The Effect of Varying Heat Temperatures on the Grading and the Swelling Behaviour of Expansive Clay Soils

Charles K. Kabubo[1], Zachary A. Gariy[2], Sixtus K. Mwea[3], Sylvester O. Abuodha[4]

[1]Senior Lecturer, Sustainable Materials Research and Technology Centre, Jomo Kenyatta Univ. of Agriculture and Technology (JKUAT), P.O Box 62000-00200 Nairobi, Kenya

[2]Associate Professor, Department of Civil, Construction and Environmental Engineering (JKUAT)

[3]Professor, Department of Civil and Construction Engineering, University of Nairobi, P.O Box 30197 Nairobi, Kenya

[4]Senior Lecturer, Department of Civil and Construction Engineering, University of Nairobi, P.O Box 30197 Nairobi, Kenya

Abstract: ***Because of their alternate swelling and shrinkage behaviour during wet and dry seasons respectively, expansive soils are highly problematic, causing damage to structures and pavements built on them. There are many methods of improving such soils. This paper examines the effect of thermal stabilization on the grading and swelling characteristics of naturally occurring black cotton spoil, an expansive clay. The thermal treatment involved heating the soil in temperatures ranging from 200 to 1200^{0}C for 2 hours in a closed kiln, and then carrying out tests on the neat and heated samples so as to obtain an optimal temperature. The tests included particle size distribution, atterberg limits and free swell. The soil grading altered progressively from one consisting of clay and silt to one consisting of sand and gravel. The swelling characteristics increased as the clay was heated up-to a temperature of 250^{0}C then decreased steadily, diminishing at 700^{0}C. Thus the optimal temperature was obtained as 700^{0}C.This shows that heating expansive clays to 700^{0}Ccan reduce their swelling characteristics, and make them suitable for use as fill materials.***

Keywords: Expansive clay, heated soil, grading, liquid limit, plastic limit, plasticity index, free swell

1. Introduction

According to Nelson and Miller (1992) cited in Wubshet and Tadesse (2014), expansive soils are known to exhibit dual characteristics of excessive swelling and shrinkage under different moisture conditions which cause deformations significantly greater than elastic deformations. These deformations cannot be predicted by classical elastic and plate theory. The movements caused are usually of an uneven pattern and of such magnitude as to cause extensive damage to the structures and pavements resting on them. According to Sridharan and Prakash (2000) expansive clays are characterized by the presence of a large proportion of highly active clay minerals of the montmorillonite group which are responsible for the pronounced volume change capability of the soils. Chen (1975) cited in Sridharan and Prakash (2000) states that the presence of expansive soils greatly affects the construction activities in many parts of South-Western United States, South America, Canada, Africa, Australia, Europe, India, China and Middle East. Black cotton soil which is an expansive clay occupies about 3% of the world area, about 340 million hectares (Bhavsar and Patel, 2014). The estimated annual cost of the damage of the facilities built on expansive clays in the United States exceeds 9 billion dollars (Sun et al., 2014). Thus obtaining a good method of improving these soils will reduce the negative effects on construction and optimize on cost of construction.

Various methods that have been employed to improve expansive clays include mechanical compaction, surcharge loading, pre-wetting, lime stabilization, cement stabilization, fly ash stabilization, chemical stabilization, and organic compounds treatment according to Nelson and Miller (1992) cited in Sun et al. (2014). Other methods include cut to spoil followed by importing good material for replacement (Obuzor et al., 2011; Mohktari and Deghani, 2012 as cited in Bhavsar and Patel, 2014) and thermal treatment (O'Flaherty, 1974; Bell, 1993; Terzaghi et al., 1996; Wang et al., 2003; Kabubo et al., 2017).

