcer [12]. One of the main advantage of using plastic wastes in the soil as stabilisers to the environment is reducing the danger of blocking water bodies due to dumped plastic wastes [13]. Earlier researchers invested a significant amount of time and effort into using waste to stabilise soil. Malkanthi et al. [14] discussed using crushed concrete waste along with river sand in order to optimise the particle packing and certified that compressed stabilised earth blocks contribute to the reduction of pollution caused by building waste. Sivakumar et al. [15] utilised concrete slurry waste generated from ready-mix concrete as a soil stabilising agent and identified that slurry waste helps in stabilising soil by calcium ion exchange and reported that the use of such waste improved the soil stabilisation effect by 38% greater than that in which ordinary Portland cement was used. Pateriya et al. [16] tried stabilising weak soil using marble waste blended with fly ash, cement, and nanomaterials, and the outcomes revealed that such a blended mix acts as a potential source for stabilising weak soil. There has been a lot of research done on other waste items like granite waste, oxygen furnace slag, and fly ash [17, 18]. Because of the presence of silica, glass wastes can be utilised in multiple ways in the construction industry: as fine aggregate in the form of glass powder [19, 20], as glass fibres in fibre reinforced concrete [21], and even as a stabilising agent in weak soil [22, 23]. Similarly, e-waste can also be utilised as a different resource in construction activities as a partial replacement for fine aggregate.

1.1.3. The Use of Plastic Waste in Soil Stabilization. Plastic bottle overuse, on the other hand, can be found all over the world [24] , and there is limited usage of the disposed of plastic waste in construction industries. Oyinlola et al. [25] had tried to construct plastic bottle brick houses with water plastic filled with earth. They are utilised in various forms in construction works even then the disposed plastics are available in plenty. Plastic consumption and production are increasing in Ethiopia. According to EUROMAP (European plastics and rubber machinery), Ethiopia will produce 386,000 tonnes of plastic by 2022, with per capita consumption reaching 3.8 kg. Plastic consumption per capita in Ethiopia has increased by approximately 13.1 percent per year in recent years, rising from 0.6 kilograms in 2007 to 2.8 kilograms in 2018, and is expected to reach 3.8 kilograms in 2020. Use of plastic waste has already been tried by earlier researchers in many forms, Farah, and Nalbantoglu [26], discussed the performance of plastic waste for improving soil properties. Peddaiah, et al. [27] tried using plastic waste in the form of strips and improving the soil characteristics. Ferriera et al. [28] used polyethylene terepthalate bottles in fibre form in the sand and reported that the mixed sand showed better performance.

According to the above literature review, even though much works is done with plastic waste, it continues to be a major problem in terms of disposal. In this study, second waste of plastic material is used to reduce the aforementioned problems, with the specific goals of improving the plastic index as well as the strength values of black cotton soil and minimising environmental pollution by reducing the amount of plastic waste going to landfills.

1.2. The Scope and Significance of the Research. This paper discusses the use of various waste materials as soil reinforcement and compares them with that of second-hand plastic bottles in enhancing soil properties. Though many studies have been published on strengthening weak soils with plastic waste [29, 30] and ultrafine cementitious materials, few studies have been conducted on the second waste of plastic material. In the present study, the use of the second waste of plastic material as a soil reinforcer and its effect in strengthening the soil structure was studied by performing various tests such as Atterberg limit tests, proctor compaction tests, unconfined compressive strength tests, california bearing ratio tests, and undrained shear strength tests.

2. Materials and Methods

2.1. Materials under Consideration. Natural black cotton soil and waste plastic bottles were the basic materials considered for the current study. The samples of natural black cotton soil were collected in Addis Ababa, Ethiopia, from the bole subcity. Waste plastic bottles were processed (grinding, washing, and drying) by a plastic recycling machine at the Coba Impact plastic recycler company. The second type of plastic material is different from the other types of plastic materials by their size, shape, and strength, as well as this

Table 1: Laboratory result of a natural soil sample.

Test categories	Laboratory tests	List of description	Results
Index properties	Particle-size distribution (PSD)	Gravel	0%
		Sand	20.62%
		Silt	26.46%
		Clay	52.92%
	Atterberg limit (Casagrande cup method)	LL	102%
		PL	36%
		PI	66%
	Natural moisture content	W	38.23%
	Specific gravity	G_S	2.66
Swelling properties	Free swell index	Free swell (S_f)	110%
		Differential free swell (S_{df})	90.9%
	Swelling potential	S	12.06
Compaction tests	Standard compaction	Maximum dry density (MDD)	1.13 g/cm^3
		Optimum moisture content (OMC)	37.78%
Strength tests	Soaked California bearing ratio (CBR)	CBR	2.14%
	Unconfined compression strength (UCS)	UCS	246.54 kPa
		ε_f	8.68%
A unified soil classification system (USCS)		CH (highly plastic clay) black cotton soil	

material does not have any purpose, rather it engaged the dumping areas with too much space and contribute to the pollution of the environment.

2.1.1. Natural Soil. Using a combined sieving and hydrometer test process, a continuous particle-size distribution curve of soil can be plotted from the size of fine sand particles down to the size of clay particles. Table 1 presents the laboratory results of natural soil tests, which include the details obtained from the particle size distribution as per ASTMD6913M-17 [31], the Atterberg limits for the Casagrande cup method of liquid limit (LL), plastic limit (PL), and plastic index (PI) values as per ASTMD4318-17 [32] and CBR tests as per ASTMD1883-16 [33], Based on the results from the unified soil classification system (USCS) done as per ASTMD2166-16 [34], it is inferred that the soil sample could be categorized as highly plastic clay (CH).

2.1.2. Second Waste of Plastic Bottle Stabilized Black Cotton Soil. The waste plastic materials or bottles collected were crushed in a plastic crusher machine. The crushed materials were collected, washed, and dried. Figure 1 shows the material used in the current research. Earlier, Preethi [35] had used plastic strips such that they passed through a 4.75 mm sieve for soil stabilisation in the present work, the second waste plastics passing through a 4.75 mm sieve size and retained at 0.075 mm were segregated and considered for mixing with black cotton soil. Moreover, in the present work, the crushed pieces and the process was done as per ASTM D6913M-17, 2017 [34].

