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Research Article

Experimental Study on the Utilization of Fine Steel Slag on Stabilizing High Plastic Subgrade Soil

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The three major steel manufacturing factories in Jordan dump their byproduct, steel slag, randomly in open areas, which causes many environmental hazardous problems. This study intended to explore the effectiveness of using fine steel slag aggregate (FSSA) in improving the geotechnical properties of high plastic subgrade soil. First soil and fine steel slag mechanical and engineering properties were evaluating. Then 0%, 5%, 10%, 15%, 20%, and 25% dry weight of soil of fine steel slag (FSSA) were added and mixed into the prepared soil samples. The effectiveness of the FSSA was judged by the improvement in consistency limits, compaction, free swell, unconfined compression strength, and California bearing ratio (CBR). From the test results, it is observed that 20% FSSA additives will reduce plasticity index and free swell by 26.3% and 58.3%, respectively. Furthermore, 20% FSSA additives will increase the unconfined compressive strength, maximum dry density, and CBR value by 100%, 6.9%, and 154%. By conclusion FSSA had a positive effect on the geotechnical properties of the soil and it can be used as admixture in proving geotechnical characteristics of subgrade soil, not only solving the waste disposal problem.

1. Introduction

The byproduct of steel manufacturing in Jordan, steel slag, is dumped randomly in open areas, which causes many environmental hazardous problems. The three steel factories in Jordan are daily generating from 15–20 tons of steel slag. Most of the steel slag production in Jordan is utilized in the cement industry and has never been used in any other fields due to the lack of research in these fields [1].

It is recognized that swelling of expansive soils may cause significant distress and severe damage to overlying structures. Documented evidence of extensive damage caused by soil expansion is available from different countries in the world. In some locations, the estimated damage cost attributed to soil expansion exceeds the cost of damages from natural disasters such as floods, tornadoes, hurricanes, and earthquakes [2]. Expansive soils are thought to be the main cause of problems in light structures [3]. A total of $15 billion worth of damage is caused annually by expansive soil problems in the United States alone [4]. Similar levels of damage have also been reported in other countries [2, 5–10]. Because expansive soils damage engineering structures, extensive studies on using additives to improve these soils have been performed [11, 12]. The stabilizers of soils are categorized into two main groups as traditional and nontraditional stabilizers. Traditional stabilizers such as limestone, cement, zeolite, gypsum, industrial wastes, and fly ash are commonly used, as reported in the extensive studies by researchers [13–20], while the nontraditional stabilizer (chemical stabilizer) is usually sold as concentrated liquids diluted with water on the project site and sprayed on the soil to be treated before compaction.

Steel slag, a byproduct of steel manufacturing, is produced during the separation of molten steel from impurities in steel-making furnaces. The slag evolves as a molten liquid and is composed of a complex solution of silicates and oxides that solidifies upon cooling. Steel slag is a recycled material that can be useful in the construction industry. For example, in 2002, 50 million metric tons of steel slag was estimated to be produced worldwide [21] and 12 million tons was estimated to be produced in Europe [22]. Currently, the world annual production of steel slag is estimated to range between 90 and 135 million metric tons of steel slag. Approximately 15 to 40% of the 10–15 million metric tons of steel slag generated in the United States in 2006 was not utilized [23].

Mainly as a granular road base or as an aggregate in construction applications, steel slag aggregate (SSA) has been successfully used in the Middle East under hot weather conditions [24]. Not all types of slag are suitable for processing as SSA some have high percentages of free lime and magnesium oxides that have not reacted with the silicate structures and can hydrate and expand in humid environments [25]. Suitable SSA can be used as a replacement for normal aggregate in a variety of civil engineering applications. It can be used in concrete mixes, asphalt concrete (AC) mixes, and soil stabilization. Akinwumi [26] conducted a laboratory investigation on the stabilization of lateritic soil with steel slag. The addition of steel slag resulted in an increase in the specific gravity of the soil, reduction in liquid and plastic limits, and plasticity index. Compaction characteristics were altered due to the addition of steel slag, with increase in dry density and reduction in optimum moisture content. Steel slag resulted in an increase in the unsoaked and soaked CBR, the unconfined compressive strength, and permeability of the soil. The swell potential of the soil steadily reduced with the addition of steel slag. Akinwumi [26] concluded that steel slag can be used as a low cost soil modifier for use in subgrade stabilization. Celik and Nalbantoglu [27] studied the effect of ground granulated blast furnace slag (GGBS) on the control of swell associated with lime stabilized sulphate bearing soil. In order to study the effect of swelling associated with lime stabilization of sulphate-rich soils, three different concentrations of sulphate were chosen, namely, 2000, 5000, and 10,000 ppm. The compaction characteristics, Atterberg limits, linear shrinkage, and swell potential of sulphate dosed 5% lime stabilized soil were then investigated. The same tests were repeated on the combinations but with 6% GGBS as an additive. The test results revealed that the presence of sulphate in soil resulted in abnormal plasticity and swell potential of the soil. At 10,000 ppm sulphate concentration, the swell potential of the lime stabilized soil was three times higher than the natural soil. However, on addition of 6% (GGBS), the swell potential of lime stabilized soil reduced about 87.5% for 10,000 ppm sulphate concentration. In contrast, there was no swelling at all for 5000 ppm sulphate concentration. Hence, this suggested that addition of GGBS to lime results in effective control of swell associated with ettringite formation in sulphate bearing soils.