According to O'Flaherty (1974) heating causes a decrease in plasticity index and a reduction in attraction of water. Bell (1993) states that when the soil is heated to temperature of 400^{0}C to 600^{0}C, some irreversible changes occur which make the soil non-plastic and non-expansive and that thermal treatment reduces the swelling capacity and compressibility of clay soils. Wang et al. (2008) found that the swelling potentials of two expansive ***commercial clays***, Edgar Plastic Kaolin (EPK) and Western Bentonite (WB) diminish when heated to about 400^{0}C for EPK and 600^{0}C for WB, and concluded that heating can effectively stop expansive soils from swelling. Kabubo et al. (2017) heated expansive clay to a single temperature of 600^{0}C and established that the clay fraction diminished, and that the plasticity and the swelling characteristics improved.

This research looked into the effect on the grading and swelling characteristics on thermal treatment of the expansive clays for temperatures ranging from 200^{0}C to 1200^{0}C for 2 hours, so as to establish the temperature at which these properties become optimal for a naturally occurring expansive clay. The 2 hour period was adopted from Kabubo et al. (2017) who found it to be an effective period for thermal treatment.

2. Materials and Methods

2.1 Materials

The expansive soil used in this research was black cotton soil obtained from Mwihoko area of Kiambu County, about 25 km North East of Nairobi City in Kenya. The material used was located 500 mm below ground to avoid vegetable matter. This is the same material which was used by Kabubo et al. (2017).The tests conducted on both the air dried neat and heated samples were particle size distribution, liquid limit, plastic limit, linear shrinkage and free swell. Four tests were conducted for each test and the mean value taken. Thermal treatment at 200 - 1200^{0}C was done for 2 hours in a Esklo s.r.o closed electric kiln with a power rating of 108 kVA, to eliminate emission of greenhouse gases (figure 1). The tests were done after the soil had cooled to room temperature, thus allowing for any reversal of properties.

Figure 1: Esklo s.r.o heating kiln (Sample in position)

2.2 Test Methods

2.2.1 Determination of Particle Size Distribution (Grading).

The tests that were done were wet sieving on a nest of British Standard (BS) sieves in descending order on oven dried material passing a 20 mm sieve, and sedimentation by hydrometer analysis on soil finer than 63 microns sieve to the clay size, in accordance with BS 1377 Part 2:1990. A plot on a semi-logarithmic chart was then done of the percentage of material passing each sieve against the sieve sizes to give the grading. The coefficient of uniformity, Cu and coefficient of curvature Cc were determined as follows:

$$C_u = \frac{D60}{D10} \qquad (1)$$

$$C_c = \frac{D30^2}{D10XD60} \qquad (2)$$

Where

D_{10} is the particle diameter at which 10% by mass is finer
D_{30} is the particle diameter at which 30% by mass is finer
D_{60} is the particle diameter at which 60% by mass is finer

2.2.2 Determination of Atterberg Limits/Soil Plasticity

These tests were carried out in accordance with BS1377 Part 2:1990 and included the liquid limit, the plastic limit, the plasticity index and the linear shrinkage.

Liquid Limit (LL)

This test which was done to determine the water content at which the soil changes from liquid state to the plastic state was carried out using the cone penetrometer method which is easier to carry out and gives more reproducible results than the earlier Casagrande test, according to BS 1377 Part 2:1990. It was determined as the moisture content at cone penetration of 20 mm from a plot of the moisture content against the cone penetration on a sample passing the 425 micron BS test sieve,

Plastic Limit (PL)

The test was carried out using the paste from the liquid limit test, to establish the lowest moisture content at which the soil is plastic. A sample was dried partially on a glass rolling plate and rolled between the plate and the finger. The plastic limit was determined as the moisture content at which a 3 mm diameter thread sheared longitudinally and transversely.

Plasticity Index (PI)

This was determined as the difference between the liquid limit and the plastic limit (PI = LL-PL).