2.2. Mix Proportions, Methodology, and Testing. A total of 7 mixes, including one control mix (C) and 6 random mixtures (M1, M2, M3, M4, M5, M6) consisting of second waste of plastic bottles added (by weight) with natural black cotton soil in 1%, 2%, 3%, 5%, 7%, and 9% were made. Amena [3, 36] has used plastic stripes in 0.5%, 0.75%, 1%, 1.5%, and 2% and reported that increasing the percentage of plastic strips increased the CBR values and cohesion of soil by up to 1.5%. Gangwar and Tiwari [35] used waste plastic bottles for soil stabilisation in 0.5%, 1%, 1.5%, and 2%. Because the second waste of plastic is smaller in size and is used to implement a large usage of waste in the current work, the proportion began with 1%. Two types of laboratory tests, namely, tests for natural soil classification and analysis of the strength of black cotton soil stabilised by second-hand waste plastic bottles, were performed. Atterberg limit tests, particle size distribution, specific gravity, compaction parameters, unconfined compressive strength (UCS), CBR (soaked and unsoaked) tests, and soil swelling characteristics were performed to determine the soil samples' features. The cone penetration method was also used to check the potential of the plastic material. All the tests were done as per ASTM standards.

Figure 1: Second waste of plastic bottles.

Figure 2: Casagrande cup for performing Atterberg limits.

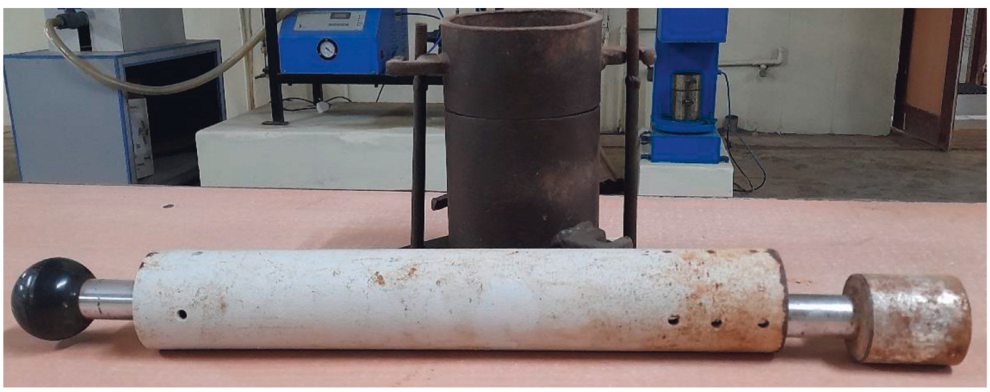

Figure 3: Set up for performing standard compaction tests.

2.2.1. Atterberg Limit. The Atterberg limit test is used to determine a soil's plasticity qualities and utilizes them as input index parameters for soil classification. The Casagrande method was used to determine the soil's liquid limit. Figure 2 shows the setup utilised for the present research. The Atterberg limits that are used to measure the distinctive features of soils in terms of water content are liquid limits (LL), plastic limits (PL), and plastic index (PI). The test was performed as per ASTMD4318-17 [32] requirements.

2.2.2. Standard Compaction Test. ASTM D698-12, 2012 [37] was used to calculate the maximum dry density (MDD) and optimal moisture content (OMC). Three layers of air-dried soil samples were crushed with 25 blows. After compaction, the mold is trimmed and the compacted dirt mass is measured. The soil moisture content is also determined. From the densities and moisture contents of the compacted soil experiments, a graph between dry density and moisture content is generated. Figure 3 depicts the test setup, and the properties are already listed in Table 1.

2.2.3. California Bearing Ratio. The CBR test was used to find the strength of the mixed soil based on the guidelines of ASTM D1883-16, 2016 [33]. The homogeneous mixture of samples was compacted in the California Bearing Ratio (CBR) test molds for both the soaked and unsoaked conditions, and CBR values of the soil with varying amounts of plastic were found. In the soaked CBR test, the samples were immersed in water for four days to investigate the environmental and climatic effects of natural and plastic stabilised black cotton soils. Figure 4 displays the CBR apparatus used for current research.

Figure 4: CBR test setup.

Figure 5: UCS test setup.

Table 2: Variations of the Atterberg limit value of stabilised soils with different percentages of the waste plastic bottles.

Mix ID	LL (%)		PL (%)	PI (%)	PI (%)
	Casagrande cup	Cone penetration	Conventional method	Casagrande cup	Cone penetration
C	102	83	36	66	47
M1	102	83	36	66	47
M2	99	82	35	64	47
M3	93	81	34	59	47
M4	91	80	34	57	46
M5	93	77	33	60	44
M6	96	74	32	64	42

2.2.4. Unconfined Compressive Strength. Unconfined compressive strength as specified in ASTM D2166-16, 2016 [34] was done for the samples collected. As a consequence of this test, the soil's maximal strength has been confirmed to be 135.72 kPa. With this UCS value, the soil is classified as stiff clay soil. The test set up is displayed in Figure 5.

3. Results and Discussion

3.1. Atterberg Limit Result. The Atterberg limits for the Casagrande cup method of liquid limit (LL), plastic limit (PL), and plastic index (PI) values are presented in Table 2.

The LL of 5 percent plastic stabilised soil is reduced by 10.8 percent for the Casagrande cup method, and the LL of 9 percent plastic stabilized soil is reduced by 10.8 percent for the cone penetration method, compared to natural black cotton soil, while the PL of 9 percent plastic reinforced soil is reduced by 11.1 percent. With coefficients of determination of $R^2 = 0.999$ (at 5% in the Casagrande cup method) and 0.98 (at 9% in the cone penetration method), it is clear that the second waste plastic reinforcement resulted in a progressive decrease in LL value. As a result, the plastic reinforcer aids the black cotton soil in lowering its moisture content in both methods.

The PI value of natural soil is reduced by 13.6 percent when stabilised by 5 percent plastic material for the Casagrande cup method, and by 10.6 percent when reinforced by 9 percent plastic material for the cone penetration method. As a result, 5 percent plastic reinforcer is the ideal content for the Casagrande cup method.

3.2. Standard Compaction Test Results. Table 3 presents the OMC and MDD results of waste plastic reinforced black cotton soil improvement using the standard compaction method.

Table 3: OMC and MDD values.