Yadu and Tripathi [28] presented that stabilization of soft soil can be improved by the addition of blast furnace slag and fly ash. According to that study, it was concluded that, by the addition of slag waste and fly Ash at different proportion, the properties of the soft soil may get changed. It has also been observed that there is an improvement in the strength characteristics of soft soil. Obuzor et al. [29] evaluated the performance of lime activated GGBS in stabilizing road pavements and embankments constructed in flood plains that is prone to submerged conditions due to flooding. Laboratory simulated flooding conditions were used to gauge the performance of stabilized soil specimens of size of 50 mm × 100 mm. The samples were immersed in water for periods of 4 and 10 days after periods of 7, 14, 28, 56, and 90 days of curing.

The specimens were subjected to durability index and UCC strength tests. The samples were prepared with a maximum stabilizer dosage of 16% and five different combinations of lime and GGBS were adopted with GGBS replacing lime in increments of 4% in each successive combination. The samples were molded at three different moisture contents at their MDD to study the effect of placement water content. The investigation revealed that 4% lime with 12% GGBS produced the highest strength and durability out of all the combinations. The strength of the stabilized soil increased with decrease in lime content and increase in GGBS content in the mix, thereby giving a clear indication of better performance of lime-industrial waste combinations when compared to pure lime or pure industrial waste stabilization. It is evident that strength of lime-clay systems was hugely dependent on the GGBS component which increases the density and permeability of the system by forming cementitious gels.

Obuzor et al. [30] investigated the durability of flooded low capacity soil by treating it with lime and GGBS. The investigation involved preparation of test specimens of 50 mm diameter and 100 mm height, statically compacted to their MDD and OMC, followed by moist curing and simulated flooding of the samples. Water absorption during flooding was measured followed by testing of UCC strength of the samples. It was found that higher lime content resulted in greater water absorption. The addition of GGBS, however, resulted in a reduction in moisture absorption and increase in the strength of the flooded samples. It was determined that the addition of GGBS resulted in the reduction in resource consumption and improved robustness of the roads.

Rao and Sridevi [31] performed a laboratory evaluation on utilization of industrial waste in pavement laid over expansive clay subgrades. The waste materials tested were granulated blast furnace slag and fly ash. Detailed laboratory studies have been carried out using these materials for cushioning soil system. The results indicate a significant increase in the soaked CBR value. This investigation points to the utility of these two waste materials for use in subbase of flexible pavement.

Moreover, a number of trial road sections with slag in unbound base course were constructed, while a comparison research carried out between layers containing steel slag as an aggregate and layers with crushed stone [22]. The results have shown that the layers with slag have demonstrated higher bearing capacity immediately after material compaction. Furthermore, the increase in the strength was explained by carbonate hardening due to free oxides of calcium presented in the slag materials.

The main focus of the current study is to evaluate the effectiveness of added FSSA to stabilize and enhance the performance characteristics of the medium-plastic subgrade soil.

2. Materials and Methods

2.1. Soil. The study soil sample used in this research was obtained from Irbid city in the northern part of Jordan. To characterize the studied soil sample, grain size distribution (Sieve analysis, hydrometer analysis) according ASTM D

Table 1: Physical characteristics and Atterberg limits of the considered soil.

Property	Soil
Gravel (%)	0.0
Sand (%)	11.6
Silt (%)	50.1
Clay (%)	38.3
Liquid limit (%)	62.4
Plastic limit (%)	28.5
Plasticity index (%)	33.9
Specific gravity	2.72
Activity	0.89

Table 2: Physical characteristics of fine steel slag aggregate.

Physical characteristics	Value
Gravel (%)	0.0
Sand (%)	96.2
Silt (%)	3.8
Clay (%)	...
Liquid limit (%)	Nonplastic
Coefficient of uniformity (cu)	9.0
Coefficient of curvature (Cc)	1.78
Specific gravity	3.205
Angularity (%)	58
Absorption (%)	4.5

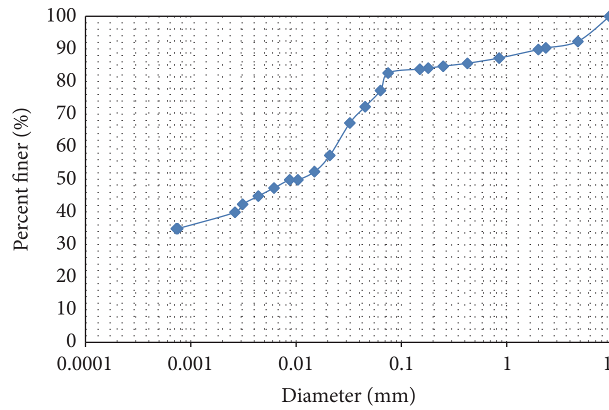

Figure 1: Grain size distribution of the soil.

