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Improving Properties of Highly Expansive Soil Using Waste Materials

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ABSTRACT Expansive clay can be found in arid and semiarid regions. Such expansive soils are the natural threat for civil structures especially when they are lightly loaded. To avoid these circumstances, soil must be stabilized and strength is to be increased. By soil stabilization technique strength properties of soil can be improved by using some kind of admixtures. There is a large amount of wheat straw production from fields and micro silica fume as a by-product from silicon alloy industries. If this waste is utilized for soil stabilization then problem of solid waste can be resolved. In present investigation, as highly expansive soil is not available in Pakistan so it is prepared by mixing 50% of bentonite and 50% of local soil from Sahiwal (Pakistan) fields and a 16% of binary additive (37.5% wheat straw powder and 62.5% micro silica fume) is selected to study the effects of the engineering properties of problematic expansive soil. So in order to utilize the silica fumes and wheat straw powder for the improvement of expansive soil a detailed programmed study have been formulated where major factors like consolidation, swell-shrink behavior and shear strength have been studied. From different experimentation on soil with and without binary additive it was found that soil with binary additive has gained its strength appreciably.

Keywords: soil stabilization, expansive soil, index and engineering properties of soil, silica fumes, bentonite, powder wheat straw.

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1. INTRODUCTION

Expansive soil is also known as clayey soil. It undergoes large volume changes with change in the moisture content. Such soils found in arid and semi-arid areas produce distresses in the lightly loaded infrastructures. As some of the regions of Pakistan fall in the semi-arid zone, such as; Southern and Central Baluchistan, Northern KPK, Sindh and Gilgit Baltistan. Therefore, swelling clays of low plasticity may be found in these areas. These soils are highly plastic in nature due to the clayey minerals like; montmorillonite that absorbs water sufficiently. Also, the swelling and shrinkage are caused due to presence of varieties of minerals, such as, montmorillonite, kaolinite, and illite groups of minerals [1]. Problematic soils often exist in shallow depth with high negative pore-water pressure, resulting in formation of serious geotechnical engineering issues. Common types of such clayey soils are expansive soils, collapsible soils, and residual soils. Such types of soils have been found worldwide that causes distresses in the light weighted infrastructures, that is, buildings, pavements, and slopes, and have been affected in various countries globally. The financial loss due to the expansive soil is comparatively larger than the combined loss of flood, tornadoes, earthquakes, and hurricanes. The major problems happen with expansive soils due to the alteration in moisture content in the shallow depth and lacking in deep layers.

Problems with existing soils are faced by geotechnical engineers. Such soils can be a serious natural hazard, which if not treated can cause extensive damage to structures such as foundations, roads, highways, building, airport runways and earth dams. Damage caused by expansive soils has exceeded the combined average annual damage from floods, cyclone and earthquake [2]. Some remedial measures can be taken to prevent the damages. Such as exchanging the soil with the other soil, controlled well compaction of expansive soil, moisturizing, altering structure of moisture barriers, lime stabilization, cement stabilization, modification of the structure and lowering the foundations from upper layer to the lower level [3].

The soil used in this study is highly plastic. When pavement subgrade is composed of highly plastic soil its bearing capacity is extremely low. Because of this, the roads require timely maintenance to take up repeated application of wheel loads over the pavement. This can be very costly, and the conditions of roads and foundation of structures during monsoon seasons is extremely poor. Therefore, a thought on how to enhance the stability of roads and structures by cheaper means demands appraisal [4]. Soil stabilization can be done using different additives, but the use of micro silica fume and wheat straw powder which are the waste material from silicon alloy industries and fields respectively, at the same time can be an efficient way to utilize such waste materials which are difficult-to-dispose.

2. MATERIALS AND METHADOLOGY

2.1. Materials

2.1.1. Micro Silica Fume (MSF)

Micro silica fume is a secondary product resulting from the reduction of high-purity quartz with coal in electric furnaces during the manufacture of silicon and ferrosilicon alloys. Silica fume is composed of very fine particles of around 100 times smaller than the average of cement particles and a surface area of approximately 20,000 m2/kg when measured by nitrogen absorption techniques [5]. Silica fume is considered as a very effective pozzolanic material due to its high silica content and its extreme fineness [6], [7]. One of its main uses is to improve the concrete properties, by increasing its compressive strength, bonding strength, and abrasion resistance, and reducing its permeability. Therefore, it helps in protection of the reinforced steel from corrosion. The chemical composition of the micro silica fume used in the present study presented in Table 2-1.

Figure 2-1 Micro Silica Fume and Powder Wheat Straw

Table 2-1 Chemical Composition of Local Soil & Bentonite Used

Sr. No.	**Composition**	**Local Soil (%)**	**Bentonite (%)**	**Silica Fume (%)**	**Powdered Wheat Straw (%)**
1	SiO_2	75.10	47.35	94.50	65.30
2	Al_2O_3	3.92	15.84	0.30	4.18
3	Fe_2O_3	2.13	8.05	0.12	1.10
4	CaO	5.53	7.90	0.85	13.72
5	MgO	1.41	2.14	0.35	2.60
6	L.O.I	10.67	14.10	2.35	6.28
7	SO_3	0.32	0.28	0.25	1.52
8	K_2O	-	-	0.08	-
9	Na_2O	-	-	0.63	-

2.1.2. Powdered Wheat Straw (PWS)

The powdered wheat straw (Figure 2-1) was obtained from a local farmer in Sahiwal, Pakistan. The fibers passed through ASTM Sieve # 40 to obtain it in required powder form. Such fibers can be used to increase strength characteristics of soil by using as reinforcing fibers in a cheaper way. But powder form of wheat straw is used in this study in order to fill the voids which can make the soil weaker by filling such voids with water and air and resulted in settlement of ground when load is applied on such surface which can be a major threat to civil infrastructures. Chemical composition of PWS is given in Table 2-1.