% of plastic waste	OMC (%)	MDD (g/cm^3)
C	37.78	1.130
M1	36.1	1.146
M2	30.8	1.151
M3	36	1.144
M4	37.47	1.143
M5	38	1.112
M6	38	1.092

Adding a second plastic waste resulted in a progressive increase in MDD and a roughly linear decrease in OMC until the reinforcer content reached its optimum, i.e., at 2%. The OMC is reduced by 18.5 percent for the replacement of waste plastic reinforcer with 2%, the reason is due to the poor water absorption capacity of the plastic reinforces, which is an important property to be considered when using it on road pavements. The same trend was reported by other researchers that plastic waste stabilizer were used [3, 38]. Also the MDD of soil increases up to 2% replacement levels and decreases thereafter as the percentage of replacement is increased. This is due to the fact that density decreases when the same volume of soil is replaced by lighter materials. The increase in MDD value indicates that the strength of black cotton soil improves as more solids might have been filled in the given volume. In terms of percentages of dry weight of soil, the moisture content of reinforced soil was reduced by replacing black cotton soil with waste plastic material. Furthermore, when a compaction force is applied, the stabilised soil becomes interlocked with one another due to the roughness of the surface of the small plastic material. The

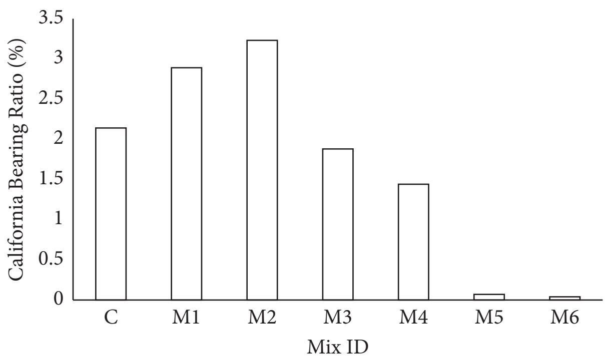

Figure 6: CBR values of soaked soil samples.

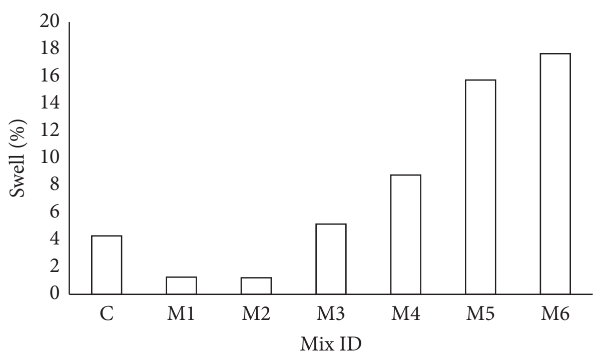

Figure 7: Swell percentage of various soil samples.

results showed that the use of second waste of plastic as a reinforcer reduced the negative effects of soils with high OMC values to some extent.

3.3. California Bearing Ratio Results. According to the study, CBR values increased for a replacement level of 2% plastic reinforcer and showed an improvement of 50.9 percent, while the percentage swell of the 2% plastic reinforced soil decreased by 71.6 percent when compared to natural black cotton soil. This indicates that the CBR of the optimum plastic stabiliser, at 2%, increased by half the amount of the natural soil. Furthermore, the results of unsoaked CBR tests with 0%, 2%, and 3% plastic reinforcer were 9, 12, and 9.31 without swell. The friction action between plastic and black cotton soil at OMC and MDD is responsible for this improvement. According to the results of the tests, the CBR values of the reinforced soil increased significantly under unsoaked and soaked conditions. As a result, the strength of the black cotton soil was enhanced by the use of a low-cost, economical waste reinforcer. In a soaked condition, however, CBR values of waste plastic reinforced soil greater than the optimum content decrease due to high swelling characteristics. The reason for such a decrease in CBR after 2% replacement of second waste of plastic was that mixture showed low resistance to penetration and mixtures may not have possessed proper interlocking as the presence of smooth plastic particles exceeded the optimum limit. Amena [3], who had been using plastic strips for enhancing the subgrade properties, also mentioned that there was an increase in CBR values up to 1.5% and the values decreased

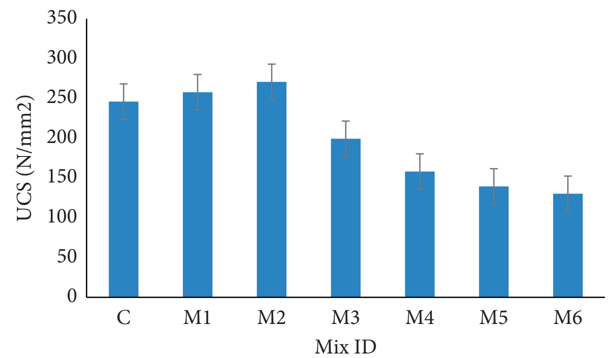

Figure 8: Values from the unconfined compressive strength test.

Figure 9: UCS failure of 2% waste plastic stabilized black cotton soil.

thereafter. Also, earlier reports state that when plastic waste is used in the form of trips, the optimum CBR values were obtained at 1% [39], so it is comparatively better as there are chances for adding plastic waste at a slightly increased level by recycling and using the second waste. Figures 6 and 7 display the CBR and percent swell values.

3.4. Unconfined Compressive Strength Results. The unconfined compressive strength (UCS) result is presented in Figure 8. It is also known that as the MDD of the soil rises, the UCS parameters of the soil also rise, eventually reaching the optimum percentage of second-waste plastic stabilizer. The UCS value at 2% optimum plastic stabilizer is 271.43 kPa, which is greater by 10.1% than natural black cotton soil. The reason for such a decrease in UCS may be due to the smoothness of the plastic reinforcer. When more p-second waste of plastic was added, the strength decreased due to the low resistance offered by the samples, as proper packing is not ensured between the soil particles and the smooth plastic waste. Amena [3] found the same pattern in their study of waste plastic bottles. So, from the results, it is inferred that when the proportion exceeds 2%, the soil begins to form weak shear lines along which it can fail quickly. Hence, the most optimal percentage of waste plastic that should be used to increase the strength of black cotton soil is 2%. The inclined failure line at failure strain is 7.37% of the unconfined compressive strength after the application of compressive load is shown in Figure 9.