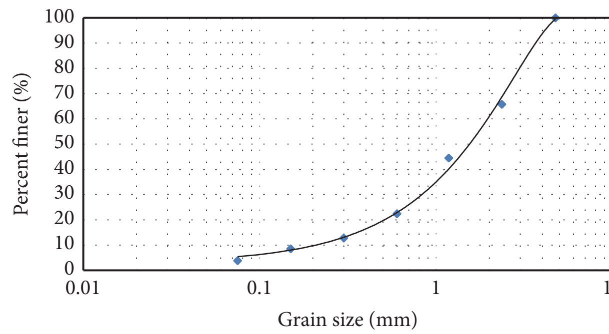

Figure 2: Grain size distribution of the fine steel slag.

Table 3: Chemical composition of fine steel slag aggregate.

Chemical characteristics	Cr	Ni	Fe	Zn	Pb	Cu	Cd
Value (%)	0.063	0.004	0.019	0.021	0.0	0.0	0.0

422-2007, Atterberg limits according to ASTM D 4318-10 and specific gravity according to ASTM D 854-14 tests were performed. Soil sampling, preparing, and testing were done according to ASTM [32] standard methods. Based on test results, the sample was classified as high plasticity clay (CH) according to the Unified Soil Classification System (USCS) (ASTM D 2487-10). The grain size distribution of the soil sample is presented in Figure 1 and soil physical properties are summarized in Table 1. The soil consists of quartz as a major mineral constitute with smectite as a minor mineral and trace amounts of the minerals illite, calcite, dolomite, and kaolinite.

2.2. Fine Steel Slag Aggregates. Fine steel slag aggregate (FSSA) was obtained from the United Iron and Steel Manufacturing Company, Amman. FSSA passing the diameter 4.75 mm (sieve #4) was used in this study. Table 2 shows the physical fine steel slag properties. The chemical tests show that the aggregates were free of cadmium (Cd) and copper (Cu) elements, as shown in Table 3. Figure 2 shows the grain size distribution of the used aggregate. According to the Unified Soil Classification System (USCS) (ASTM D 2487-10), the steel slag is classified as SW (well graded sand).

2.3. Sample Preparation. The soil sample used in this research was dried at 105°C in a drying oven and then passed through sieve number 4 (4.75 mm in diameter) to obtain a uniform distribution. Fine steel slags with amount of 0, 5, 10, 15, 20, and 25% by dry weight of the soil were added and mixed with dry soil to obtain a homogeneous mixture. The standard proctor compaction test according ASTM D-698, 1994, was done for all FSSA mixed sample, to obtain the optimum water content and maximum dry density values which needed to prepare the samples for use in free swell and unconfined compressive strength tests for each FSSA additive percentage. Table 4 shows the mix proportions used in preparing the samples.

2.4. Atterberg Limits. Each soil-additive mixture was passed through sieve number 40 (0.425 mm in diameter) and then liquid limit (LL) and plastic limit (PL) tests were done according to ASTM D 4318-00, 1994. Liquid limit (LL) is defined as the water content, in percent, at which a part of soil is placed in a standard cup and then cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2 in.) when subjected to 25 shocks from the cup being dropped 10 mm in a standard

Table 4: Mix proportions used in preparing samples.

FSSA (%)	Max. dry density (g/cm^3)	Optimum water content (%)	Dry soil mass (g)	FSSA (g)	Required water (g)	Final mix proportion
0.0	1.622	16.9	1000	0.0	169	1000 g (dry soil) + 0.0 g (FSSA) + 169 g (water)
5.0	1.641	15.1	1000	50	151	950 g (dry soil) + 5.0 g (FSSA) + 151 g (water)
10.0	1.648	13.7	1000	100	137	900 g (dry soil) + 100 g (FSSA) + 137 g (water)
15.0	1.692	13.1	1000	150	131	850 g (dry soil) + 150 g (FSSA) + 131 g (water)
20.0	1.734	12.8	1000	200	128	800 g (dry soil) + 200 g (FSSA) + 128 g (water)
25.0	1.766	12.2	1000	250	122	750 g (dry soil) + 250 g (FSSA) + 122 g (water)

Casagrande liquid limit apparatus operated at a rate of two shocks per second. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm (1/8 in.) diameter threads without crumbling.

2.5. Free Swell Tests. The procedure used to perform the free swell tests is based on the procedure recommended by ASTM D 4829 standard. All the test specimens are compacted to maximum dry density and optimum water content using a mold of 70 mm diameter and 20 mm height. After 24 hours of curing under a 7 kPa pressure. The soil specimen starts swelling and vertical displacements are recorded until the expansion is completed and no vertical movement is observed. Vertical displacements are plotted against time. Free swell ratio or free swell percentage is defined as the ratio between the initial and final height of the sample.

2.6. Unconfined Compressive Strength Test. The unconfined compressive strength test method was used to evaluate the shear strength parameters of the samples with additives. The samples were molded in stainless steel tubes of 76 mm height and 38 mm diameter and compressed to the desired compaction characteristics of each additive level. The samples were removed from the tubes and tested directly with a rate 1 mm/min. The unconfined compressive strength test was performed in according to ASTM D2166/D2166M-13 standards.

