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Geotechnical Characteristics Ofexpansive Soilsfrom GRA, Mubi, Adamawa State, Nigeria.

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ABSTRACT

A geotechnical evaluation of the soil in GRA of Mubi, Adamawa State was carried out to ascertain the causes of rampant cracking of walls and slabs on buildings in the area. Properties of the soils studied were the natural moisture content, specific gravity, grain size gradation, consistency characteristics, compaction, X-ray diffraction analysis, permeability, consolidation, and California Bearing Ratio tests. Tests on these properties were mostly on disturbed samples. Test results showed that the soils are silty clays of low to medium plasticity, and of A – 7 class (equivalent to CL of Unified Soil Classification System). The linear shrinkage and liquid limit values tended to classify the soils as marginally swelling in nature. The low value of losses on ignition values also classified the soils as inorganic silt clay. The high group indexes (GI) of 14-15 of the soils rated them as poor subgrade. The results showed that the soils exhibit some swelling characteristics. X-ray diffraction analysis revealed the presence of quartz –α low (silicon oxide – SiO_2), montmorillonite and microcline (potassium aluminum silicate – $K(AlSi_3)O_8$ in the samples. Consolidation test results also showed that structures on the deposit need deeper foundation to make settlement moderate and acceptable.

INTRODUCTION

Recognition of swelling soils and estimation of the swelling potential are the first step in avoiding the damaging effect of swelling soils (Ezeukwu, 1988). Failures of foundations in residential buildings in parts of Mubi town in Adamawa State have been of concern for years. Financial loses occasioned by such failures have been high. The unsightly appearances of cracks in the wall reduce both the value and aesthetics of the buildings; hence the decision to study the geotechnical characteristics of the soil and proffer solution arises from the observation.

There are two broad methods of identifying swelling soils; they are field and laboratory identification.

Field Identification

There are different methods of identifying potentially expansive soils in the field. In the field, expansive clay soils can be easily recognized in the dry season by the deep cracks, in roughly polygonal patterns in the ground surface. The zone of seasonal moisture content fluctuation can extend from three to forty feet deep. This creates cyclic shrink/swell behavior in the upper portion of the soil column, and cracks can extend too much greater depths than imagined by the Engineers. (Rogers et al, 2005). Colour can be used to approximately identify swelling soils in the field, but it is not a good criterion since swelling soils come in different colour. Potentially expansive clays are usually recognized in the field by their fissured or shattered and slickenside condition or by obvious structural damage caused by such soil to existing buildings (Wagener 1975). The appearance of deep cracks indicates the presence of swelling clays in an area. The strength of a soil in a dry state is an indication of its cohesion and hence of its nature. It can be estimated by crushing a 3mm size dried fragment between thumb and forefinger. A clay fragment can be broken only with a great effort, whereas a silt fragment crushes easily. A sample of a moist soil can be manipulated between palms of the hand and finger. If it rolled in to a long thread of about 3mm diameter with no cracks, then the soil contain significant clay, but if the rolled thread had severe cracks it means silt is more. Dispersion test is used for making rough estimate of sand silt and clay present in a material. It is done by dispersing a small quantity of soil in water taken in a glass cylinder and allowing the particles to settle. The coarser particles settle first followed by finer ones. Ordinarily sand particles settled first within 30 seconds, then silts particles in about 30seconds to 4hours and finally clay particles size remain in suspension for about 7hours to few days. (Murthy, 2006)

Gidigasu&Andoh (1980) investigated the effects of topography on swelling clays and come out with a findings that upland soils are generally kaolinitic while valley soils are generally montmorillonitic. Alluvial clay soils in lower slopes and valley bottoms may contain montmorillonite and therefore could be potentially expansive unlike the summit and upper slope soils.

Laboratory Identification

There are two different methods of identifying potentially expansive soils; they are the mineralogical identification and single index method.

- Mineralogical identification

This is useful in the evaluation of the material composition of a soil but is not sufficient in itself when dealing with natural soils. The mineralogical composition of expansive soils has an important bearing on its swelling potential. The negative electric charges on the surface of a clay mineral, the strength of the inner layer bonding and the cation exchange capacity all contribute to the swelling potential of the clay. Hence the swelling potential can be evaluated by identification of the constituent mineral of this clay. The five techniques which may be used are: X-ray diffraction, differential thermal analysis, dye adsorption (methylene blue), thermogravimetry, and electron microscope resolution. The test results require experts' interpretation and specialized apparatus.