4. Conclusion

By suggesting the use of second waste of plastic along with black cotton soil for reinforcing processes, this project intends to improve the strength of expanding soils while also reducing the environmental pollution. Thus, the research could serve three goals. The first is to develop a good way of disposing of plastic waste; the second is to enhance the performance of black cotton soil; and the third, and most importantly, is to make the process economical by using waste.

The following findings are obtained based on the analysis and interpretations: The second waste of plastic bottle material proved to be a good reinforcement when used in different proportions for the replacement of dry weight of soil by improving the engineering properties of black cotton soil; use of second plastic waste in 5% has led to a considerable decrease of 13.6% compared to the natural black cotton soil as presented in the Casagrande cup method; the plastic as a reinforcer had increased the MDD values up to an optimum level of 2% usage, beyond which the values started decreasing; and the OMC values were seen decreasing up to 2% replacement level and thereafter an increase in moisture content was noticed. So, it is inferred that the M2 soil mix with a 2% effective amount of plastic waste, remained as the optimum mix based on the analysis of the results from tests such as compaction, CBR, and possessing less swelling percentage and registering a higher value for UCS than the natural black cotton soil.

Thus, according to the findings, using the second waste of plastic bottles as reinforcement of black cotton soil can allow for proper plastic waste disposal and soil enhancement.

Data Availability

All the data used to support the findings of this study are included in the manuscript.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

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Research Article

Mechanical Behavior of Plastic Strips-Reinforced Expansive Soils Stabilized with Waste Marble Dust

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Expansive soil needs to undergo treatment to be used as safe foundation soil for roads and buildings. From an environmental conservation and economical point of view, the usage of agricultural and industrial wastes is the best option. In this study, the effects of utilizing plastic waste and marble waste dust on the engineering properties of expansive soils were examined. Various laboratory tests were carried out on sampled expansive soil by adding 10, 15, and 20% of marble and 0.25, 0.5, and 0.75% of 5×8 mm^2 plastic strips. The laboratory test results showed that there are good enhancements on strength parameters due to the addition of marble dust and plastic strips. With an increase in percentages of marble dust and plastic strips, California Bearing Ratio (CBR) values rise. With the addition of marble dust, unconfined compressive strength (UCS) values increase linearly, while they increase only up to 0.5% with the addition of plastic strips. As the proportions of marble dust and plastic strips increase, the soil's free swell and CBR swell are decreased significantly. This shows that environmental pollution waste marble dust and plastic strips can be utilized to strengthen the weak subgrade soil and minimize its swelling properties. Therefore, this study found out that the expansive soil treated with polyethylene terephthalate (PET) plastic and marble dust can be used as a subgrade material since it fulfills the minimum requirement needed by standards.

1. Introduction

Expansive soils are sensitive to moisture content, which results in volume change during seasonal changes. They are present in large amounts all over the world. However, the volume changes during seasonal fluctuation, cause excessive deformation in soil, which is destructive to civil infrastructure. In Ethiopia, expansive soils are found abundantly, causing much damage to road pavements [1, 2].

In subgrade road construction, expansive soils must be replaced with other materials or treated to be used as pavement layer materials. Replacing poor soil with the selected material is unrealistic and expensive due to the massive volume of subgrade work. Therefore, treating poor soil in its in situ place is the preferable option, both economically and in reality. There are many ways by which the swelling property of expansive soils can be improved. The widely used mechanisms are stabilizing the soil mechanically and chemically. Mechanical stabilization is the physical property improvement by using techniques like compaction, dry wetting, prewetting, and reinforcement. Chemical stabilization stabilizes the soil using additives such as lime, cement, fly ash, and others [3, 4].

Recently, from an environmental conservation and economic point of view, the use of agricultural and industrial wastes as soil stabilizers has been recommended. Using waste by-products as stabilizing agents has two major advantages [5–7]. It plays an important role in reducing environmental pollution and improving the strength and swelling ability of expansive soils. Research has been conducted on the use of wastes such as plastic waste, brick waste, stone dust, fly ash, marble dust, sugar cane molasses, and other factory by-products to stabilize expansive soils [8–12].

Due to economic growth and changing consumption and production patterns, there is a rapid increase in the generation of plastic waste all over the world. In many areas, including Ethiopia, society has lack of awareness that plastic bottles and bags are thrown away in an open area, to the environment [10, 11]. Plastic waste is not biodegradable and takes time to decompose, which is why it is a common occurrence in open dumps and landfills. On the other hand, the high demand for marble products for finishing work in the construction industry generates a high accumulation of marble dust. In the process of cutting marble blocks, marble powder is mixed with water to form a suspension of marble waste, of which about 25% is powder. This accumulation of waste marble dust causes environmental pollution, and it occupies free construction area [15–17].

The reinforcement used in the study in [18] was plastic strips with 0.25, 0.5, and 1% and lengths of 10 mm, 20 mm, 30 mm, and 40 mm. They found that the strength of the stabilized soil increased by up to 0.5% with the addition of plastic strips, followed by a slight decrease in the CBR. The study was investigated in Ethiopia using plastic strips of different sizes (15×20 mm^2, 10×15 mm^2, and 5×7.5 mm^2), and 0.5, 1, and 2% additions by weight were added to the soil. The study showed that the optimum moisture content, swelling, and cracking decreased, while the maximum dry density increased slightly [8]. The experimental study conducted on an expansive soil sample having expansive behavior with 0.25, 0.5, 0.75, and 1% plastic fiber additions by weight of the soil resulted in significant improvements in swell potential, cohesion value, tensile strength, and unconfined compressive strength [19].

A study performed on the plastic fiber reinforcement in silty sand showed that significant improvement is achieved with the addition of 0.4% plastic fibers having a 15×15 mm^2 size [20]. While adding plastic strips and brick powder showed significant improvement by increasing UCS and CBR values, swelling potential decreased [21]. The study conducted by reinforcing expansive soils with plastic strips at 5% constant lime addition found that the plasticity and strength properties of the soil significantly improved at 0.75% plastic waste strips and 0.5% lime [22].

Several studies have investigated the effects of marble dust addition on the mechanical properties of expansive soils. In this study, the percentage of marble dust in the soil mass ranged from 10 to 50% proportions of the dry weight of the soil. A significant improvement in the values of the liquid limit and shrinkage limit was observed [23]. The effectiveness of this material to stabilize a weak soil has been studied by varying the percentage of addition to 5, 10, and 15% of the soil sample. The study found that the maximum dry density and optimum moisture content increased with increasing percentages of the stabilizer [24]. The study examined expansive soils mixed with percentages of waste marble dust (5, 10, 15, 20, and 25%) showed that maximum dry weight, USC, plasticity index, and CBR increased with the addition of marble powder while optimum moisture content and swelling potential decreased [25].