2.1.3. Highly Expansive Soil

In the present study, the highly plastic soil used is prepared in the laboratory. The local soil was brought from fields of CUI, Sahiwal and the bentonite was obtained from local market. Figure 2-2 shows the bentonite and local soil sample used. As highly plastic soil has plastic limit greater than 50%, so this laboratory prepared plastic soil has plastic limit 90% as tested by Atterberg's limit test.

Figure 2-2 Bentonite & Local Soil

Chemical compositions are presented in Table 2-1. The Figure 2-3 shows results of free swell index results. Hence, bentonite is found to be 27 times more expansive than the local soil. That is why the bentonite has been added in local soil in order to get an expansive soil of required swelling potential.

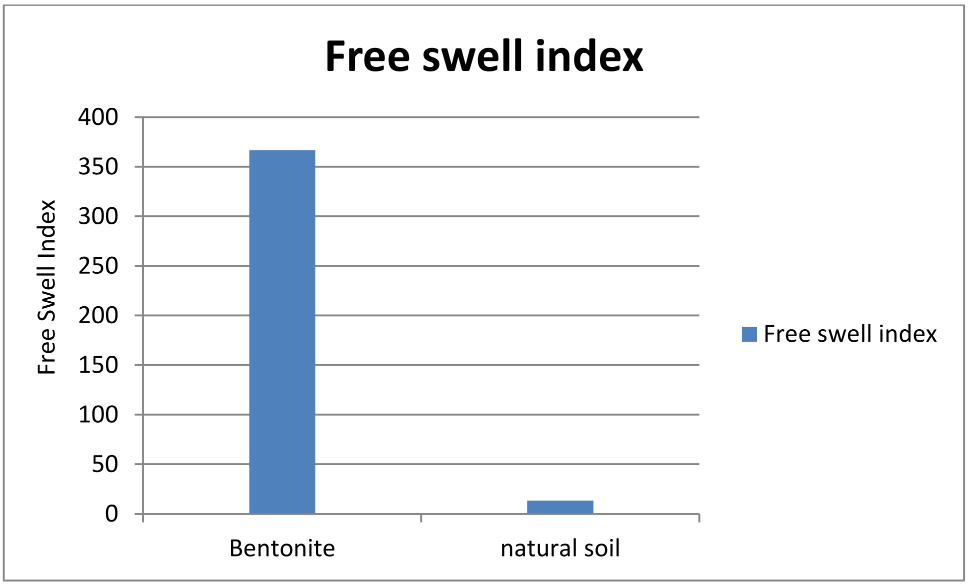

Figure 2-3 Free Swell Index Test Results for Bentonite and & Local Soil

The grain size distribution of the bentonite and local soil are given in Figure 2-4. Below chart shows that the grain size of local soil collected from Sahiwal, Pakistan ranges from 0.075 mm to 4.75 mm and the grain size of bentonite used ranges from 0.008 mm to 4.75 mm.

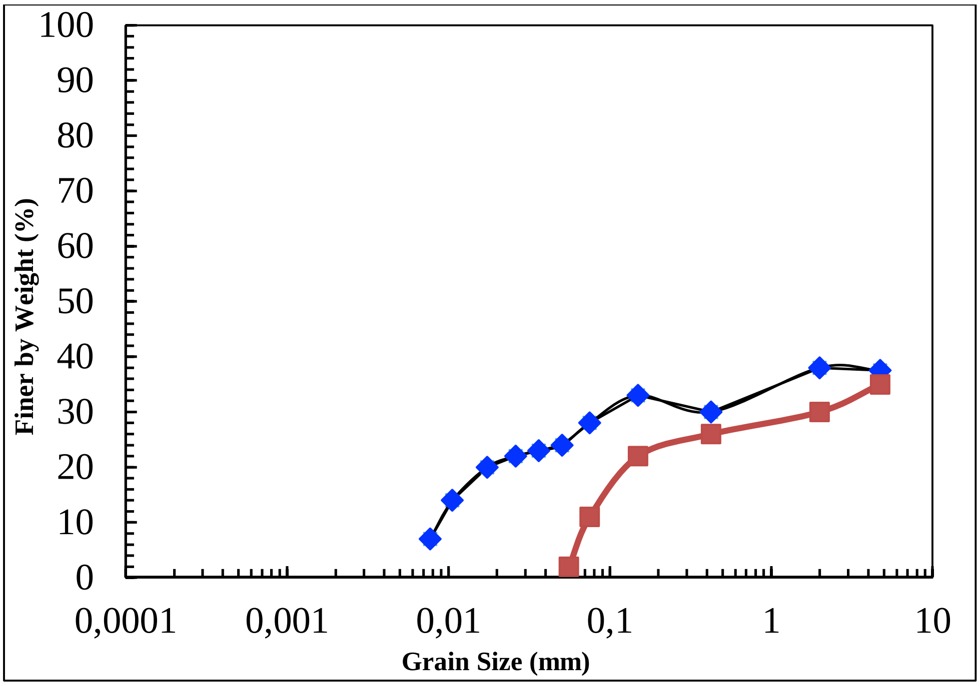

Figure 2-4 Grain size Distribution Chart of Local Soil

2.2. METHODOLOGY

In this research study, in order to observe major factors like consolidation, swell-shrink behavior and shear strength, the specimens are tested by seven major experimentations i.e. Atterberg's Limits, Standard Compaction Test, Swell index Test, Direct Shear Test, Wetting/Drying Cycles, Swell Pressure and Swell Potential Test (Swell Test) and Chemical Analysis (Wet Analysis).

