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A GEOTECHNICAL CHARACTERISTICS OF SWELL-SHRINK SOILS IN KIBAHA, TANZANIA

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ABSTRACT

The properties of Swell-shrink of the soils in Kibaha were studied. Geotechnical and mineralogical tests were carried out on disturbed and undisturbed samples recovered from trial pits at different locations. Using empirical relationships, the swelling potential of the soil was established by relating the soil plasticity limits and clay contents obtained. The average values for the plastic, liquid and linear shrinkage limits for soils were 22.2%, 60.7% and 14.5% respectively. The plasticity Index (PI) which is the difference between the liquid limit and the plastic limit ranges from 27% to 47.4% with an average of 38.5%. The natural water content was found to be very small ranging, from 7% to 11% with an average of 9.6% which is smaller than the corresponding shrinkage limit.

Furthermore, the samples were tested for percentage of volume change in free swell tests and swelling pressure in one-dimensional swell tests. The free swell and upward pressure were in excess of 100% to 150%, and 50 kPa respectively. In addition, the coefficient of linear extensibility ranged from 0.09 to 0.14 signifying high to very high swell-shrink potential.

Finally, the main clay mineral present in the sample was determined by running the x-ray diffraction (XRD) test. The x-ray diffraction scan indicated the presence of high proportion of smectite as the main clay minerals in the soils.

KEYWORDS: Swell-shrink soils, soil index properties, potential swell, montmorillonite (smectite)

INTRODUCTION

Swell-shrink soils are defined as those which exhibit large volume changes when wetted and large shrinkage when dried. Apart from the moisture change, the swell-shrink behaviour is controlled by the amount of clay minerals and soil structure. Investigations to determine the physical properties of soils which undergo swelling and shrinkage call for the geological study and the geotechnical and mineralogical tests of the soils from the area.

Over the past decade, many structures have been constructed in the Kibaha with little knowledge of the presence and effect of swell-shrink soils in the area. As a result many structures especially light weight structures are showing signs of damage in the form of cracks. This study brings to light the fact that the soils in Kibaha exhibit swell-shrink properties and so call for special attention during civil construction works in the area.

This paper seeks to identify the characteristics of swell-shrink soils in Kibaha area. Geotechnical analysis was carried out by running the sieve analysis test, Atterberg limits (liquid limit, plastic limit, and shrinkage limit) tests and swelling tests. The soil consistency limits were correlated with clay contents to assess the degree of expansiveness of the soils. Furthermore, the free swell was conducted followed by the coefficient of linear extensibility (COLE). The above tests were supplemented with x-ray analysis to determine the predominant clay minerals that tend to influence shrinkage and swelling properties of the soils in the area. Finally, swell-shrink mitigation measures were recommended on the basis of accepted construction experiences on expansive soils.

Location and Climate of the area

The study area is positioned at an altitude 107m above sea level and lies approximately between latitudes of 06°35'S and 06°55'S and longitudes of 38°45'E and 39°15'E. This area has a climate which is a clear contrast between humid subtropical region and semiarid region. It has two distinct seasons which are distinguished by duration of rainfall rather than by changes in temperature. The period of long rains is from March to May and that of short rains from October to December. The two seasons are separated by dry seasons; the hottest months falling between January and March and the coldest with minimum average of 21 °C coming between June and August. In the hot months, air and soil temperatures are high exceeding 30°C. These conditions lead to decrease of soil moisture that leads directly to a desiccation of the soil surface.

Geology

The soils in this region comprise deposition of alternating sedimentary sequences consisting of mudstones, sandstones, limestones, shales, sand, gravel and vertebrate fossils (Mpanda, 1997). The sediments are considered as the first phase of the sedimentary cycle dating from late Mesozoic and early Cenozoic periods. The soil that is exposed at the surface consists chiefly of sandy clay. The sediments were cemented together resulting into soils of high clay content. They are normally consolidated, highly plastic and exhibits high swell potential when wet.