Several studies have shown that the addition of marble dust and other additives can increase strength characteristics while decreasing the swelling of the studied soils. An experimental study on expansive soil showed that the addition of 0, 5, 10, 10, 15, and 20% marble dust and rice husk ash reduced swelling and increased the maximum dry density, CBR, and compressive strength [26]. In another study, varying the addition percentages of the lime and marble powder resulted in a decrease in the plasticity index and OMC and an increase in UCS and MDD [27].

This study investigated the impact of the use of plastic and marble waste on the engineering properties of expansive soils. The choice of these materials was based on the observation that marble dust can improve the strength and plasticity of the soil, while plastic waste strips can improve the strength of the soil. This article describes the use of waste materials, including plastic waste and marble dust, to improve the strength and plasticity properties of soils.

2. Materials and Methods

2.1. Materials

2.1.1. Expansive Soil. The natural soil sample used for this research work was collected at a depth of 1.5 m from the ground level in Jimma town, Oromia, Ethiopia. Both undisturbed and disturbed samples were sampled and transported to the laboratory for experimental tests. An undisturbed sample was taken for the unconfined compressive strength test using the Shelby tube, and the in situ moisture content was determined by covering the sample with plastic bags. The soil is gray-black and highly plastic.

The results of laboratory tests on the index and strength properties of the soils are summarized in Table 1. The soil sample collected from the selected site was dried and sieved through the 425 microns for index property determination. The air-dried samples sieved through a 4.75 mm sieve were used for CBR and compaction tests. Laboratory soil tests such as particle size analysis (ASTM D6913), the Atterberg limits test (ASTM D4318), the compaction test (ASTM D698), unconfined compressive strength (ASTM D2166), and the CBR test (ASTM D1883) were conducted to determine soil properties [28–32]. Based on free swell and the plastic index, the soil sampled was categorized as highly expansive soil [33, 34]. The particle size analysis of the expansive soil sample is shown in Figure 1.

Gradation curves for soil and marble dust samples were obtained using wet sieve analysis. The results of the particle size analysis of the soil and marble dust are shown using gradation curves plotted in Figure 1.

2.1.2. Waste Marble. Waste marble dust (MD) was collected from construction sites, where it was left as waste during cutting, processing, and reshaping in finishing works. The dust of marble was placed in an oven for 24 hours to remove moisture and pulverized to remove agglomeration before use. The grain size analysis showed that the marble dust used in this study consists of 46% coarse-grained and 54% fine-grained from which 22% is clay and 32% is silt soils indicated in Figure 1. The specific gravity of marble is found to be 2.74.

Table 1: Summary of laboratory test results.

Parameters	Values	Units
Natural water content	42	%
Specific gravity	2.69	%
Plastic limit	35	%
Liquid limit	73	%
Maximum dry density	1.4	g/cm^3
Optimum water content	27.3	%
Unconfined compressive strength	71	kPa
CBR	1.5	%
CBR swell	8.48	%
Free swell	96.6	%

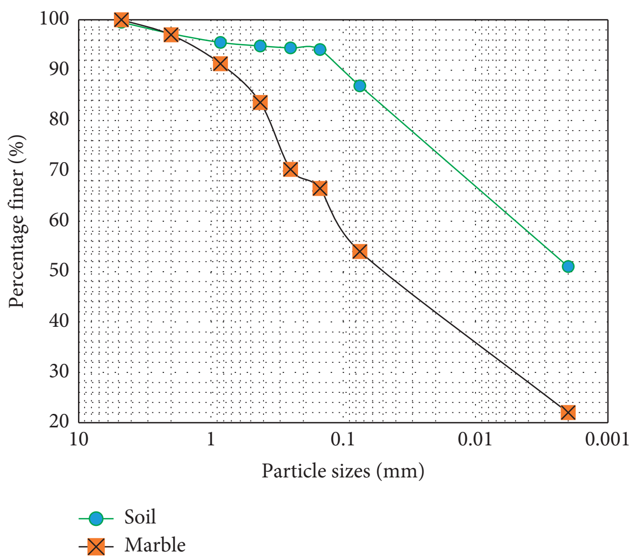


Figure 1: Gradation curves for the marble and soil.

2.1.3. Plastic Waste. As a reinforcing material, plastic waste strips (PS) made up of PET were used. Plastic waste materials, mainly water and soft drink bottles, were collected from different places in the study area. The collected PET water bottles were cut into strips of $5 \times 8\,\text{mm}^2$ manually as illustrated in Figure 2.

2.1.4. Sample Preparation. The marble waste was mixed with soil by proportions of 10, 15, and 20%. The plastic waste was cut by hand manually into 20 mm long strips. The plastic strips were mixed with the soil with marble dust at 0.25, 0.5, and 0.75% of the dry soil for determination of free swell, compaction parameters, CBR, and UCS (Figure 3).

To obtain the uniform mix soil sample, marble dust and plastic strips were mixed together carefully for each trials. The optimum moisture content and maximum dry density of the soil determined from the compaction test were used for remolding the stabilized samples for the CBR and UCS tests.

During sample preparation, as shown in Figure 3, the soil sample is allowed to air dry until it loses its moisture and dries completely. Then, the required amount of soil sample, marble dust, and plastic strips for each trial test are weighted. Marble dust and plastic strip percentages are determined by dry weight of the soil. For the specified amount of the soil sample, the required percentage of marble dust is added and thoroughly mixed until it makes the uniform mix. After that, the required percentage of plastic strips is added accordingly.

3. Results and Discussion

The addition of plastic strips and marble dust can significantly increase the strength of the treated soil. Many laboratory tests were carried out to observe the improvement in strength parameters and plasticity properties due to the addition of plastic strips and marble dust. The improvement in strength parameters was seen using the results of the compaction test, California Bearing Ratio (CBR), and unconfined compressive strength (UCS). The results of several laboratory experiments showing the effects of adding plastic sheets and marbles on the geotechnical properties of soil are discussed.