In order to determine the liquid limit, plastic limit of MSF, PWS and BA various percentages of these materials were carried out to achieve maximum output. The results of consistency limits were taken immediately from the tests. Tests were conducted in accordance with ASTM D 4318. Refer to Figure 2-5, 2-6.

This test is performed to determine the relationship between the moisture content and the dry density of a soil at a specified compactive effort in accordance with ASTM 698 by using standard compaction test apparatus. In order to achieve maximum dry density (MDD) at lesser compaction effort, only 16% binary additive was used here. Refer to Figure 2-7, 2-8.

To determine the swelling potential of soil samples under at room temperature in submergence of water. Test was conducted in accordance with IS: 2720 (Part 40)-1985. This test is performed to observe change in the volume of plastic soil with 0% and 16% of BA, without any external impact, on submergence in water. Refer to Figure 2-9, 2-10.

To determine the shear strength parameters of a soil at known density and moisture content. Test was conducted in accordance with IS: 2720 (Part 13)-1986. Where the shear strength parameters like angle of internal friction and cohesion of specimens are obtained with 0 % and 16 % of BA at curing of 0 and 28 days. Refer to Figure 2-11, 2-12.

To determine the effect of wetting and drying cycles on swell/shrink properties of plastic soil specimen. Methods for Wetting and Drying ASTM D559 / D559M – 15-is conducted. Each cycle is of 48 hours where 24 hours is for wetting and 24 hours is for drying. At the end of each cycle average readings of the specimens were taken in order to obtain graphical representations. Refer to Figure 2-13, 2-14.

Volume of plastic soil goes on increasing when they come in contact with water. If the soil is not allowed to expand or its volume change is arrested, the pressure exerted in this case is known as Swelling Pressure of Soil. So measurement of swelling pressure of soils is conducted accordance with IS 2720 (Part 41):1977. Specimens were observed for 05 days using floating ring in consolidometer in saturated condition. Refer to Figure 2-15, 2-16.

This test covers the measurement of moisture content, ash content, and organic material existence in soils, such as organic clays, silts, and mucks. Test Method provides for determining the water (moisture) content in mineral soils and rock. Its Reference is ASTM D2974. Refer to Figure 2-17.

Figure 2-1 Applying Soil in Brass Cup of Cassagrande Apparatus

Figure 2-6 Soil Thread made to find Plastic Limit of soil specimens

Figure 2-2 Standard Compaction Test Apparatus

Figure 2-3 Standard Compaction Test Specimen

Figure 2-4 Specimen for Free Swell Index Test with Different Proportions of Binary Additive

Figure 2-10 Specimens under Observation in Free Swell Index Test

Figure 2-11 Specimen for Direct Shear Test

Figure 2-12 Direct Shear Testing Box

Figure 2-13 Unconfined Test Mold

Figure 2-5 Flexural test mold

Figure 2-6 Oedometer Frame

Figure 2-16 Consolidometer

4. RESULTS AND DISCUSSION

4.1. Selection of Binary Additive from Atterberg's Limits

The optimum values of Atterberg's limit for MSF and PWS were found to be 25% and 15% respectively, as shown in Figure 4-1 and Figure 4-2. The binary additive was finally prepared by adding 62.5% of MSF and 37.5% of PWS. The ratio/amount of these additives was based on above Atterberg's limit results. After getting a binary additive (62.5% MSF and 37.5% PWS) the proportions 0, 4, 8, 12, 16, 20 and 24% of BA were added in expansive soil to obtain highest optimum values of BA in expansive soils.

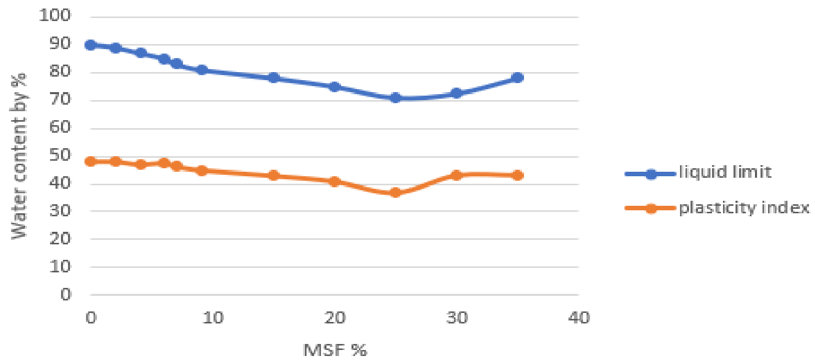

Figure 4-1 Atterberg's Limits of MSF for the Selection of Binary Additive.