Linear Shrinkage (LS)

It was done on a sample passing a 425 micron sieve, having the moisture content of the liquid limit. The sample was placed in a brass linear shrinkage mould, air dried for 2 days until the soil shrunk from the walls of the mould, after which it was oven dried at 105^{0}C. The length of the oven-dry specimen (Ld) was measured and the linear shrinkage calculated as a percentage of the original length of the specimen (Lo) as follows:

$$\% \text{ of linear shrinkage} = (1-\frac{Ld}{Lo}) \text{ x100.} \qquad (3)$$

2.2.3 Free Swell

The test was carried out on an air dried material passing the 425 micron sieve in accordance with the method proposed by Holtz and Gibbs (1956). The material was slowly poured into a graduated cylinder, after which the cylinder was filled with distilled water. The soil was allowed to settle and the initial volume of the soil taken as Li. The mixture was allowed to stand for 24 hours, and the swelled volume taken as Lf. The free swell was computed as percentage as follows:

$$\text{Free swell} = \frac{Lf-Li}{Li} \text{x 100.} \qquad (4)$$

Figure 2: Free Swell Test

3. Results and Discussion

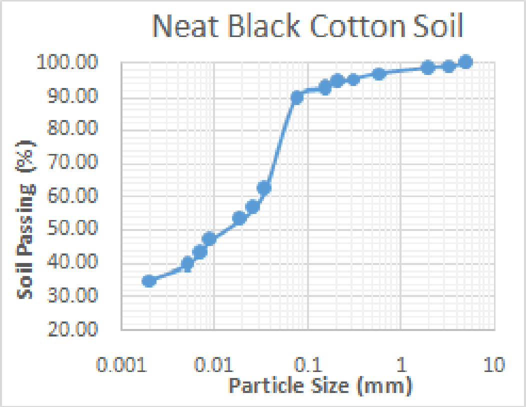
3.1 Grading

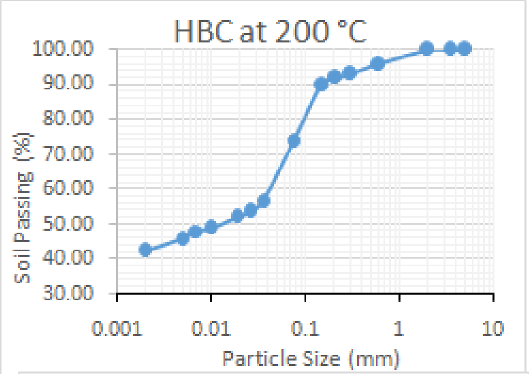
The grading results are shown in figure 3(a-j) for the neat black cotton soil (NBC), and heated black cotton soil (HBC) at temperatures of 200 to 1200°C. The classification is in terms of size, with clay being less than 0.002 mm, silt between 0.002 and 0.06, sand 0.06 and 2 mm, while gravel is material between 2 mm and 60 mm. The NBC has 35% clay, 48% silt, 15% sand and 2% gravel, and is thus classified as a sandy clayey silt. The HBC at 200°C has 42% clay, 26% silt, and 32% sand, and is thus classified as a silty sandy clay. The HBC at 300°C has 48% clay, 22% silt, and 30% sand and is thus classified as a silty sandy clay. The HBC at 400°C has 21% clay, 37% silt, 41% sand and 1% gravel, and is thus classified as a clayey silty sand. The HBC at 500°C has 4% clay, 21% silt and 75% sand, and is thus classified as a silty sand. The HBC at 600°C has 19% silt, 44% sand and 37% gravel, and is thus classified as a silty gravelly sand. The HBC at 700°C has 35% silt, 24% sand and 41% gravel, and is thus classified as a sandy silty gravel. The HBC at 800°C has 27% silt, 37% sand and 36% gravel, and is thus classified as a silty gravelly sand. The HBC at 1000°C has 2% silt, 44% sand and 54% gravel, and is thus classified as a sandy gravel. The HBC at 1200°C has 17% silt, 50% sand and 33% gravel, and is thus classified as a silty gravelly sand.