- Single index method

Simple soil property tests can be used for the evaluation of the swelling potential of the expansive soils. These tests include the following: Atterberg limits, linear shrinkage, free swell, Colloid content.

Atterberg Limits: As moisture is removed from a fine-grained soil, it passes through a series of states, i.e. liquid, plastic, semi solid and solid. The moisture content of the soil at a points where it passes from one stage to the next are known as consistency limits (Smith, 1978).

1. Liquid limit (LL) - The water content at which the soil changes from the liquid state to the plastic state.

2. Plastic limit (PL) - The minimum moisture content at which the soil can be rolled into a thread 3mm diameter without breaking up. This is also defined as the water content below which the soil stops behaving as a plastic material.

Plasticity index (PI) = LL – PL.

3. Shrinkage limit (SL) - The maximum moisture content at which further loss of moisture does not cause a decrease in volume of the soil. This is the smallest water content at which the soil is saturated. The smaller the shrinkage limit, the more susceptible a soil is to volume change. If the shrinkage limit is 5%, then when the in-situ water content exceeds this value, the soil will begin to expand.

Plasticity index and liquid limit are useful indices for determining the swelling characteristics of most clay (Holts &Gibbs, 1956). Seed, et al (1962) have demonstrated that the plasticity index alone can be used as a preliminary indication of swelling characteristics of most clay. Chen (1975) established a relation between the swelling potential of clays and plasticity index as shown in Table 2.4. Gidigasu and Andoh (1980) reported the importance of liquid limit and plasticity index in the recognition of potential expansiveness of soils and noted good correlation between the field behavior of expansive soils from Ghana and their position on the Casagrande's chart. In the identification, soils with liquid limit above 50% and plasticity index above 20% are considered potentially expansive. The shrinkage characteristics of the clay should be consistent and reliable index to the swelling potential. Altmeyer (1955) presented a guide to the determination of potential expansiveness for various values of shrinkage limits and linear shrinkage as shown in Table 2.1

Soil is an essential component in the construction and stability of a house usually overlooked by designers, contractors, and clients. Structural damage to a house can occur if the foundation soil expands contracts or slides. Often, damage from expansive soils can be seen within the first few months or years after construction. As water from irrigation or rainfall migrates underneath the *house* foundation, the soil around the edge of the foundation expands, pushing up on the edges of the foundation. This condition causes cracks in the foundation, floors and the basement wall. Over a period of years, as the moisture further migrates underneath the center of the slab, center- lift can occur, causing additional damage to the house. Even though expansive soils cause enormous amounts of damage, most people are not aware of them. This is because damage takes place slowly and is hardly attributable to a specific event, but rather to poor construction practices or misconception that all buildings experience this type of damage with age.

A typical example of expansive soil in Nigeria is the black cotton soil of Maiduguri area in Borno State. More of such expansive soils will be identified with increasing construction. There is, therefore, the need to identify this type of soil. This will ensure that appropriate treatment and design consideration are given to forestall the detrimental effect of swelling and shrinkage on civil engineering structures and constructions. The depth of the expansive soil at which adverse periodic changes of moisture content occurs is known as the active zone. In most cases the depth of the active zone is limited to 3 to 4m (Arora, 2008). Crack depths may be much in such formations.

The existence of expansive soil and problems associated with buildings on them is well known among geotechnical engineers and geologists in Nigeria and many parts of the world that need continuous attention. Expansive soils are soils which expand with water intake. The clay mineral montmorillonite is mainly responsible for the swell/shrink characteristics of the soil. The very expansive soils are called swelling soil or black cotton soils. These soils are mostly confined to semi-arid and arid regions of the tropical and temperate climate zones. Expansive soils are in abundance where the annual evapo - transpiration exceeds the precipitation (Ola, 1978).

Expansive soils in many parts of the world pose a significant hazard to foundations for light buildings. Swelling clays derived from residual soils can do considerable damage to lightly-loaded wood-frame structures.

Expansive soils owe their characteristics to the presence of swelling clay minerals. As they get wet, the clay minerals absorb water molecules and expand; conversely, as they dry they shrink, leaving large voids in the soil. Swelling clays can control the behavior of virtually any type of soil if the percentage of clay is more than about 5 percent by weight(Rogers e tal). Soils with smectite clay minerals, such as montmorillonite, exhibit the most profound swelling properties.