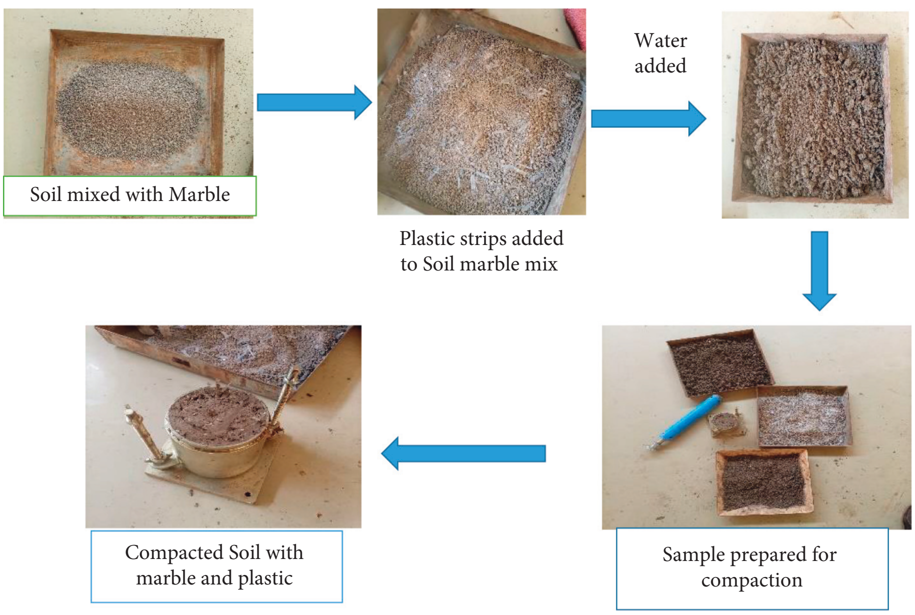
3.1. Effects of Plastic Strips and Marble Dust on Free Swell Value. For a free swell test, a $100\,\text{cm}^3$ test tube was filled with water. Then, for each trial, a total of 10 gm soil, marble dust, and plastic strips were added. Percentages of marble dust and plastic strips added to the mix were determined from a dry weight of 10 gm of soil. Figure 4 shows that the free swell of the soil decreases as the percentage of marble dust and plastic strips increases. The use of nonswelling marble dust helps keep the soil from swelling. Between the plastic strips and the soil, no chemical reactions take place.

However, the addition of plastic strips forms a gap between the particles of the soil so that it alters a free movement of soil particles. The alteration of the movement of particles due to the addition of the plastic strips resulted in the reduction of free swell. With increasing amounts of marble dust and plastic strips, the swelling potential of specimens decreased. There are no previously conducted studies using plastic strips and marble dust as a stabilizing agent for expansive soil. However, in past studies, marble powder according to [35–37] and plastic strips according to [8] were able to reduce the clay swelling.

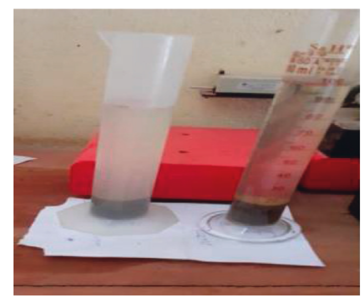
3.2. Effects of Marble Dust and Plastic Strips on Compaction Parameters. Compaction tests were conducted on treated soils with different amounts of marble powder and plastic additives. From the results of the compaction tests shown in Figure 5, it is observed that the MDD values vary slightly with the addition of the plastic strips and marble powder. The laboratory test results showed that MDD increases with an increase in marble dust content while it decreases with increasing plastic strips as shown in Figure 5. At a fixed proportion of marble dust, increasing proportions of plastic strips resulted in a decrease in MDD. However, if the percentage of the plastic strip is constant, the MDD increases as the percentage of marble dust increases.

There are several reasons why marble dust increases the MDD. The occupation of clay soil particles in marble dust void spaces, the cementitious behavior of calcium oxide

Figure 2: Waste marble and plastic strips prepared for tests.



(a)



(b)

(c)

Figure 3: Laboratory tests: (a) compaction tests, (b) free swell, and (c) unconfined compressive strength.

(CaO) content of marble dust, and replacement of low specific gravity expansive clay particles with high specific gravity marble dust. However, plastic strips are lightweight materials compared to expansive soil particles, which decrease the density of the mix.

Figure 5 shows the variation of the optimum water content for different proportions of marble powder and plastic strips. It can be seen that the optimum water content decreases as the proportion of marble powder and plastic bars increases. The optimum water content decreases as the percentage of plastic mass increases because of the low water absorption of the plastic bars. The addition of the marble powder slightly reduces the optimum water content due to the high density of the material and its low water absorption capacity compared to natural soil.

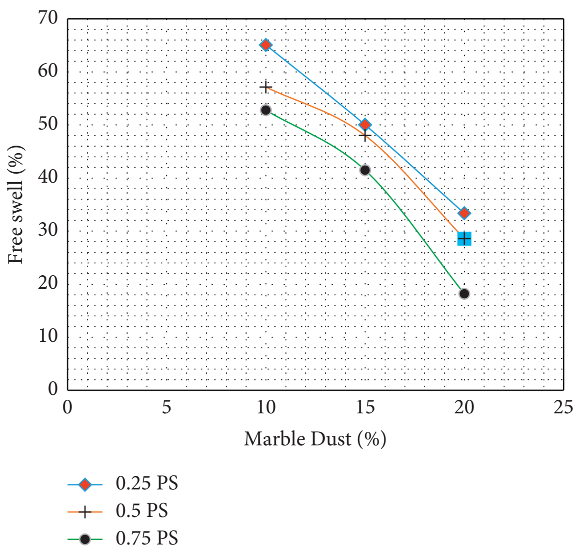


Figure 4: Variation of the free swell with marble dust and plastic strips.