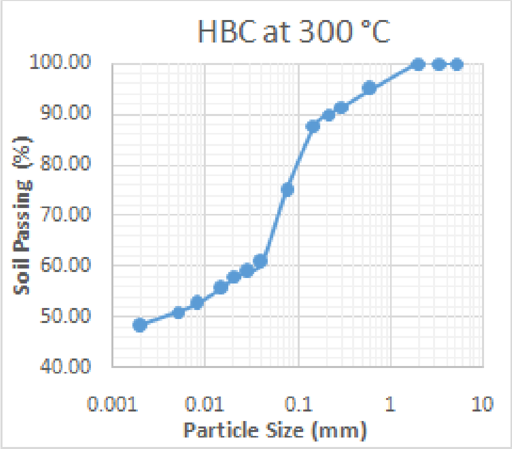
As heating progressed, the clay fraction increased from 35% for the NBC reaching a maximum of 41% for HBC at 300°C, then decreased eventually diminishing at a temperature of 600°C. Further, the silt fraction decreased erratically from 48% for the NBC, becoming negligible at 1000°C. The sand fraction increased erratically from 15% for the NBC to 50% for the HBC at 1200°C. The gravel fraction appeared suddenly at 37% for HBC at 600°C reaching a maximum of 54% HBC at 1000°C becoming 33% for HBC at 1200°C. Heating gradually reduced the cohesiveness of the soil, and at 800 °C, the soil could not bond.

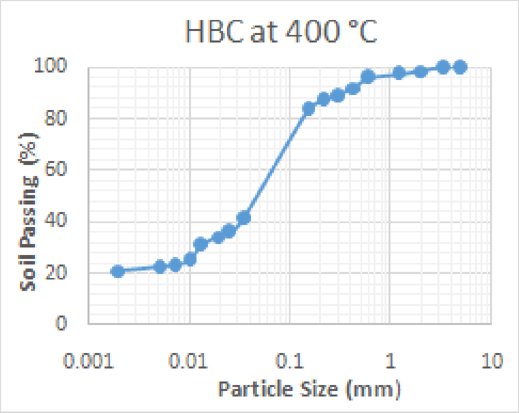
The grading types of the various temperatures ranges are shown on Table 1. According to O'Flaherty (2002), a soil is either well graded, uniform graded or poorly graded. It is well graded if the coefficient of uniformity C_u is 5 or more and the coefficient of curvature C_c is between 1 and 3. Such a soil compacts to a uniform mass. A uniform soil has a small range of sizes with a steep gradation curve, and it is poorly graded if C_u is equal to or less than 2. A soil which may have a near-horizontal "hump" or "step" in its gradation is poorly graded indicating that it is missing some intermediate sizes; it is called gap graded. The NBC, the HBC at 200, 300, 400, 700, 800 and 1200°C do not have a D_{10} value; C_u and C_c cannot be defined and are therefore gap graded. The NBC and HBC at 200, 300 and 400°C also have particles with sizes at the lower and upper sizes, while the HBC at 800°C has a horizontal step in the lower sizes, further reinforcing their being gap graded. The HBC at 500 and 600°C have C_u greater than or equal to 5 and C_c between and 3, are therefore well graded with a smooth and upward curve. The HBC at 1000°C has C_u value of 4.3, which fairly close to 5 and a $C_{c\ of}$ 1.1 making it fairly well graded which is also apparent from the grading curve.