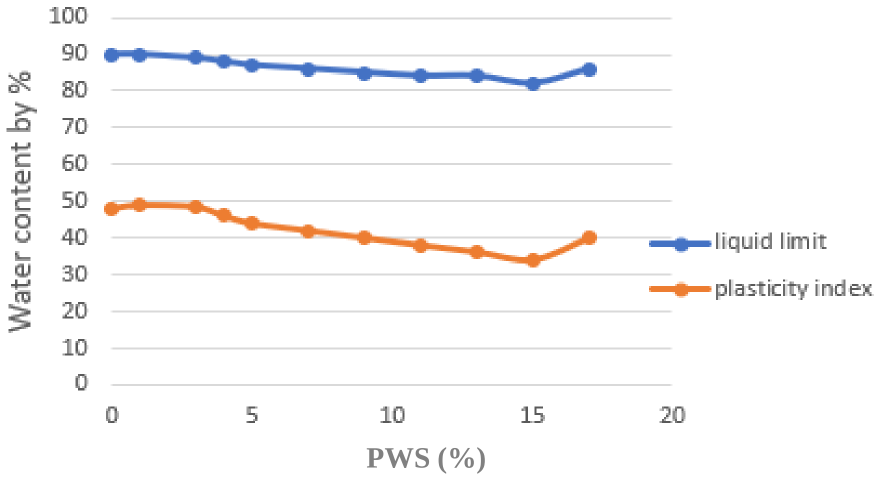

Figure 4-2 Atterberg's Limits of PWS for the Selection of Binary Additive.

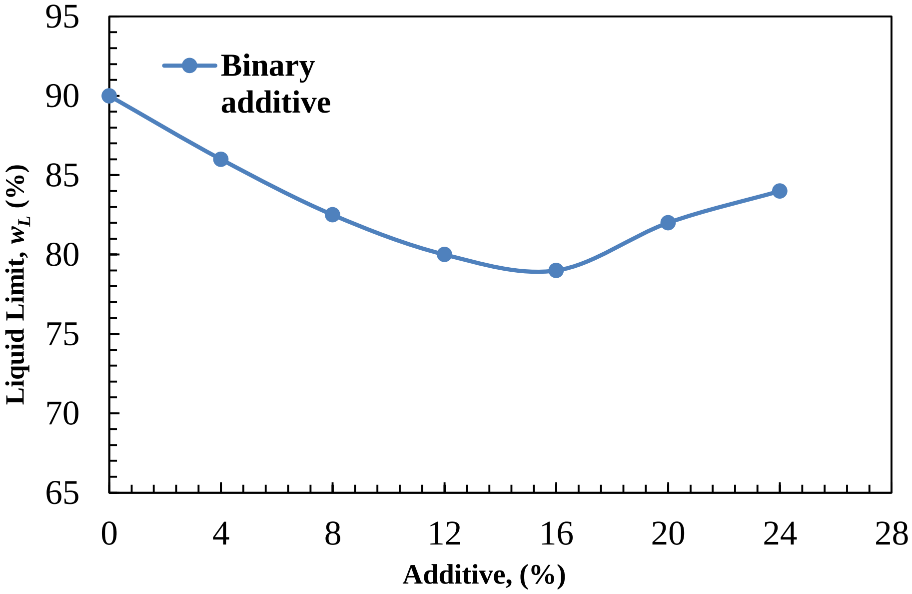

Figure 4-3 Liquid Limit Test Result of Binary Additive Mixed Expansive Soil.

From above Figure 4-3, we can see at 16% of BA, there is a minimum value (79%) of liquid limit. This means there will be less transition of soil from plastic state into liquid state. This factor determines greater shear strength, less permeability, less settlement and less expansive behavior. So, this factor indicates a good behavior of soil with 16% of binary additive.

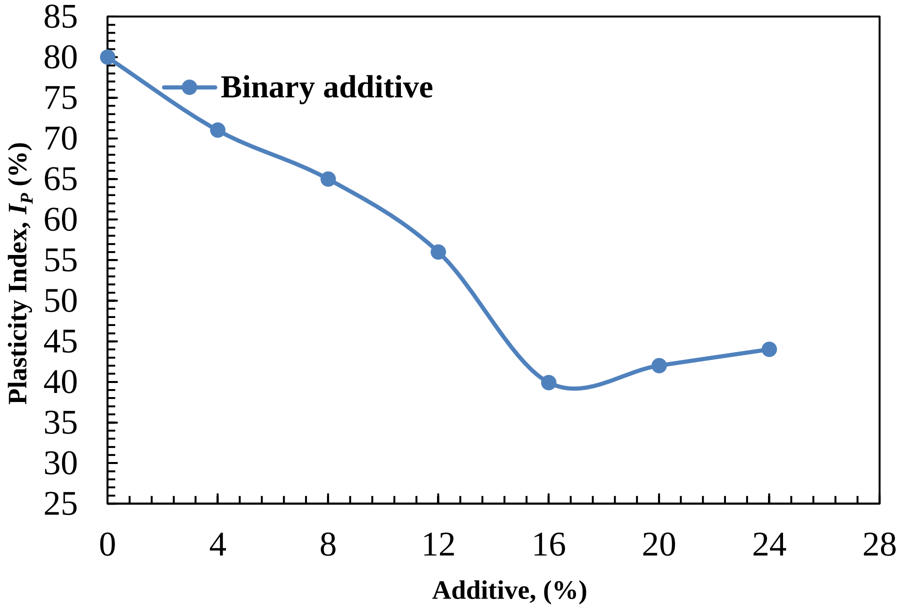

Figure 4-4 Plasticity Index of Binary Additive Mixed Expansive Soil

From above Figure 4-4, we can see at 16% of binary additive, there is a minimum value (39.5%) of plastic limit. This means there will be less transition of soil from plastic state into solid state. This factor determines greater shear strength, less permeability, less settlement and less expansive behavior. So, this factor indicates a good behavior of soil with 16% BA.

From above observations, minimum of Atterberg's Limits were achieved at 16% BA. In this regard, for accomplishing the remaining objectives of the study, only 16% of binary additive is used.