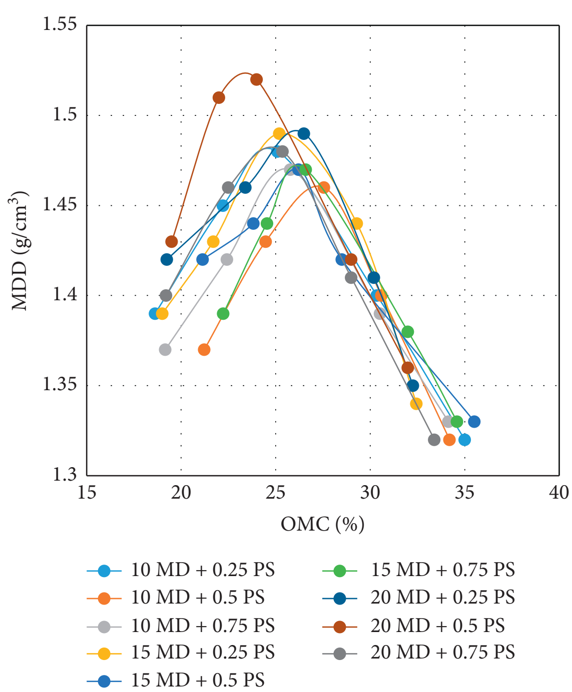


Figure 5: Variations of compaction characteristics with marble dust and plastic strip contents.

Studies have shown that the addition of marble dust and plastic strips has a significant effect on the compaction properties of expansive soils. The addition of marble dust increased the MDD and decreased the OMC, while the addition of plastic strips decreased both the MDD and OMC. Previous studies have found that exposure to plastic strips and marble dust is almost similar [5, 16, 18, 19, 21, 24]. According to [8], the greatest reduction was achieved by adding 2% plastic strips, which reduced the moisture content by 31%. The reduction in OMC may be due to the zero water absorption capacity of the plastic strips. Therefore, it was possible to compact the soil to its maximum dry density with little water addition, which is a very good improvement. A study conducted by [24] showed an improvement in OMC from 15.7% to 18.22%, which was proportional to the amount of marble powder. According to the study performed by [27], OMC reduced and MDD increased with an addition of marble dust. The addition of marble (a non-plastic material) increased the water-holding capacity of the soil mixture, which results in an increase in OMC. Deboucha et al. [39] observed a similar reduction in MDD and an increase in OMC from 10.78% to 12.96% when 5% marble dust was added to the fine-grained soil. The decrease in MDD is due to the increase in volume and the decrease in the mass-to-volume ratio [37].

3.3. Effects of Marble Dust and Plastic Strips on the California Bearing Ratio (CBR). The California Bearing Ratio (CBR) values are used as an indication of strength and bearing capacity in the design of road pavements between road base and subgrade. Soaked CBR tests were carried out with varying percentages of marble dust and plastic strips. The CBR tests were conducted at the soil's maximum dry density and optimum moisture content. Figure 6 shows the final results of the CBR when soil is treated with varying percentages of plastic strips and marble dust. The CBR value increased from 1.50% to 6.2% with the addition of plastic strips and marble dust. This shows significant improvement in treating expansive soil with plastic strips and marble dust.

This is because the addition of marble improves the soil gradation, and the addition of plastic strips alters the movement of the soil particles, reducing the change in soil volume and ensuring a high-bearing capacity. As the marble improved the soil gradation and the plastic strips changed the movement of the swollen soil particles, leading to higher CBR values. The results of this study are consistent with previous studies that were treated expansive soils with marble blocks and plastic strips [5, 15, 18, 22, 23]. An increase in the CBR by 27–55% was observed with the addition of plastic strips [37] and by 104% [8]. The study showed that an improvement of 108.4% was obtained in the CBR value from 6.19 to 12.9% when concentration of marble dust increased to 25% [25]. This improvement can be explained by the fact that marble fills the voids between particles in swollen soils, improves sorting, and increases dry density, leading to an increase in CBR values.

According to the Ethiopian Road Authority (ERA) manual [40], which is utilized in Ethiopia to build a low-volume flexible paving system. The CBR value of most clay soils is less than 15, and the soil having the CBR value less than 5 is classified as a poor subgrade material. Between 5 and 10, they are intermediate for subgrade materials. According to Schaefer et al. [41], soils with a CBR greater than 10 are suitable for the subgrade of road foundations. Natural soils have low CBR values and are therefore not suitable as untreated soil. This result is consistent with the study conducted at the same site [5, 41]. The study showed that the expansive soil stabilized with marble waste and

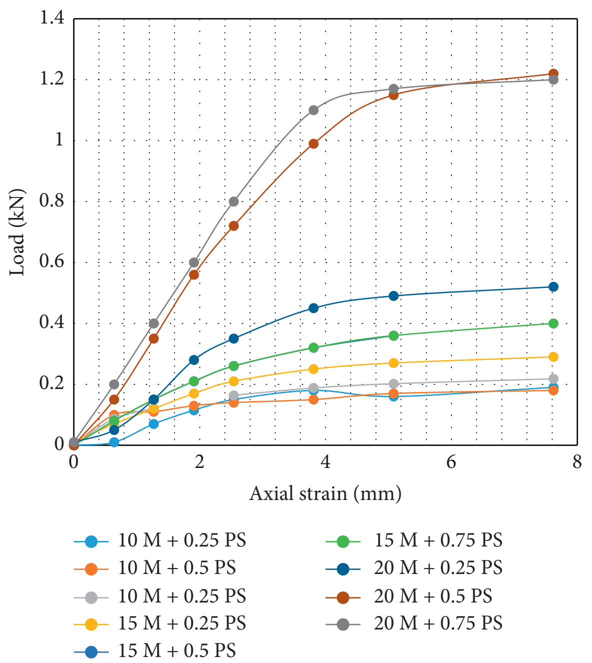


Figure 6: CBR graphs of improved soil at different percentages of marble dust and plastic strips.

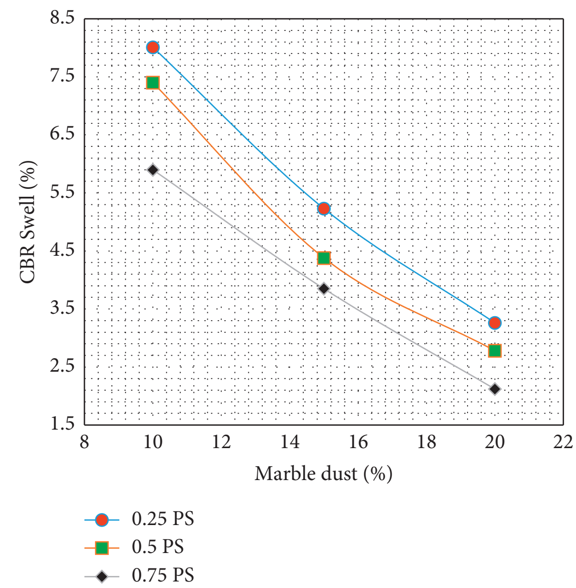


Figure 7: CBR swell variation with percentages of plastic strips and marble dust.

plastic strips met the minimum CBR of 5% required by the Ethiopian Road Administration (ERA) based on the CBR results [43].