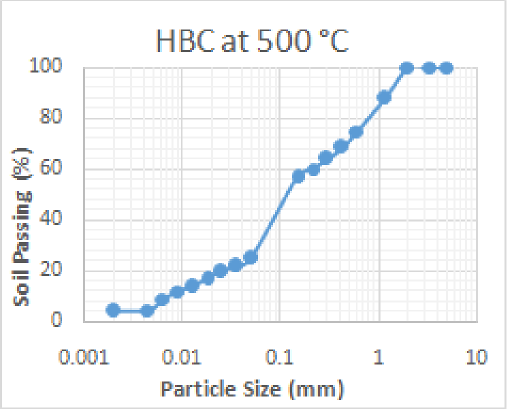
Wang et al. (2008) cited in Kabubo et al. (2017) attributed the loss of cohesiveness to destruction of the crystal structure of expansive soils from de-hydroxylation as the temperature increased beyond 400^0C leading to formation of larger particles possibly due to electro-chemical bonds resulting in formation of larger particles. According Bell (1993) also cited in Kabubo et al. (2017) soils at temperatures of between 200 and 400^0C begin to aggregate into granules and when the soil is heated to temperatures of 400^0C to 600^0C, some irreversible changes occur which make the soil non-plastic and non-expansive. Wang et al. (2008) cited in Kabubo et al. (2017) also suggested that the formation of the larger particles is irreversible. Thus the black cotton under study experienced de-hydroxylation followed by an irreversible formation of larger particles/granules. According to Bell (1993) cited in Kabubo et al. (2017) fusion and verification set in at temperatures of 600^0C and 700^0C explaining the sudden appearance of the gravel fraction for HBC at 600^0C. With further increase in temperature, there was some fusion and verification, and a brick like material was obtained, which was similarly reported by Bell (1993) cited in Kabubo et al. (2017).

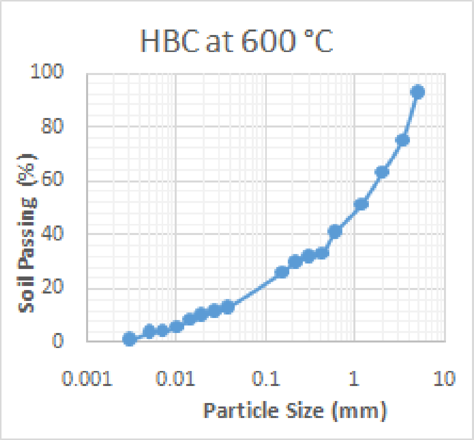


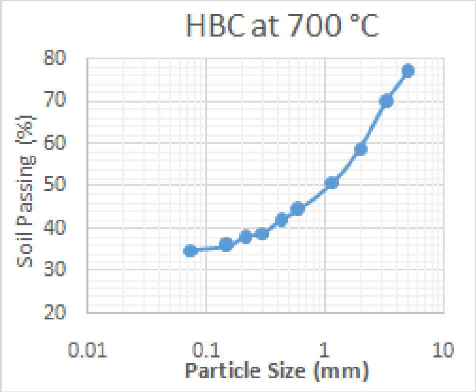

HBC at 200 °C
Soil Passing (%)
100.00
90.00
80.00
70.00
60.00
50.00
40.00
30.00
0.001
0.01
0.1
1
10
Particle Size (mm)

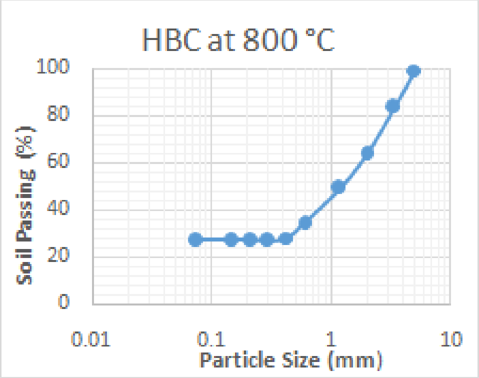

HBC at 300 °C
Soil Passing (%)
100.00
90.00
80.00
70.00
60.00
50.00
40.00
0.001
0.01
0.1
1
10
Particle Size (mm)

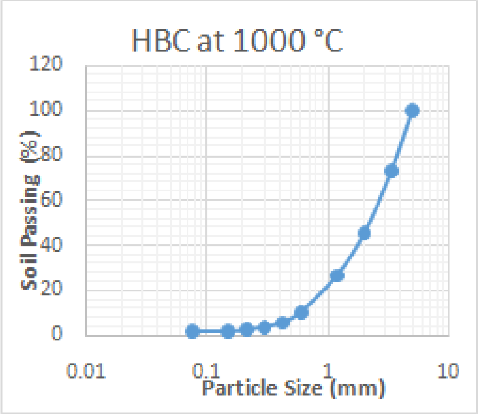

HBC at 400 °C
Soil Passing (%)
100
80
60
40
20
0
0.001
0.01
0.1
1
10
Particle Size (mm)