Geotechnical Investigation

Geotechnical site investigation consisted of profile description and the collection of both disturbed and undisturbed samples. Four open trial pits, each measuring 2m long and 1m wide and 3m deep were manually dug at representative locations. Samples were collected at approximately 50 cm vertical intervals down to the bottom of ach pit. The collected samples were carefully sealed in plastic sachets in the field, labeled according to the grid co-ordinates and subsequently transported to the laboratory for analysis. The soil profiles cross-sections show that the ground is dominated by clayey soils with alternations of sandy-silt layers.

Laboratory Tests to Characterise the Soils

Common index property tests to characterize the expansiveness of soils were carried out on 13 undisturbed samples collected from the trial pits. These included determination of clay contents, dry unit weight, water contents and Atteberg limits.

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Particle Size Analyses and Unit Weight

The samples are predominantly course textured at all depths with predominance of sands. The grain size distribution indicate high proportion of sand followed by moderate amount of fines and small amount of granular deposits (Figure 1).

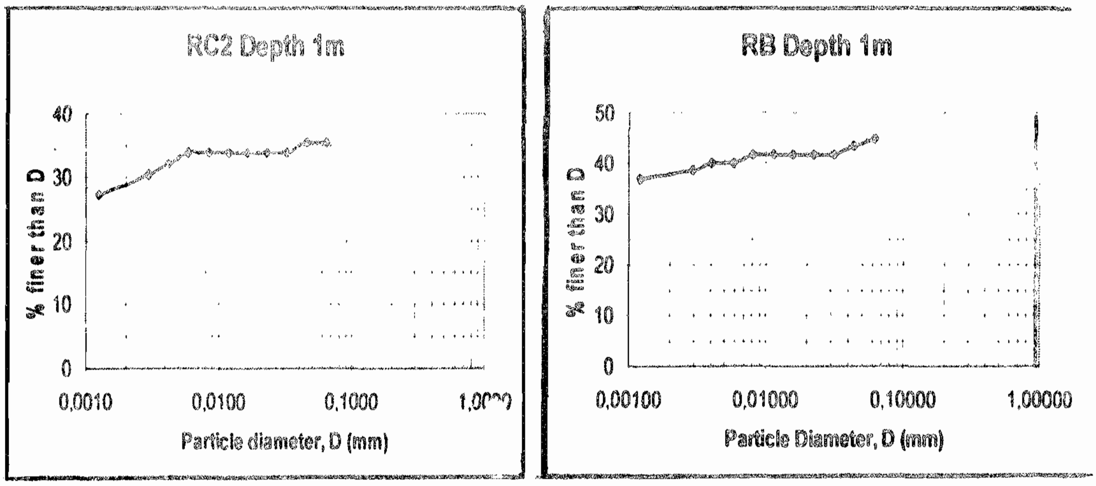


Figure 1: Base two logarithmic particle size distribution curves for 2 samples at 1 m depth

The total unit weights were in the range between 1.5g/cm^3 and 1.9g/cm^3 with an average value of 1.7g/cm^3 indicating that the soils were hard at the time of inspection (Table 1).

Consistency Limits

A summary of laboratory test results for the soil samples from three pits is presented in Table 1. The liquid limit (LL) varies from 50% to 69% and averages 60.7%. The plastic limit (PL) is between 15% and 30% with an average value of 22.2%. The plasticity index (PI) which is the difference between liquid limits and plastic limits ranges from 27% to 47.4% with an average of 38.5%. The natural water content is very small ranging from 7% to 11% with an average of 9.6% which is smaller than the corresponding shrinkage limit. The values of natural water content below shrinkage limit values indicate the state of intense dryness and desiccation of the clay at the period the observation was carried out.