2.7. California Bearing Ratio (CBR). The CBR test method is used to evaluate the potential strength and bearing capacity of a subgrade soil, subbase, and base course material for use in road and airfield pavements. Two samples are usually prepared for CBR tests; one is tested directly after sample preparation, to simulate the normal field conditions, and the other after soaking in water for 96 hours to simulate the worst conditions in the field. According to ASTM D-1883-99, the test is carried out under a seating pressure of 4.5 kg and a penetration speed of 1.27 mm/sec. The CBR specimens are prepared by a standard mold with an internal diameter of 152.4 mm (6 inches) and a height of 177.8 mm (7 inches). In this study, specimens were compacted at the optimum moisture content determined by standard proctor tests. Three specimens were compacted in 5 layers using 10 blows, 25 blows, and 75 blows, as recommended by ASTM D-1883-99, part 7.2, and then tested without soaking in water. The CBR value is calculated according to the following formula:

$$\mathrm{CBR} = \frac{X}{Y} \times 100\%, \tag{1}$$

where X is material resistance or the unit load on piston (pressure) for 2.54 mm or 5.08 mm of penetration and Y is standard pressure for well graded crushed stone equal to 6.9 MPa for 2.54 mm or 10.3 MPa for 5.08 mm. The CBR values were determined at 2.5 mm and 5.08 mm penetration. If the value of CBR at 5.08 mm penetration is greater, the test was repeated, and if the checked test gives a similar result, then the CBR value at 5.08 mm penetration was used.

3. Test Results and Discussion

3.1. Atterberg Limits. Liquid limit and plastic limit tests were conducted on each sample prepared with 0, 5, 10, 15, 20, and 25% of the FSSA additive. The liquid limits (LL), plastic limits (PL), and plasticity indices (PI) are shown in Figure 3. Figure 3 shows that the liquid and plastic limits of the samples decreased as FSSA content increased. The decrease in the liquid limit is larger than the decrease in the plastic limit; for this reason, the plasticity index decreased as the additive content increased. For example, at 20% additive level the soil will be classified as CL (low plasticity clay), according to the

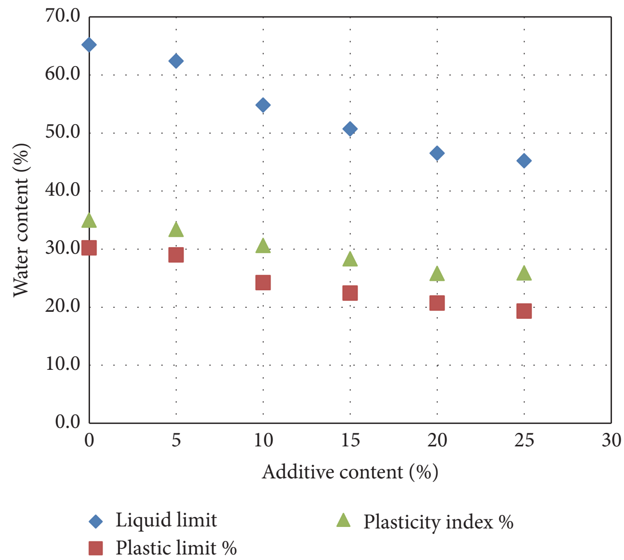

Figure 3: Variation of Atterberg limits with FSSA contents.

Unified Soil Classification System (USCS) (ASTM D 2487-10). The reduction in liquid and plastic limits and plasticity index with increases in slag content is due to increase in the amount of the sand-size particles in the mixture (FSSA is well graded sand) and reduction of the amount of clay-size particles in the soil sample used in this study.

3.2. Compaction Test. Standard proctor compaction (ASTM D-698, 1994) was conducted on the samples with different percentage of FSSA content (0, 5, 10, 15, 20, and 25%). The compaction curves of samples with and without the additive of FSSA are presented in Figure 4. From Figure 4, adding FSSA to the soil increased the maximum dry density and decreased the optimum water content for the same compactive effort. This increase in maximum dry density of treated soil sample occurred since FSSA has greater specific gravity than the studied soil. Moreover, the exposure of FSSA to local weather for more than one year resulted in the hydration of the lime content of FSSA. This has led to the reduction of optimum moisture content of the treated soil sample with increasing FSSA content. Also sand-size particles in the mixture (FSSA is well graded sand) required less amount of water to reach the optimum moisture content.

3.3. Free Swell Percentage. The soil used in this research is classified as CH soil and consists of quartz as a major mineral constitute with smectite as a minor mineral and trace amounts of the minerals illite, calcite, dolomite, and kaolinite. These clay minerals, which have negatively charged surfaces, attract to the positive ions in the void by electrostatic attraction, leading to a concentration of ions near the diffuse double layer (DDL). The intersection or overlap of DDLs causes repulsive forces to arise between the particles; these forces cause the free swelling [33]. According to Chen [2] the studied soil was classified as high swell pressure since the plasticity index ranged from 29 to 35%. Figure 5 shows the relation between the free swell percentage and the time for different FSSA contents. Table 5 shows that the free swell percentage decreases as the FSSA content increases. This reduction is due to reduction in plasticity of soil mixture with FSSA.