HBC at 500 °C
Soil Passing (%)
100
80
60
40
20
0
0.001
0.01
0.1
1
10
Particle Size (mm)


HBC at 600 °C
Soil Passing (%)
100
80
60
40
20
0
0.001
0.01
0.1
1
10
Particle Size (mm)


HBC at 700 °C
Soil Passing (%)
80
70
60
50
40
30
20
0.01
0.1
1
10
Particle Size (mm)


HBC at 800 °C
Soil Passing (%)
100
80
60
40
20
0
0.01
0.1
1
10
Particle Size (mm)


HBC at 1000 °C
Soil Passing (%)
120
100
80
60
40
20
0
0.01
0.1
1
10
Particle Size (mm)

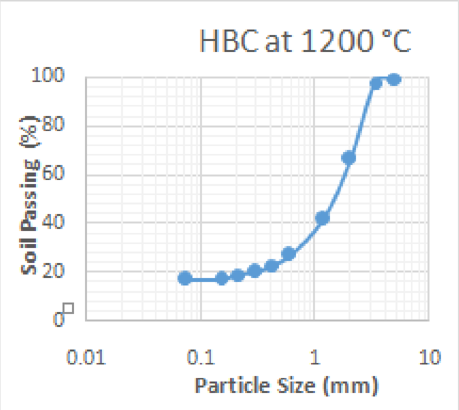


Figure 3(a-j): Grading

Table 1: Variation of Coefficients of Uniformity (Cu) and of Curvature (Cc) with Temperature

Reference	D10	D30	D60	C_u	C_c	Grading Type
NEAT	-	-	0.034	-	-	gap graded
HBC 200	-	-	0.042	-	-	gap graded
HBC 300	-	-	0.04	-	-	gap graded
HBC 400	-	0.013	0.065	-	-	gap graded
HBC 500	0.0064	0.06	0.212	33	2.7	well graded
HBC 600	0.019	0.212	1.8	95	1.3	well graded
HBC 700	-	-	2	-	-	gap graded
HBC 800	-	0.43	1.8	-	-	gap graded
HBC 1000	0.6	1.3	2.6	4.3	1.1	well graded
HBC 1200	-	0.65	1.8	-	-	gap graded

3.2 Atterberg Limits and Free Swell

According to Bhavsar and Patel (2014), for neat black cotton soils in India, liquid limit (LL) values range from 50 to 100%, plastic index (PI) from 20 to 65%, and shrinkage limit from 9 to 14%. The black cotton under study had LL value of 65%, PI of 35%, LS of 15%, fitting fairly well in the range stated by Bhavsar and Patel (2014). As shown in figure 4, as the heating progressed the LL, PI, LS and FS increased to a maximum value at a temperature of about 250^{0}C after which they all decreased to a minimum at 700^{0}C with the LL and FS falling very sharply. The PL decreased erratically; the LL, PL, PI and LS become zero at 700^{0}C. As the temperature was increased, the crystalline structure of the clay mineral underwent irreversible changes. The soil experienced reduction in swelling characteristics as the heating progressed beyond 250^{0}C; significant reductions in the LL (15%) and FS (20%) occurred at 400^{0}C; at 600^{0}C, the LL, the PL, the PI, the LS and FS fell very significantly by 32%, 31%, 33%, 33% and 37% respectively; the LL, PL, PI and LS diminished at700^{0}C, while FS reduced by 88% to 9%, as shown in figure 4.