4.2. Standard Compaction Test

The addition of 0% binary additive resulted in 25.46 % of moisture content and a maximum dry density of 15.85 KN/m^3. Though the addition of 16% of additive resulted at 24.16% of moisture content and a maximum dry density of 15.95 KN/m^3 is achieved, as shown in Figure 4-5. Therefore it is perceived that the addition of 16% binary additive is effective to achieve maximum dry density at lower effort. This concludes that there will be less settlement of foundation soil under loading conditions by using binary additives in the expansive soil.

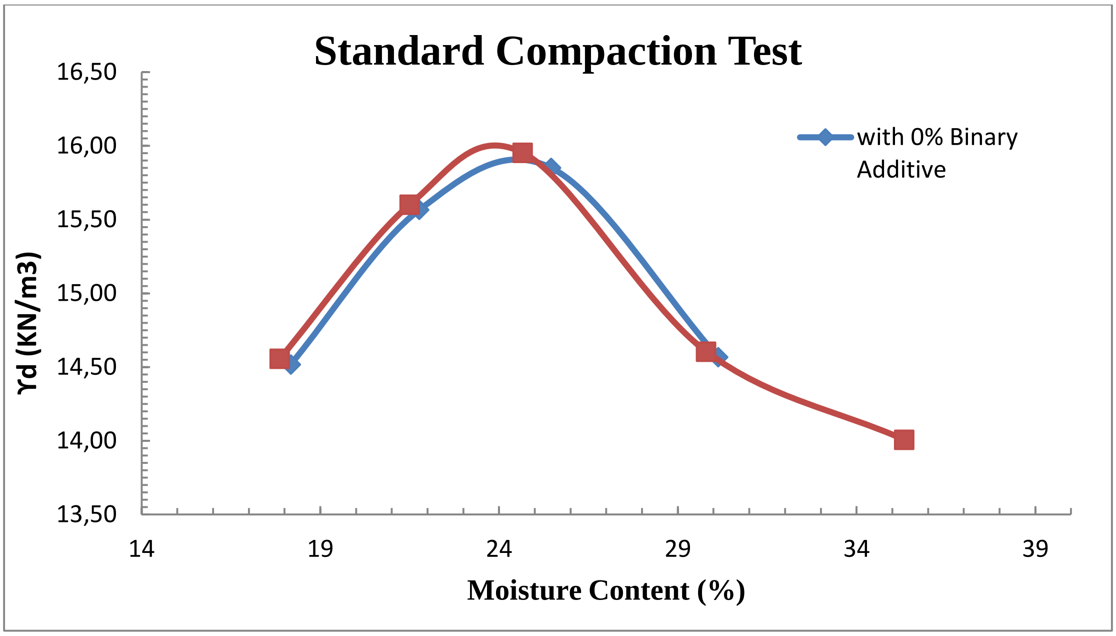

Figure 4-5 MDD/Moisture Content Curve of Standard Compaction Test

4.3. Free Swell Index Test

From Figure 4-6, we can see that as we increase the binary additive in expansive soil, swelling goes on decreasing respectively. As swelling in soil is a major threat to civil structures. Ruptures appear in structures standing on such expansive soil. So in order to reduce such threat this BA proved to be very efficient. As soil with 0% BA the swell index is 143% as well as with the addition of BA the swell index is (128, 118, 109, 91, 80, 62) % for (4, 8, 12, 16, 20 and 24) % respectively.

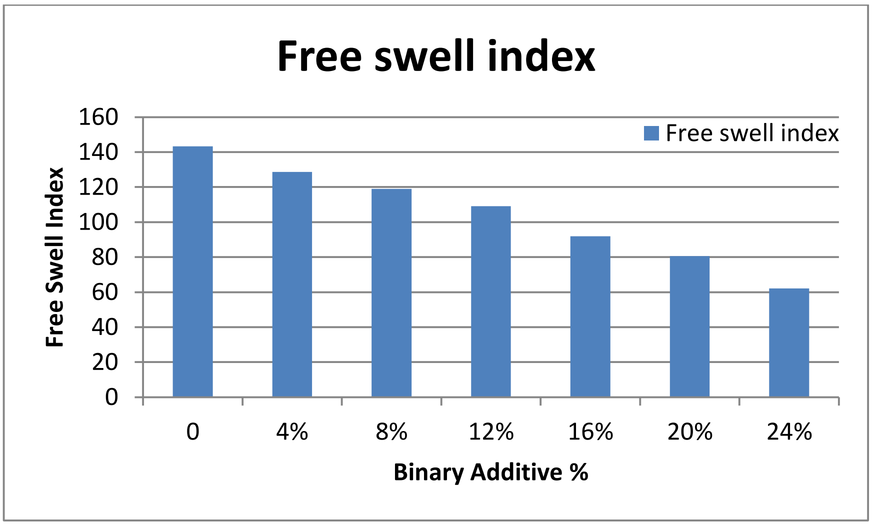

Figure 4-6 Free Swell Index results

4.2. Direct Shear Test Results of Specimen with (0, 16)% Binary Additive

4.2.1. After Zero (0) Days Curing

Given chart represents the shear stress to shear strain changes of soil specimen with 0% and 16% binary additive at 0 curing days. Here 10.2 kg of load is applied on DST machine. Below figure 4-7 shows that specimen with 0% BA at 0 curing day shown a maximum shear strength of 116 kPa and 7.3 % of shear strain produced while on the other hand specimen with 16% BA had shown a maximum shear strength of 160 kPa and shear strain of 6.5 % under the load of 10.2 kg.