Table 5: Effect of the FSSA additive content on the free swell.

FSSA content (%)	0.0	5.0	10.0	15.0	20.0	25.0
Free swell amount (%)	5.15	4.80	4.60	3.00	2.15	1.65

3.4. Unconfined Compressive Strength. The variation in unconfined compressive strength with FSSA content is shown in Figure 6 and Table 6. From Table 6 the unconfined compressive strength increased with increasing FSSA content from 142.95 kPa for 0% content to 310.12 kPa for 20% FSSA content and reduction in strain at failure from 7.24% for 0% to 3.42% for 20% FSSA content. The increase in unconfined compressive strength from 142.95 kPa to 310.12 kPa is due to increase in maximum dry density and reduction of optimum water content. The reduction in the unconfined compressive strength shear strength to 285.11 kPa for 25% FSSA content shows that the limit effect of the solidification had been reached.

3.5. California Bearing Ratio (CBR)

3.5.1. CBR of Soil without FSSA Content. In this study, three samples were prepared for CBR tests at the same moisture content. Each sample was compacted with a different number of blows (10, 25, and 75) as recommended by ASTM D-1883-99, part 7.2, note 3, since the maximum dry density was determined from compaction in the 4 in. (101.6 mm)

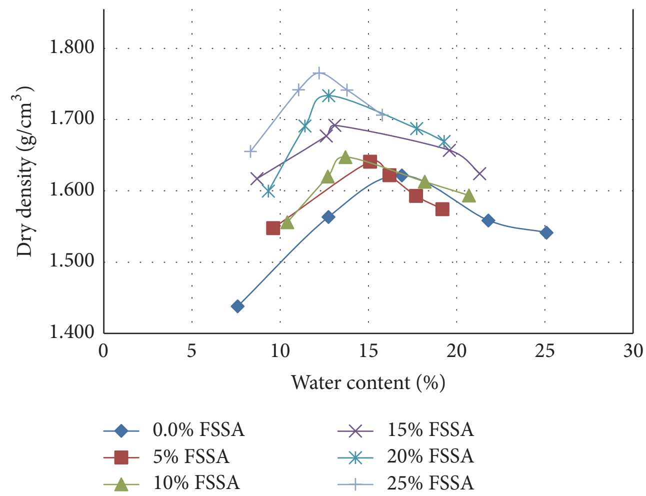

FIGURE 4: Compaction curves of samples with the FSSA additive.

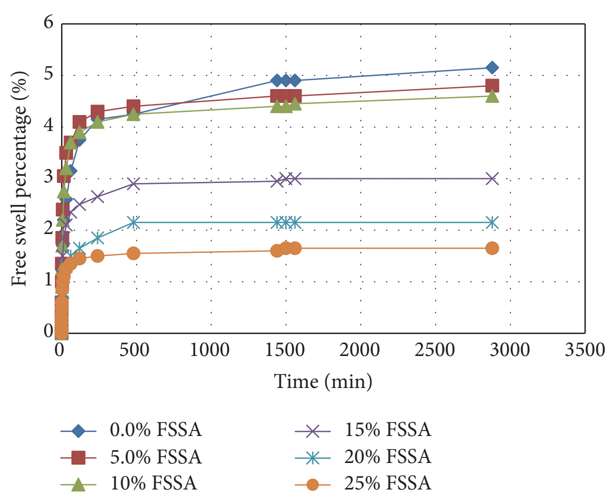

FIGURE 5: Free swell percentage versus time for samples with different FSSA contents.

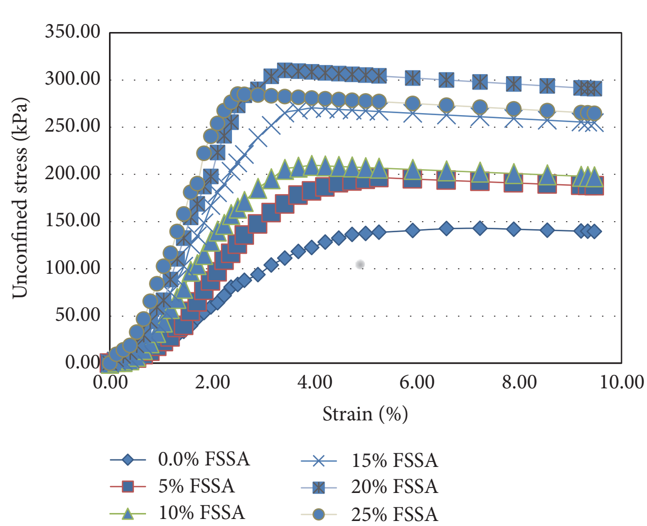

FIGURE 6: Stress shear strength versus strain for samples with different FSSA contents.