According to Bell (1993), the fine particles begin to aggregate into granules as heating is increased and at temperatures of 400^{0}C to 600^{0}C, some irreversible changes occur which make the soil non-plastic and non-expansive. The reduction in plasticity occurred due destruction of the crystalline structure of the expansive soil from de-hydroxylation as the temperature increased beyond 400^{0}C, resulting in formation of granules possibly due to electro-chemical bonds (Wang et al. (2008) cited in Kabubo et al.,2017). Beyond 600^{0}C there was some fusion and verification, and a brick like material was obtained, which was similarly reported by Bell (1993) cited in Kabubo et al. (2017), explaining the diminishing of swelling characteristics.

Holtz and Gibbs (1956) cited in Murthy (2007) suggested that soils having free swell values of 100 can cause considerable damage to lightly loaded structures and that soils having a free swell below 50 seldom exhibit appreciable volume change even under light loadings. Based on swell potential, the United States Army Engineers Waterways Experimental Station cited in Bell (2007) classifies soils based on the PI, the LL and initial suction; soils with PI less than 50 and LL less than 25 are classified as having a potential swell of 0.5%, and a low classification of swell potential. Ministry of Transport and Communication (1987) limits the plasticity index to a maximum of 50. The soil under study had PI less than 50, while the free swell dropped to below 50 for soil heated at 500 ^{0}C and over. However, the LL only dropped to below 25 at 700 ^{0}C.Thus at 700 ^{0}C, the soil has a swell potential of 0.5%, is classified as having a low swell potential, and will hardly experience a volume change even under light loads. Based on swelling characteristics, expansive soils heated to 700 ^{0}C are therefore suitable for use as fill material.

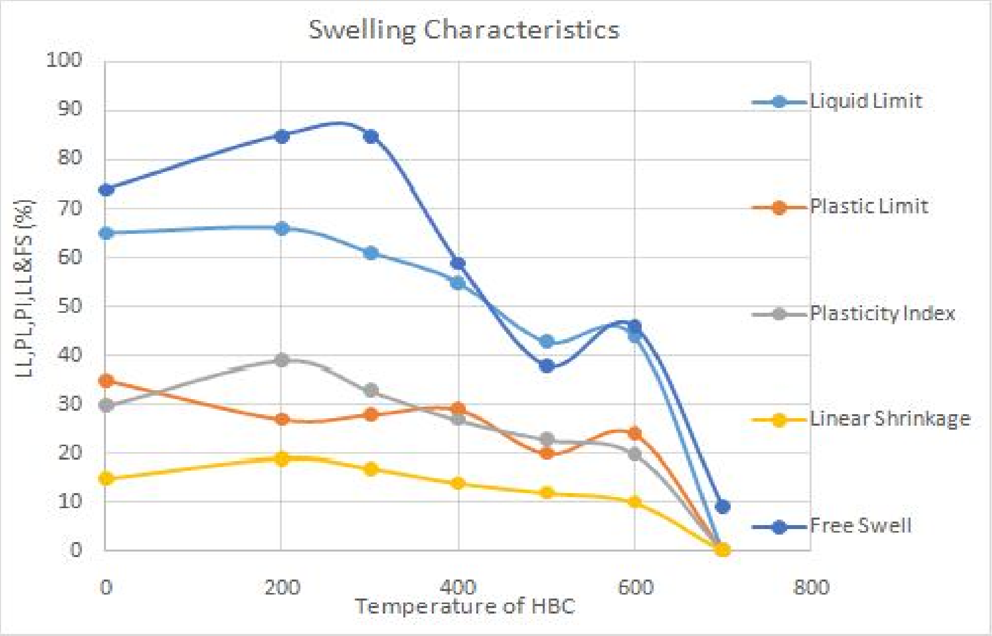


Figure 4: Variation of Liquid Limit, Plastic Limit, Plasticity Index, Linear Shrinkage and Free Swell with Temperature of Black Cotton Soil

4. Conclusion

In the temperature range 200 to 1200^{0}C the black cotton soil under study experienced irreversible chemical changes which resulted in the following:

- The particle size distribution altered. As heating progressed, the clay fraction increased up to 300°C, then decreased eventually diminishing at a temperature of 600°C. Further, the silt fraction decreased erratically, becoming negligible at 1000°C. The sand fraction increased erratically, from 15% for the NBC, to 50% for the HBC at 1200°C. At 600°C, the gravel fraction suddenly appeared. Heating gradually reduced the cohesiveness of the soil, and at 800 °C, the soil could not bond.
- Heating greatly reduced the plasticity index, the linear shrinkage and the free swell. Above 700 ^{0}C, the soil became non plastic, with the swelling behavior diminishing.
- Thus, the optimum temperature at which the expansive clay should be heated to become non - plastic and for swelling behavior to diminish is 700^{0}C.