Table 1: Physical properties of the Kibaha clay samples

Sample No:	Depth (m)	Grain size (%)				Atterberg's Limits				Bulk Density	Activity	Free swell
		Gravel	Sand	Fines	Clay (<2µm)	LL	PL	PI	SL	(g/cm^3)		
RC1	0.6	11	50	39	34	64	21.0	43.0	12.5	1.87	1.5	150
	1.0	14	71	15	30	63	24.0	39.0	13.3	1.84	1.6	130
	2.0	16	61	23	29	58	23.0	35.0	14.2	1.83	1.3	100
	3.0	5	59	36	33	59	22.0	37.0	14.0	1.67	1.4	130
RC2	1.0	9	42	49	29	69	23.0	46.0	13.1	1.61	2.0	140
	2.0	12	63	24	22	68	28.0	40.0	16.6	1.53	1.8	100
	3.0	1	67	32	27	69	21.6	47.4	14.6	1.87	2.0	100
RB1	1.0	2	65	34	39	54	21.0	33.0	16.5	1.84	0.9	130
	2.0	1	60	39	35	53	15.0	38.0	15.0	1.83	1.2	120
	3.0	3	52	44	34	50	23.0	27.0	15.1	1.67	0.9	140

Swelling Potential identification

The most commonly used method to identify swelling potential is the indirect method where the influence of physical properties of the soils such as liquid limits, plasticity index and colloid contents are matched on established charts by different workers to indicate the degree of swell potential. In this study the plot of plasticity index against liquid limit was according to Casagrande's plasticity chart is shown in Figure 3 while that of plasticity index against colloids contents was according to Seed, *et al.* (1960) is shown in Figure 4. In both cases all samples possessed high to very high swell potential. A number of samples were tested, but to save on space, fewer samples are shown on Figures 3 and 4.

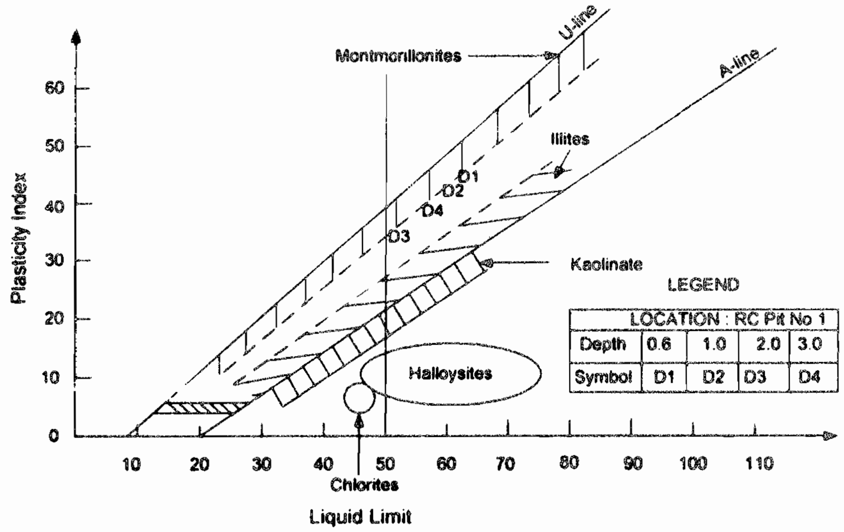


Figure 3: Plot of Clay Minerals at on Casagrande's Chart (Chieborad, *et al, 2005*)

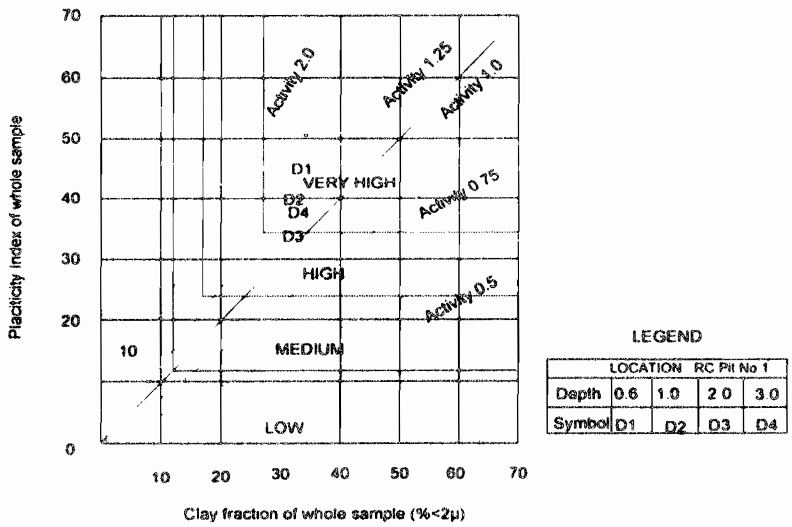


Figure 4: Plot of Clay Minerals at on a Chart for evaluation of potential expansiveness (Seed, *et al.*, 1960)