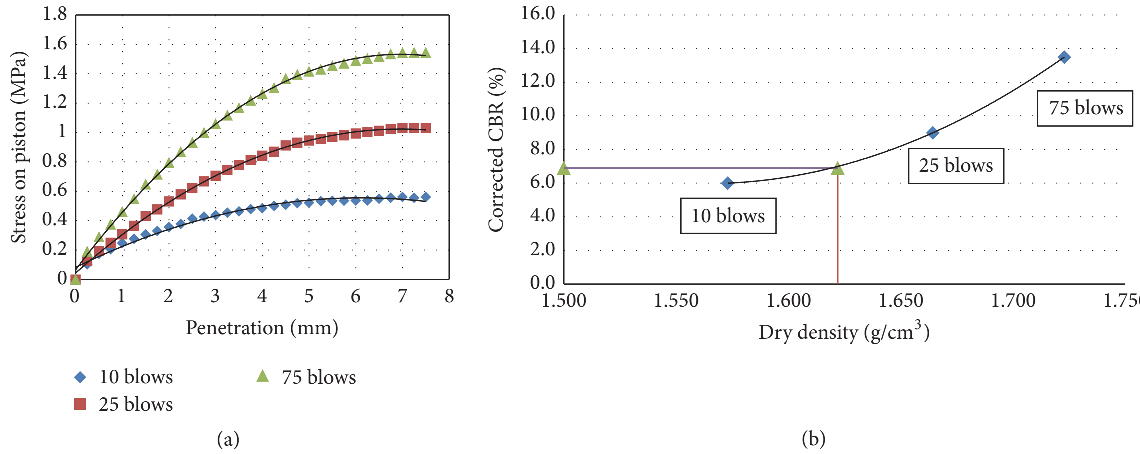

Figure 7: (a) Relationship between resistance and penetration for soil without additives. (b) CBR value for soil without additives.

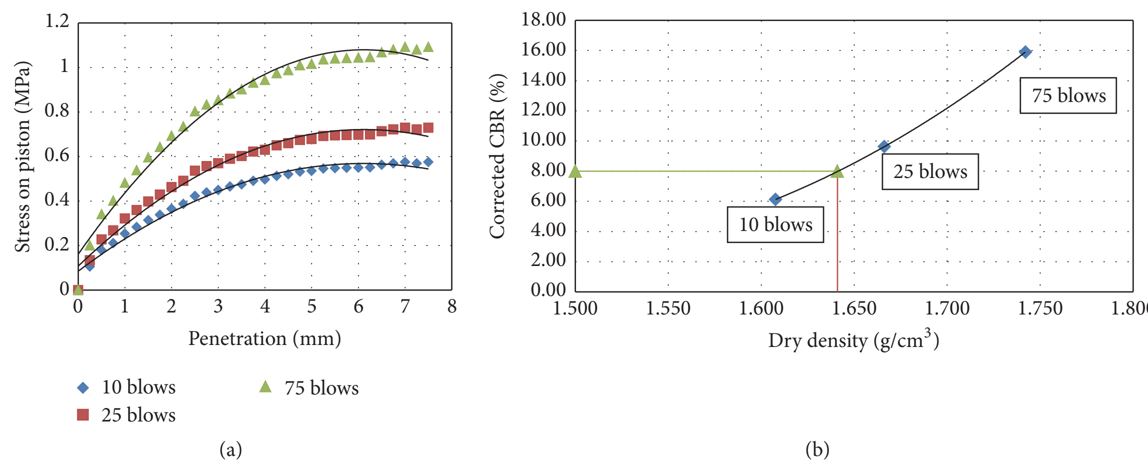

Figure 8: (a) Relationship between resistance and penetration for soil with 5% FSSA additives. (b) CBR value for soil with 5% FSSA additives.

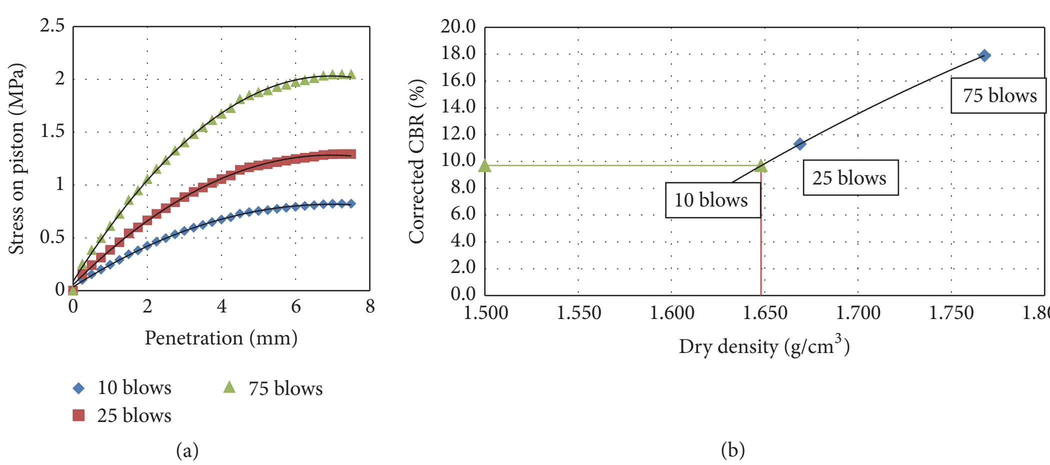

Figure 9: (a) Relationship between resistance and penetration for soil with 10% FSSA additives. (b) CBR value for soil with 10% FSSA additives.