5. Recommendations

There is need to develop a cost effective in-situ thermal-stabilization method. However, the fact the soil became fully non plastic at 700 ^{0}C and that the swell behavior diminished gives the soil a great potential for use as a fill material.

References

[1] Bhavsar, S.N. and Patel, A. J. (2014). *Analysis of Swelling and Shrinkage Properties of Expansive Soil using Brick Dust as a Stabilizer*. International Journal of Emerging Technology and Advanced Engineering, Vol. 4, pp 303-308.
[2] Bell, F.G. (1993). *Engineering Treatment of Soils*. pp 240-267. E&FN Spon, London.
[3] Bell, F.G. (2007). *Engineering Geology*. 2nd ed. p 528. Elsevier ltd. Oxford, UK
[4] BSI. 1990. BS1377. Methods of testing for soils for civil engineering purposes. British Standards Institution (BSI), London, UK.
[5] Holtz, W. G. and Gibbs, H. J. (1956). *Transactions of ASCE*, 121, pp. 666-669.
[6] Kabubo C.K., Gariy, Z.A., Mwea, S.K., Abuodha, S.O. (2017).*The Effect of Heat on the Properties of Expansive Clay Soil*. International Journal of Science and Research (IJSR). Vol. 6(1), DOI: 10.21275/28121602, pp 464-469.
[7] Obuzor, G.N., Kinuthia, J.M. and Robinson, R.B. (2011). Utilization of *lime-activated-GCBS to reduce the deleterious effect of flooding on road structural materials: A laboratory Simulation*. Engineering Geology, Vol.122, pp.334-338.
[8] Obuzor, G.N., Kinuthia, J.M. and Robinson, R.B. (2012). *Soil Stabilization with lime-activated-GCBS – A mitigation to flooding effects on road structural layers/embankments constructed on flood plains*. Engineering Geology, Vol.151, pp.112-119.
[9] O'Flaherty, C.A. (1974). *Highways 2nd Edition. Vol. 2 Highway Engineering*, pp. 221 – 267. Edward Arnold.
[10] O'Flaherty, C.A. (2002). *Highways 4th Edition*. pp.95 – 101. Butterworth-Heineman
[11] Murthy, V.N.S. (2007). *Text Book of Soil Mechanics and Foundation Engineering*. pp. 567. CBS Publishers and Distributers Pvt Ltd, New Delhi
[12] Sridharan, A. and Prakash, K. (2000). *Classification Procedures for Expansive Soils*. Proceedings of the Institute of Civil Engineers. Geotech. Engng, **143**, pp. 235-240

[13] Sun, Y., Petry, T., Ge, L., and Zhao, H. (2014). *Effects of Chemical Stabilizers on an Expansive Clay. KSCE Journal of Civil Engineering 18(4),*DOI: 10.1007/s12205-013-1013-5.

[14] Terzaghi, K., Peck, R. B., and Mesri, G. (1996). *Soil Mechanics in Engineering Practice*. 3rd Edition. John Wiley & Sons Inc., pp. 3 -17 and 312-313.

[15] Wang, M.C., Jao, O. and Ghazal, M.S. (2008). *Heating Effect of Swelling Behavior of Expansive Soils.* Geomechanics and Geoengineering, pp. 121-127.

[16] Wubshet, M. and Tadesse, S. (2014). *Stabilization of Expansive Soil Using Bagasse Ash and Lime*. Journal of EAA, Vol. 32, pp. 21-25.