3.4. Effects of Marble Dust and Plastic Strips on CBR Swell. Figure 7 shows the effect of PET plastic strips and marble chips on CBR soil swelling. Treatment of soil swelling with plastic strips and marble chips significantly increased the CBR of soil swelling.

The addition of nonswelling materials to swelling soil alters the change in volume of the soil. Since a mixture of plastic strips and marble dust replaces some percentage of the soil, nonswelling materials replace the swelling volume of the soil which decrease some percentage of the soil. The results show that the CBR swell values decrease with increasing plastic strips and marble powder percentages. With the addition of 0.75% plastic strips and 20% of marble dust, the highest improvement in the CBR swell value is observed. At this point, the value of CBR swell changed from 8.19 to 2.31%, which is a very significant improvement. This is because the particles of expansive clay are replaced by particles of nonexpansive marble dust particles, which act as an inert material. This study is consistent with previous investigations on the effect of plastic strips and marble fines in reducing the elasticity of expansive soils [18, 23, 24, 39, 40]. Tamiru et al. [46] measured the CBR swell of two soil samples after treating with marble dust, and the value decreased by 139 and 115%, respectively. The swelling rate of CBR expansive soils decreases as the marble dust content increases. This indicates that the swelling capacity of the samples decreases as the marble dust stabilizes. This is due to some chemical reactions between particles of expansive soils and marble dust. This is also due to the replacement of part of the volume occupied by the swollen clay minerals with marble dust.

3.5. Effects of Marble Dust and Plastic Strips on UCS Values. The unconfined compressive tests on natural and stabilized expansive soil were carried out. For plastic strips and marble dust treated expansive soil, UCS tests were carried out using the soil's MDD and OMC at varying percentages of marble dust and plastic strips.

Figure 8 shows the results of UCS tests at varying percentage additions of plastic strips and marble dust. The addition of marble dust and plastic waste strips has been shown to increase the UCS value up to 0.5% for plastic strips, but the value decreases slightly above 0.5%.

The increase in UCS comes from modification in compactness of the soil because of the addition of plastic and coarser marble dust, enhancement in gradation, the load-resisting capacity of plastic strips, and improved bonding between marble dust and soil particles. It is observed that plasticity of the soil decreased, and its strength increased with an addition of marble dust. However, plastic strips above the optimum ratio create weak layers and make the soil more susceptible to shear failures along the planes. With the addition of 20% marble dust and 0.5% plastic strips, the UCS value changed from 71.1 kPa for soil alone to 411 kPa, which is a very significant improvement. These trends on the effects of marble dust and plastic strips agree with the previously conducted studies on the effects of waste stabilizers on UCS of stabilized soils [9, 18, 42, 43]. Ashiq et al. [47] found that the unconfined compressive strength increased and reached a maximum value of 152 kPa (43.64% higher than the soil strength) when the content of marble dust content increased to 15%. The resulted high values of UCS may be due to the improvement in cementitious properties of the treated soil as the calcium content of MD

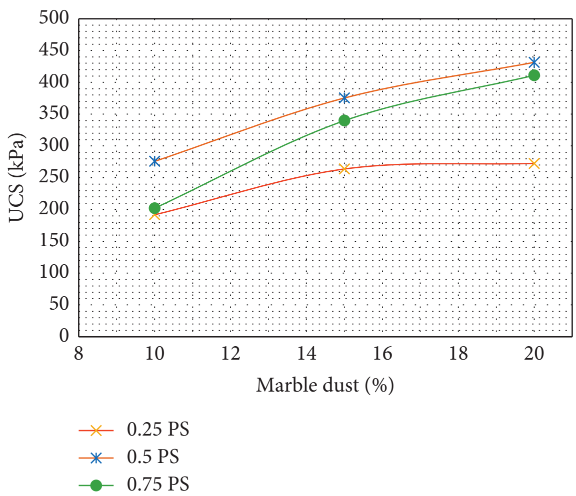


FIGURE 8: UCS value variations with percentages of marble dust and plastic strips.

and silica/alumina content of expansive soil form hydrates. As the amount of marble increases (over 15%), the soil particles are replaced by water and the soil lacks shear strength due to the higher optimum water content, resulting in lower UCS values. Mohammad [12] also found that increasing the percentage of plastic waste from 0.25 to 1.0% resulted in a gradual rise in the strength. A study conducted by [21] found that 0.75% of plastic strips are the optimum content at which a significant change in UCS (142 kPa) obtained with corresponding improvement of 90.13%. The study [48] also described the variation of UCS as a function of plastic strip percentage for three types of soil. It was found that as the UCS increased when plastic strip percentages increase up to 1% beyond which the values of UCS start declining.

4. Conclusion

This research involved the investigation on the modification of subgrade expansive soil using marble dust and plastic wastes. The following conclusions from experimental results are drawn for stabilization of expansive soils with plastic strips and marble dust:

(1) From waste management, environmental pollution reduction, and economic perspectives, using plastic wastes and waste marble dust can save the cost of construction and reduce environmental pollution.

(2) The addition of marble waste increased MDD and decreased OMC, while the addition of plastic strips decreased MDD and MOC. The addition of plastic strips and marble waste to the soil slightly increased MDD and decreased OMC.

(3) As the percentages of plastic strips and marble dust increase, an increase in values of the CBR was observed. The values of CBR change from 1.5 to 6.2% in addition to 20% marble dust and 0.75% plastic strips. The CBR swell of the expansive soil decreases with the addition of marble dust and plastic strips. It changed from 8.19 to 2.31%, with the addition of 20% marble waste dust and 0.75% plastic strips.

(4) The unconfined compressive strength value of the expansive soil increased when marble dust was added. However, when plastic strips were added, the UCS increased to 0.5% and then started to decrease.

(5) The free swell of the expansive soil is reduced by the addition of marble and plastic strips. The expansiveness of the soil changed from a highly expansive category to a low expansive category in addition to plastic strips and marble dust [6, 13, 14, 29–38, 44, 45].

Data Availability

The data used in the study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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