Potentially expansive soils can typically be recognized in the laboratory by their plastic properties. Inorganic clays of high plasticity, generally those with liquid limits exceeding 50 percent and plasticity index over 30, usually have high inherent swelling capacity. Expansion of soils can also be measured in the laboratory directly, by immersing a remolded soil sample and measuring its volume change.

In the field, expansive clay soils can be easily recognized in the dry season by the deep cracks, in roughly polygonal patterns, in the ground surface as seen in the figure below. The zone of seasonal moisture content fluctuation can extend from three to forty feet deep. This creates cyclic shrink/swell behavior in the upper portion of the soil column, and cracks can extend to much greater depths than imagined by most engineers.

Figure 1: Nature of the site where sample was obtained for the analysis.

Damage of Foundation

The obvious way in which expansive soils can damage foundations is by uplift as they swell with moisture increases. Swelling soils crack lightly-loaded footings, and frequently cause distress in floor slabs. Because of the different building loads on different portions of a structure's foundation, the resultant uplift will vary in different areas.

Concrete drainage devices can be adversely affected by expansive soils. Swelling clays can lift and crack concrete ditches, seriously impairing their ability to convey runoff. Subsequent contraction may leave a void under the concrete, leading to piping and erosion as runoff flows under the ditch.
Expansive soils pose the greatest hazard in regions with pronounced wet and dry seasons. The annual cycle of wetting and drying causes soils to shrink and swell each year. Thus, the arid regions of the country are much more susceptible to damage from expansive soils than regions that maintain moist soil conditions throughout the year.
The biggest problem in expansive soil areas is that of difference in thewater content. Sources of water in developed areas are not limited to temporal weather cycles, but can be introduced by people. A frequent source of damage is the differential swelling caused by pockets of moist soil adjacent to dry soil. For example, lawn and garden watering creates a moist zone on the exterior of a foundation, whereas the interior is dry; this creates differential swelling pressure on foundation elements. There is frequently a moisture differential between the soils beneath a house and thosethat are more directly exposed to changes in the weather. Cesspools, leaky pipes, and swimming pools are other common sources of water.

Table 1: Consolidation Test Results of the Reconstituted Soil

Coefficient of volume compressibility, a_v (kN)	2.50
Coefficient of volume change M_v (m^2/kN)	1.923×10^{-4}
Coefficient of consolidation C_v (m^2/year)	4.457
Coefficient of permeability K_v (m/sec.)	8.41×10^{-4}

Table 2: Laboratory Report of chemical Analysis of the Reconstituted Soil Sample

Sample ID	SiO_2	Al_2O_3	Fe_2O_3	CaO	K_2O	Na_2O	TiO_2	Au	MgO	others	LOI
% oxide	61.0	19.0	2.2	1.0	2.0	1.5	1.0	0.5	0.4	0.3	11.1

Table 3: Index Properties of Soil Samples A, B and C

Test	A	B	C
Natural moisture content, %	14.0	12.0	13.0
Samples passing No.200 sieve %	64	60	63
Liquid limit, %	49.0	48.0	50.0
Plastic limit, %	14.0	15.0	13.0
Plasticity index, %	35.0	33.0	37.0
Linear shrinkage, %	14.0	15.0	13.5
Maximum dry density, kg/m^3	2160	2070	1970
Optimum moisture content, %	16.1	26.4	13.4
Specific gravity	2.4	2.41	2.54

Table 4: California Bearing Ratio of the three samples

Additives		CBR (%)		
Soil Samples		A	B	C
0%		15.0	13.2	15.4
3%	Cement	20.4	18.5	19.5
	Lime	18.3	17.0	16.6
5%	Cement	26.3	27.5	29.0
	Lime	19.5	20.4	21.2
7%	Cement	34.3	33.6	36.4
	Lime	23.2	21.8	23.0
9%	Cement	42.7	45.7	47.6
	Lime	34.0	36.2	33.0
12%	Cement	58.3	61.4	63.6
	Lime	48.2	45.7	50.4