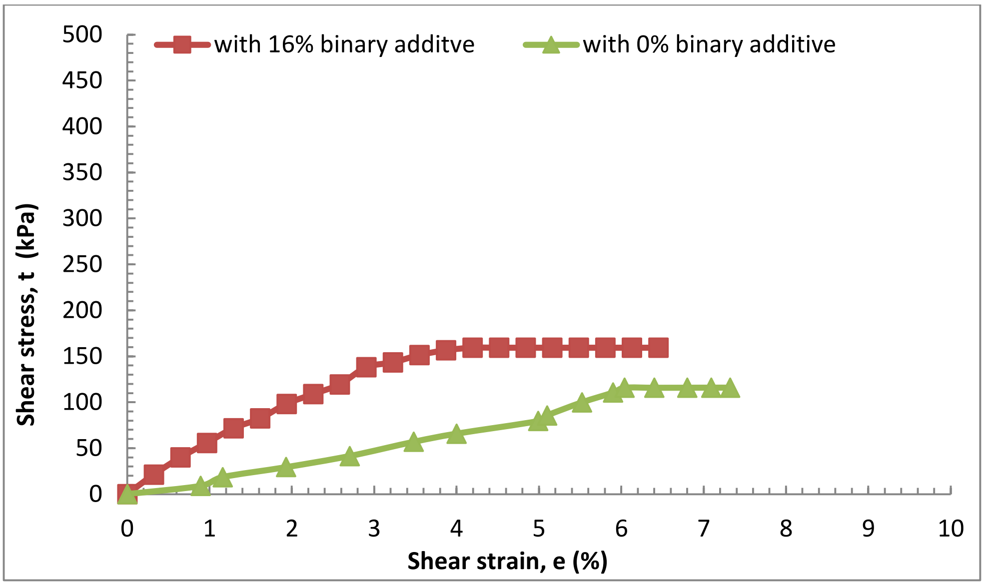

Figure 4-7 Direct Shear Test Result of Specimen with 0% and 16% BA (0 Days Curing)

Table 4-1 Cohesion and Angle of Internal Friction of Specimen with 0% and 16% BA after 0 Days of Curing

	Specimen with 0% Binary Additive	Specimen with 16% Binary Additive
Cohesion (kPa)	85	35.7
Angle of internal friction (Degree)	61	28

4.2.2. After (28) Days Curing

Given Figure 4-8 represents the shear stress to shear strain chart of soil specimen with 0% and 16% additive at 28 curing days. Here 10.2 kg of load is applied on DST machine. Below Figure 4-8 shows that specimen with 0% BA at 28 curing day shown a maximum shear strength of 260.8 kPa and 5.78% of shear strain produced while on the other hand specimen with 16% BA had shown a maximum shear strength of 451 kPa and shear strain of 6.8 % under the load of 10.2 kg.

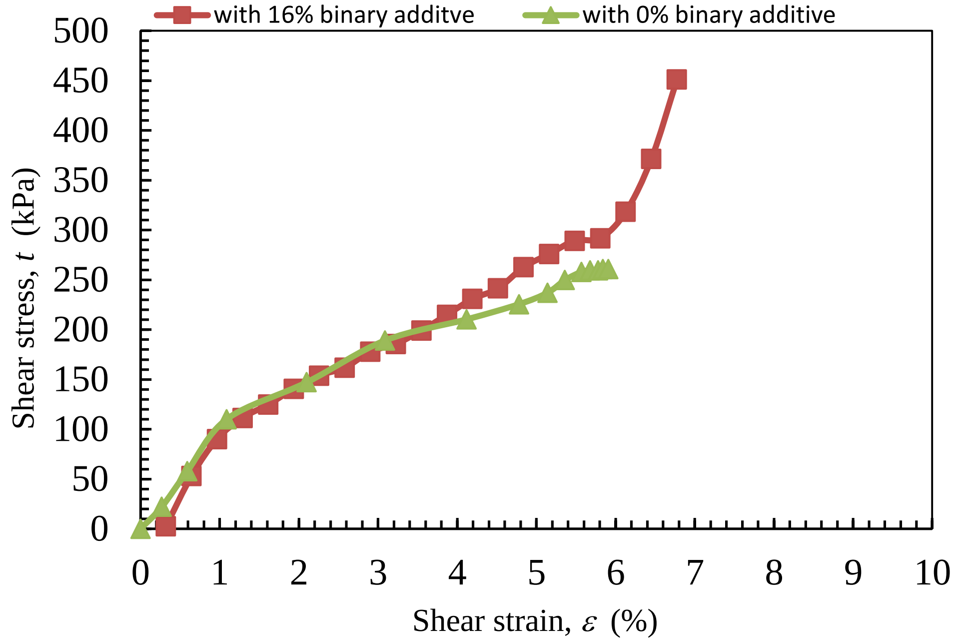

Figure 4-8 Direct Shear Test Result of Specimen with 0% and 16% BA (28 Days Curing)

Table 4-2 Cohesion and Angle of Internal Friction of Specimen with 0% and 16% BA after 28 Days of Curing

	Specimen with 0% Binary Additive	**Specimen with 16% Binary Additive**
Cohesion (kPa)	198	140
Angle of internal friction (Degree)	61	56.4

4.3. Wetting / Drying Cycle

After 5th cycle, specimens were completely destroyed no further cycles was able to apply so results are taken and graphical representations generated are given below. Below figure 4-9 shows the Hz deformation of flexural specimen with 0% BA in which positive (+) approach is for swelling during wetting cycle and negative (-) approach is for shrinkage during drying cycle. Here max swelling is 12.6% and maximum shrinkage is -20.5 %. Above chart also shows the Hz deformation of flexural specimen with 16% BA in which positive (+) approach is for swelling during wetting cycle and negative (-) approach is for shrinkage during drying cycle. Here max swelling is 8.7% and maximum shrinkage is -12.6 %.