Free Swell Test

The Swell Index or Free Swell test procedure is used to determine the general swelling characteristics of Soils. The soil specimens were tested to measure the free swell according to Holtz and Gibbs (1956) and standard free swell tests in oedometer cells. For the Free Swell test the specimens are pulverized, put in test-tubes and the initial volumes are measured. Then water is added to each test tube and the powder is allowed to swell freely. The degree of free swell is expressed as a percentage of the increase in volume, ΔV, to the original volume, V, of the soil. The minimum and maximum swelling obtained from the free swell test are 100% and 150% respectively (Table 1). The results indicate that the soils are associated with clay which could swell considerably when wet.

On the other hand, the test in oedometer was conducted on four undisturbed soil specimens. The undisturbed soil samples were cut at their in-situ moisture content, with minimum disturbance to the test specimens, and trimmed to fit in the oedometer The test specimens in the oedometer were saturated and brought to equilibrium under a small surcharge of about 1kPa. The load on the specimen was increased periodically until the height of the specimen returned to original height. For each increment of load, the specimen is allowed to consolidate fully before the application of the next load. The amount of swell is recorded with the dial gauge and the maximum vertical stress necessary to attain original height of sample is the swelling pressure. The tests on Kibaha soils yielded a swell pressure between 45kPa and 50kPa. This is envisaged to be greater than the counteracting pressure from the lightweight structures (Lucian, *et al.*, 2006 and Lucian 2006).

Potential for swelling by the coefficient of linear extensibility (COLE).

The coefficient of linear extensibility (COLE) can be used to characterize the shrink-swell behaviour of soil (Nelson and Miller 1992). Undisturbed clods or cores are briefly immersed in a flexible resin and allowed to dry in laboratory. The resign coating must be impermeable to water but permeable to water vapour (Thomas 1998). The clods are put to field tension of 33kPa or 10kPa tension (1/3- or 1/10-bar tension), weighed in air and water to determine weight and volume using Archimedes principle (i.e. Specific gravity= Mass in air ÷ Loss of weight in water). The clods are brought to oven dryness, their weight and volume measured again. When coarse fragment are present, the COLE (Lucian, 2006) is calculated as follows:

$$COLE_{ws} = \left[\frac{1}{C_m * \left(\frac{\gamma_{d33<2mm}}{\gamma_{d<2mm}} \right) + (1 - C_m)} \right]^{\frac{1}{3}} - 1 \qquad 1$$

Where

$COLE_{ws}$ =Coefficient of linear extensibility on a whole-soil base in cm cm^{-1}

$\gamma_{d33<2mm}$ =Dry Density at 33kPa water retention on a <2mm base (g/cm^3)

$\gamma_{d<2mm}$ =Dry Density, oven-dry or air-dry, on a <2mm base (g/cm^3)

C_m = Coarse fragment (moist) conversion factor.

C_m is calculated as follows:

C_m = [Volume moist <2mm fabric (cm^3)]/ [Volume moist whole soil (cm^3)]

or C_m = (100-Vol>2mm)/100 where Vol>2mm = Volume percentage of the >2mm fraction

If no coarse fragments, C_m=1, and the previous equation reduces to:

$$COLE_{ws} = \left(\frac{\gamma_{d<2mm}}{\gamma_{d33<2mm}} \right)^{\frac{1}{3}} - 1 \quad \text{--} 2$$

According to the calculated COLE, a range of soil swell-shrink potential can be distinguished based on data from Table 2.