Table 6: Effect of the FSSA additive content on the unconfined compressive strength and strain at failure.

FSSA content (%)	0.0	5.0	10.0	15.0	20.0	25.0
Unconfined compressive strength (kPa)	142.95	196.41	209.54	270.30	310.12	285.11
Strain at failure (%)	7.24	5.26	3.95	3.68	3.42	2.50

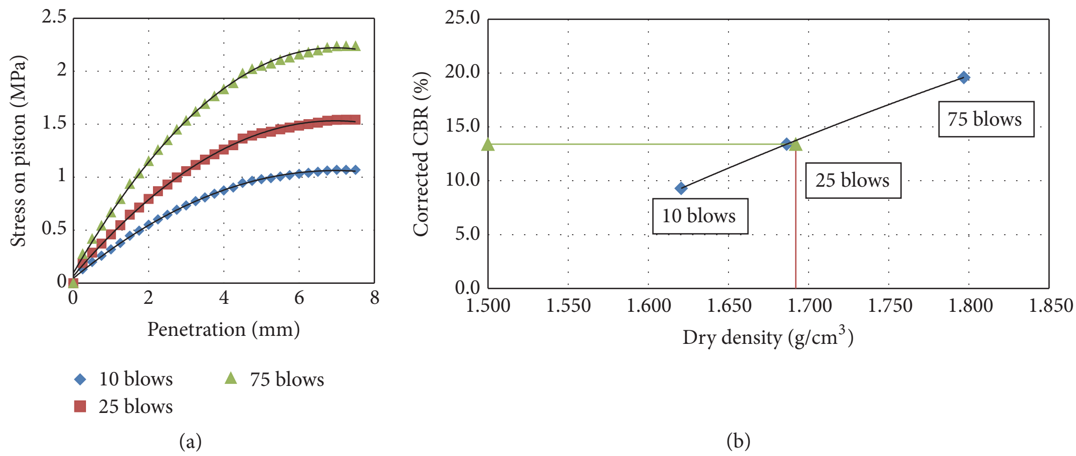

FIGURE 10: (a) Relationship between resistance and penetration for soil with 15% FSSA additives. (b) CBR value for soil with 15% FSSA additives.

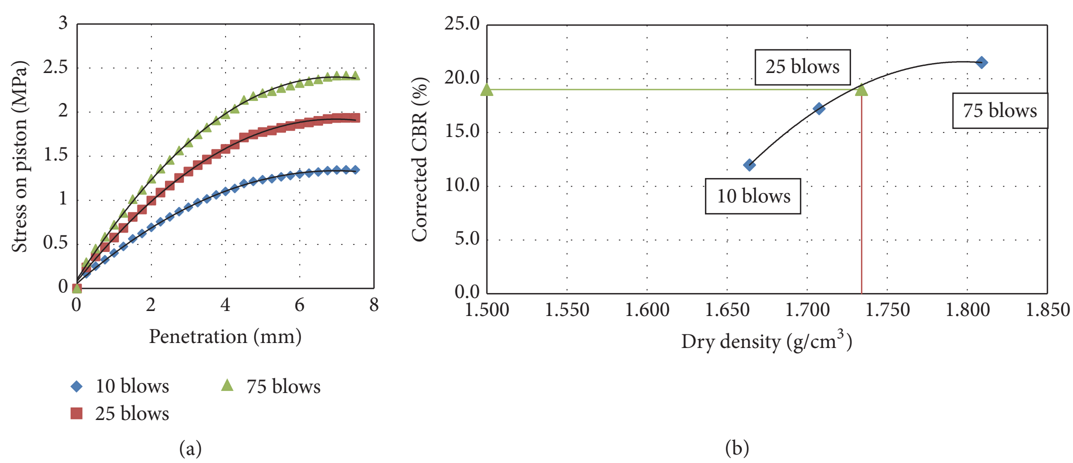

FIGURE 11: (a) Relationship between resistance and penetration for soil with 20% FSSA additives. (b) CBR value for soil with 20% FSSA additives.