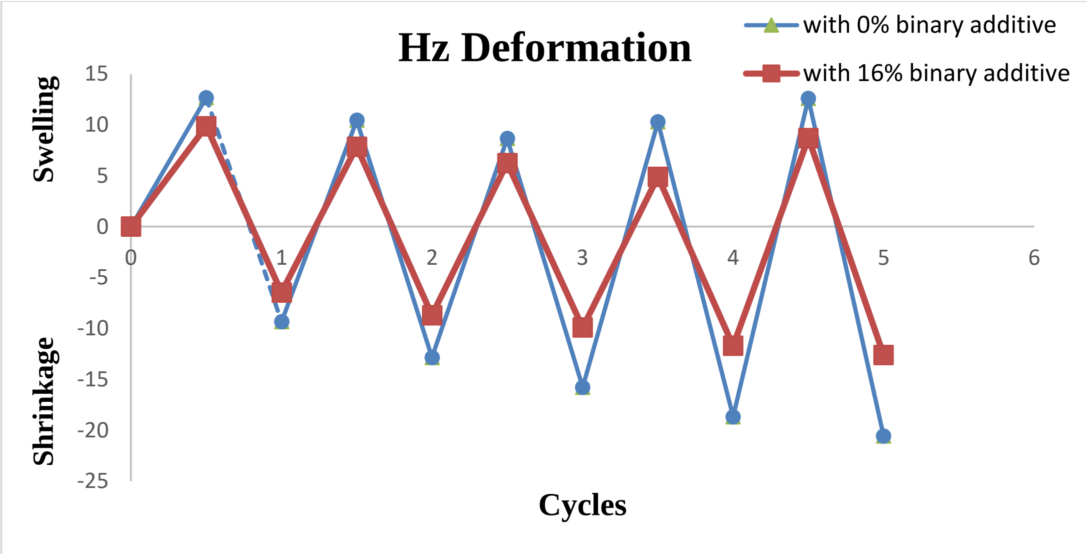

Figure 4-9 Horizontal Deformation of Flexural Sample with 0% and 16% BA during Wetting \ Drying Cycles

Now Hz deformation results of unconfined specimens with 0 and 16% BA are given below.

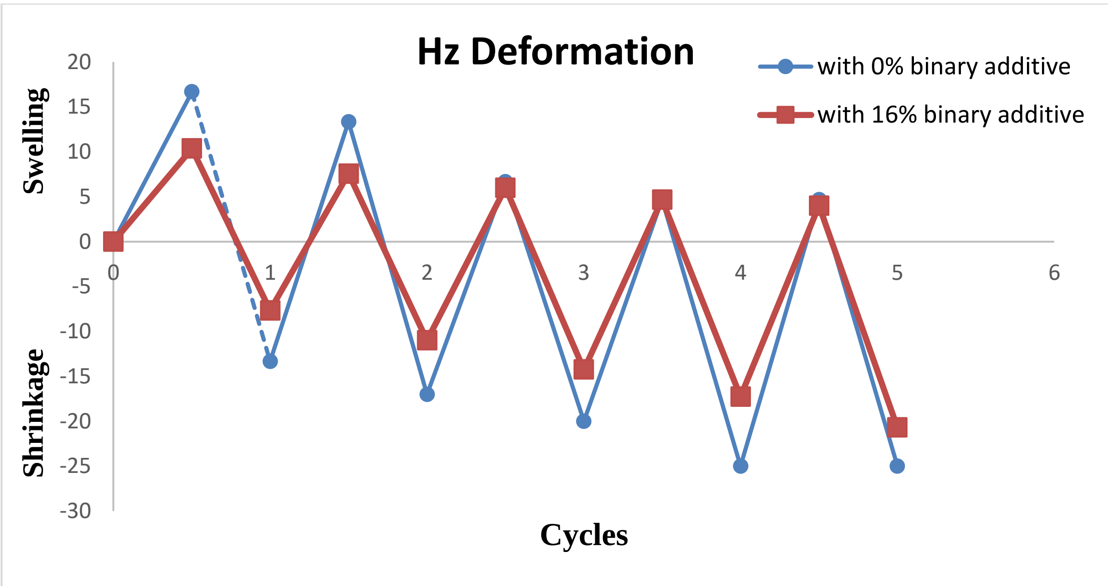

Figure 4-10 Horizontal Deformation of Unconfined Sample with 0% and 16% BA during Wetting \ Drying Cycles

Above figure 4-10 shows the Hz deformation of unconfined specimen with 0% BA in which positive (+) approach is for swelling during wetting cycle and negative (-) approach is for shrinkage during drying cycle. Here max swelling is 16.5% and maximum shrinkage is -25%. Above chart also shows the Hz deformation of unconfined specimen with 16% BA in which positive (+) approach is for swelling during wetting cycle and negative (-) approach is for shrinkage during drying cycle. Here max swelling is 10% and maximum shrinkage is -20.6%.

4.4. Swell Potential and Swell Pressure Test (Swell Test)

In this section results of swell test performed on re-molded soil with 0% and 16% BA were discussed. Specimens were observed for 05 days using floating ring in consolidometer saturated condition.