Table 2: Ranges of COLE to determine soil swell-shrink potential (Thomas, *et al*, 2000)

Soil swell-shrink potential	COLE
Low	<0.03
Moderate	0.03-0.06
High	0.06-0.09
Very High	>0.09

Table 3 shows results of the coefficient of linear extensibility (COLE) and bulk densities of a few samples selected from the case area. The results indicate that these soils are to be classified having high swell potential. The quantitative swell potential was also estimated by correlating the means of colloids content and the COLE as on Figure 5. Once again, the samples fell in the region of High to very high expansion potential.

Table 3: Calculated Coefficient of linear extensibility (COLE) of samples from measured bulk densities.

Sample	Depth (m)	Bulk density Oven-Dry	33kPa	COLE(cm/cm)	Clay Content (%)
RCC1	1.0	1 87	1 32	0.12	34
RCC1	2.0	1.84	1.24	0.14	28
RCC1	3.0	1.83	1.23	0.14	32
RBB1	1.0	1.67	1.23	0.11	35
RBB1	2.0	1.61	1.25	0.09	33
RBB1	3.0	1.53	1.16	0.10	29

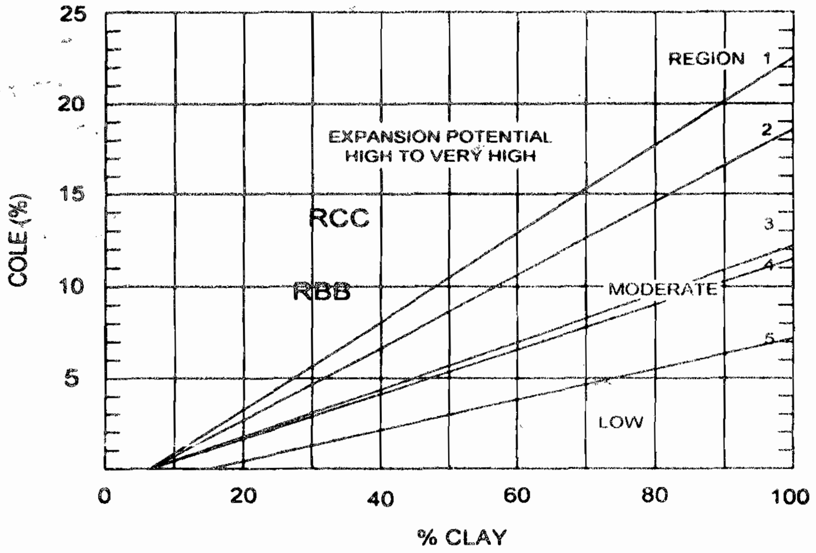
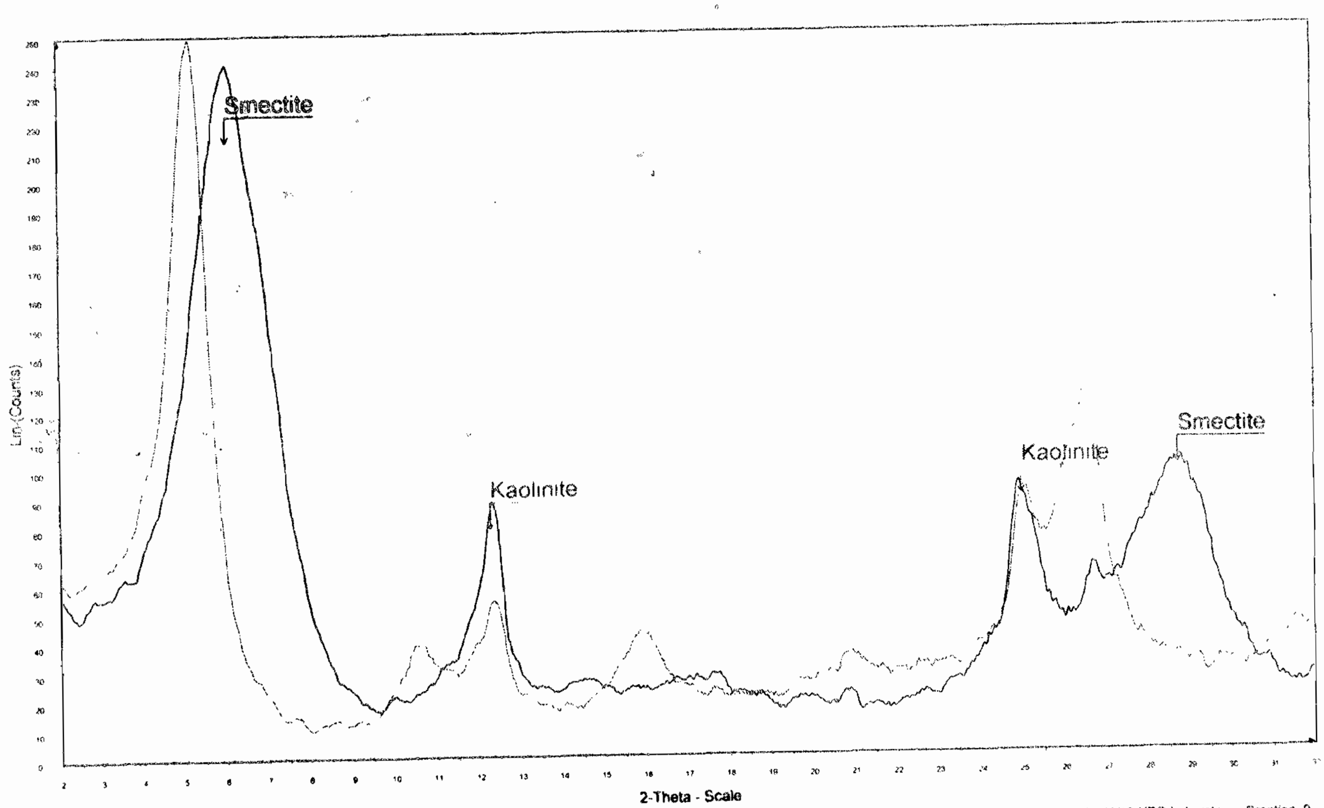