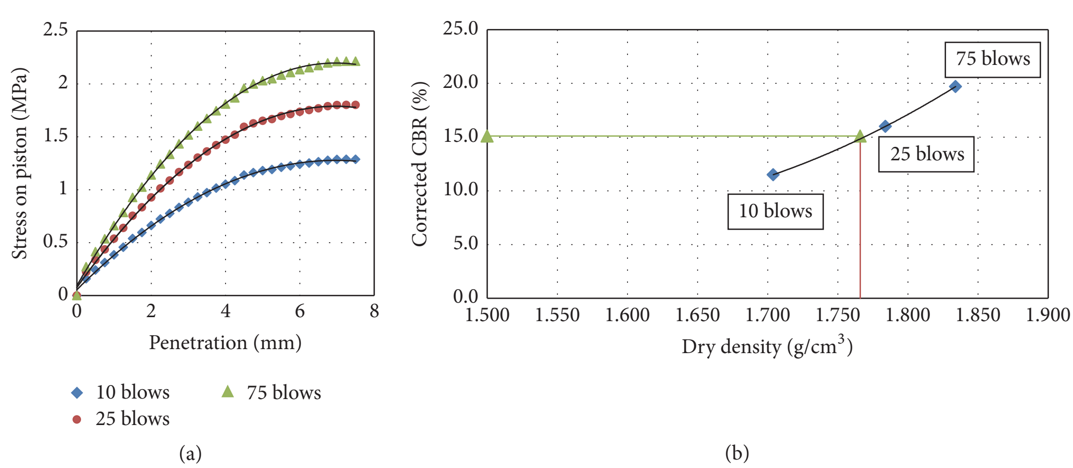

FIGURE 12: (a) Relationship between resistance and penetration for soil with 25% FSSA additives. (b) CBR value for soil with 25% FSSA additives.

Table 7: CBR values of the three samples of soil without additives.

Number of blows	Water content (%)	Dry density (g/cm^3)	CBR (%)
10	16.9	1.573	6.0
25	16.9	1.664	9.1
75	16.9	1.723	13.5

Table 8: CBR values of the three compaction efforts samples of soil with FSSA additives.

FSSA content (%)	Number of blows	Water content (%)	Dry density (g/cm^3)	CBR (%)
5.0	10	15.1	1.6080	6.2
	25	15.1	1.666	9.6
	75	15.1	1.742	15.9
10.0	10	13.7	1.618	7.2
	25	13.7	1.669	11.3
	75	13.7	1.768	17.9
15.0	10	13.1	1.621	9.3
	25	13.1	1.686	13.4
	75	13.1	1.797	19.6
20.0	10	12.8	1.664	12.0
	25	12.8	1.708	16.0
	75	12.8	1.809	20.0
25.0	10	12.2	1.704	11.5
	25	12.2	1.784	16.3
	75	12.2	1.834	19.7

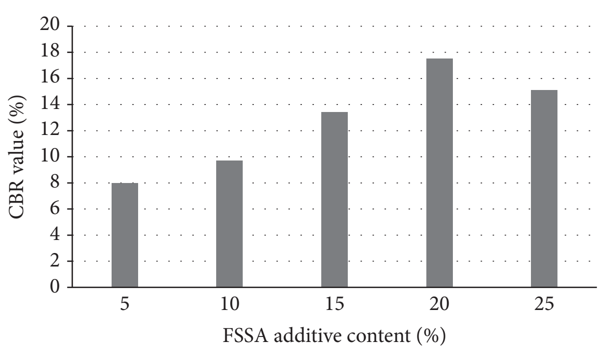

Figure 13: CBR values correspond to maximum dry density for FSSA additive content.

mold; it is necessary to compact specimens, using 75 blows per layer. The CBR tests were conducted directly (without soaking in water). Table 7 presents the CBR test results of the three samples of soil without FSSA content. The relationships between resistance and penetration are shown in Figure 7(a). The relationship between corrected CBR and the dry density is shown in Figure 7(b). The CBR value which corresponds to maximum dry density is determined to be 6.9%.

3.5.2. CBR of Soil with Different Percentage of FSSA Content. Table 8 presents the results of CBR tests for soil with 5%, 10%, 15%, 20%, and 25% of FSSA for three samples, respectively, at different compaction efforts: 10, 25, and 75 blows. The relationships between resistance and penetration are shown in Figures 8(a)–12(a). The relationship between corrected CBR and the dry density is plotted in Figures 8(b)–12(b). The CBR values which correspond to maximum dry density for different FSSA content are shown in Figure 13. From Figure 13 the unsoaked CBR values are increased from 6.9% for 0% FSSA to 17.5% for 20% FSSA and then decreased to 15.1% for 25% FSSA content. This increase is due to increase in maximum dry density and reduction of optimum water content. The reduction in CBR for 25% FSSA content shows that the limit effect of the solidification had been reached.

3.6. Conclusions. This research focuses on the effect of stabilizing high plastic subgrade soil by adding FSSA. From the test results, the following conclusions can be drawn:

(1) Liquid limit, the plastic limit, and the plasticity index decrease as FSSA content increases. For example at 20% of FSSA content, the soil changes from high plasticity clay (CH) to low plasticity clay (CL).

(2) As the FSSA increases, the maximum dry density increased and the optimum water content decreased.

(3) The free swell percentage decreases from 5.15% for 0.0% FSSA content to 2.15% for 20% FSSA content. This decreasing in free swell percentage makes the soil more stabilizing from civil engineering view.

(4) The unconfined compressive strength increases and the strain at failure decreases as the FSSA content

was increased up to 20%. The effect of 20% FSSA on compressive strength is more than the other percentage.

(5) The unsoaked CBR value increases from 6.9% for soil with 0.0% FSSA to 17.5% at 20% FSSA content. According to Bowles [34], 20% FSSA content has been converted the fine grained soil from(poor to fair) to good subgrade soil.

Competing Interests

The authors declare that there is no conflict of interests regarding the publication of this paper.

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