4.4.1. Swelling Potential Result of Specimen with (0, 16) % Binary Additive

Swelling potential is the amount of swelling observed within test period in saturated condition. Following chart (Figure 4-11) shows the swelling trend of soil specimen with 0% and 16% BA. In below chart, maximum of 18.12% of swelling (after 05 days) is observed when no BA is added in highly plastic soil and maximum of 7.37 % of swelling (after 05 days) is observed when 16% BA is added in highly plastic soil. So 10.75% of swelling is reduced due to the addition of 16% BA.

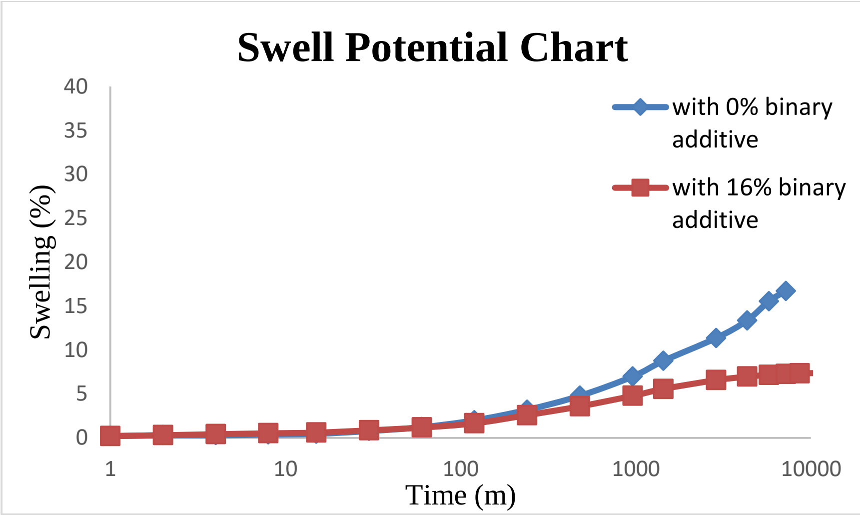

Figure 4-11 Swelling Potential of Specimen with (0, 16) % Binary Additive

4.4.2. Swelling Potential Result of Specimen with (0, 16) % Binary Additive

Swelling pressure is the amount of pressure exerted by the specimen within test period in saturated condition. Following chart (Figure 4-12) shows the pressure trend of soil specimen with 0% and 16% BA. Below figure shows, maximum of 110 kPa pressure is induced by the specimen (after 05 days) when no BA is added in highly plastic soil and maximum of 43 kPa pressure is induced (after 05 days) by the specimen when 16% BA is added in highly expansive soil. So 67 kPa of swelling pressure is reduced due to the addition of 16% BA.

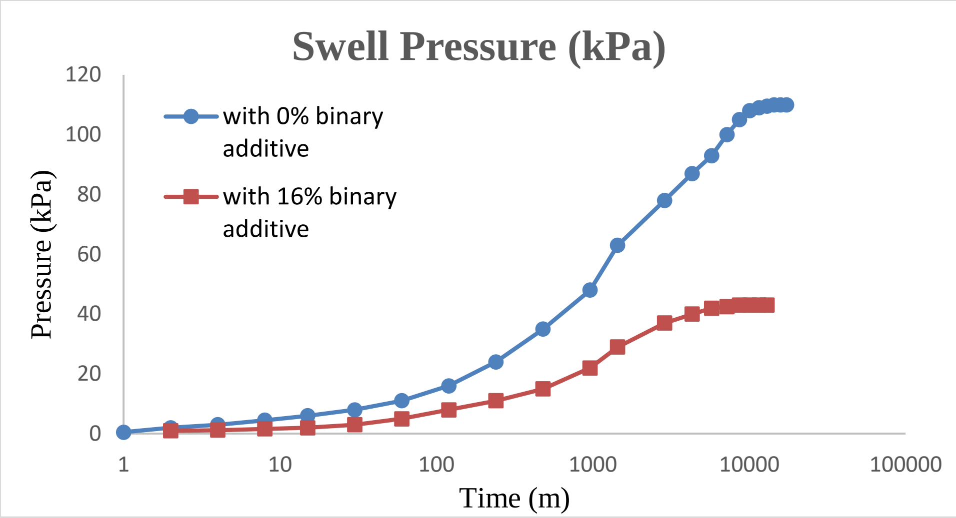

Figure 4-12 Swelling Pressure of Specimen with (0, 16)% Binary Additive

5. CONCLUSION

Following conclusions are drawn based on all above experimental investigations and results

1. The incorporation of 16% BA lowered the Atterberg's Limits (PL from 80% to 39.5% and LL from 90% to 79%) of highly plastic soil which indicates improvement in its engineering properties.
2. In case of 16% BA maximum dry density of 15.95 $KN/m3$ is achieved at lower compaction effort.
3. Without BA the swell index of highly plastic soil is 143% on the other hand with the addition of 16% BA swell index is reduced to 91%.
4. After 0, 3, 7 and 28 days of curing a considerable increase in shear strength of specimens with 16% BA.
5. After five W/D cycle specimen without BA has shown 12.6% swelling and 20.5% shrinkage while specimen with 16% BA had 9.8% of swelling and 12.6% shrinkage.
6. Specimen without BA has swell potential of 18.12% while specimen with 16% BA has swell potential of 7.37%. So 10.75% of swelling is reduced due to the addition of 16% BA.
7. Specimen without BA has induced swell pressure of 110 kPa while specimen with 16% BA induced swell pressure of 43 kPa. So 67 kPa of swelling pressure is reduced due to the addition of 16% BA.

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