Figure 5: Expansion Potential as a Function of Colloids and COLE (Hardcastle, J. H., 2003)

X-Ray Diffraction (XRD)

For mineral identification, x-ray diffraction via powder technique was carried out according to the method of Brown G. and Brindley G.W. (19849. Typical X-ray diffraction scans for soil sample from the area under study are illustrated in Figure 6. The results show that the soils are predominantly composed of smectite (~60%) with some traces of kaolinite. The high amount of smectite and less kaolinite in the soils indicates a high swelling potential.



Charles Lucian, Sample 2 (S2) Airdried, File:2067701a - Type 2Th/Th locked - Start: 2.000 ° - End: 32.000 ° - Step 0.020 ° - Step time 1.0 s - Anode Cu - Company SEAMIC XRD Laboratory - Creation 0
Operations Smooth 0.296 | Fourier 20.000 x 1 | Strip kAlpha2 0.500 | Import
Charles Lucian, Sample 2 (S2), Glycolated, File:2067701b - Type 2Th/Th locked - Start 2.000 ° - End: 32.000 ° - Step 0.020 ° - Step time 1.0 s - Anode Cu - Company SEAMIC XRD Laboratory - Creation
Operations. Smooth 0.296 | Fourier 20.000 x 1 | Strip kAlpha2 0.500 | Import

Sample 2 (S2)

Figure 6: Diffractogram of a sample soil (Montmorillonite)

CONCLUSIONS

The soils in Kibaha have morphologic, physical and mineralogical properties typical of expansive soils because of the expanding nature of clay lattice (smectite). The geology and climatic conditions in the area favour the formation of smectite clay minerals in soils which are the main cause of ground heave problems. The smectite clay minerals in these expansive soils were developed during the late Mesozoic and early Cenozoic times.

Identification of the propensity of a soil to shrink and swell before construction should be a key to proposing relevant measures to guard against building damages on expansive soils. This paper has presented few techniques which could be employed in the case study to identify the characteristics of swell-shrink soils.

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