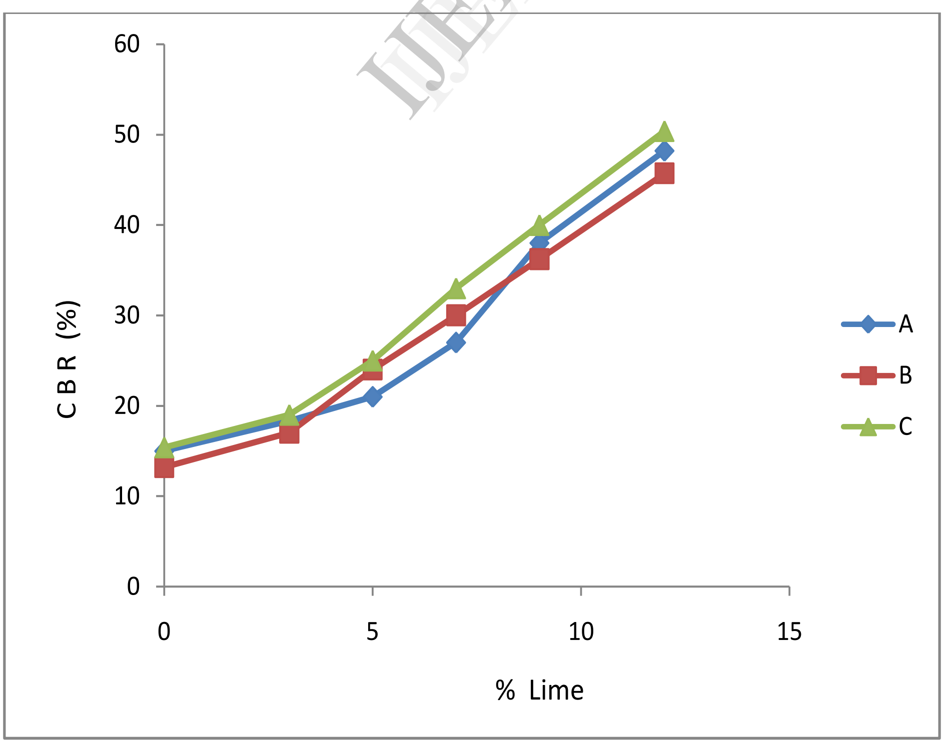

Figure 2: California Bearing Ratio of Lime – Stabilized Samples

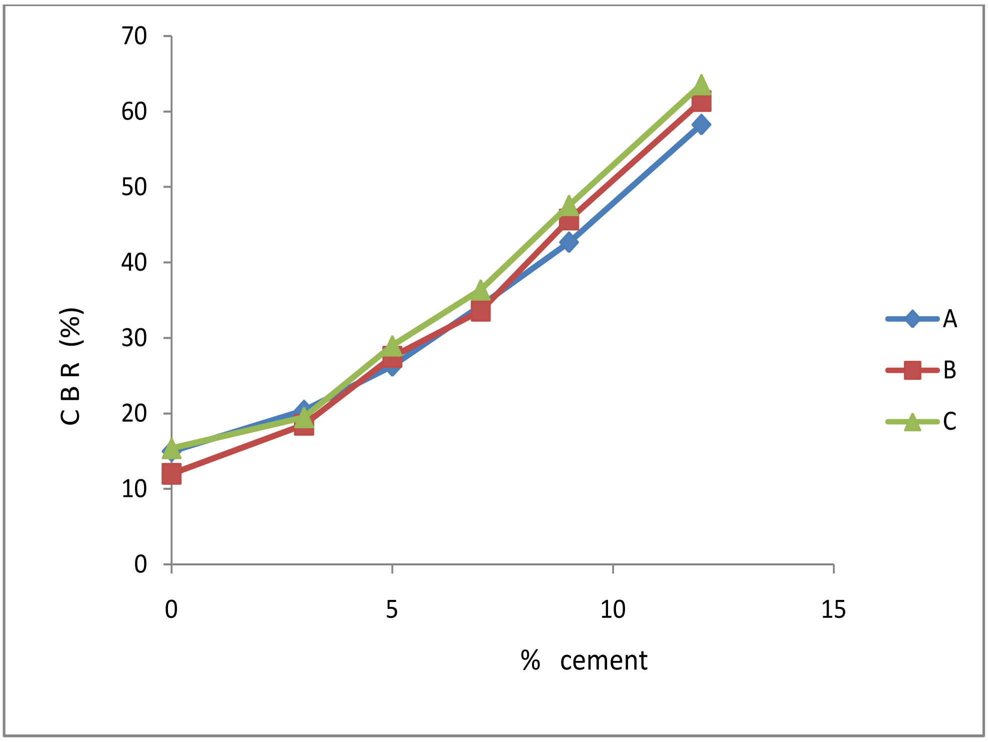

Figure 3: California Bearing Ratio of Cement – stabilized Soil Samples

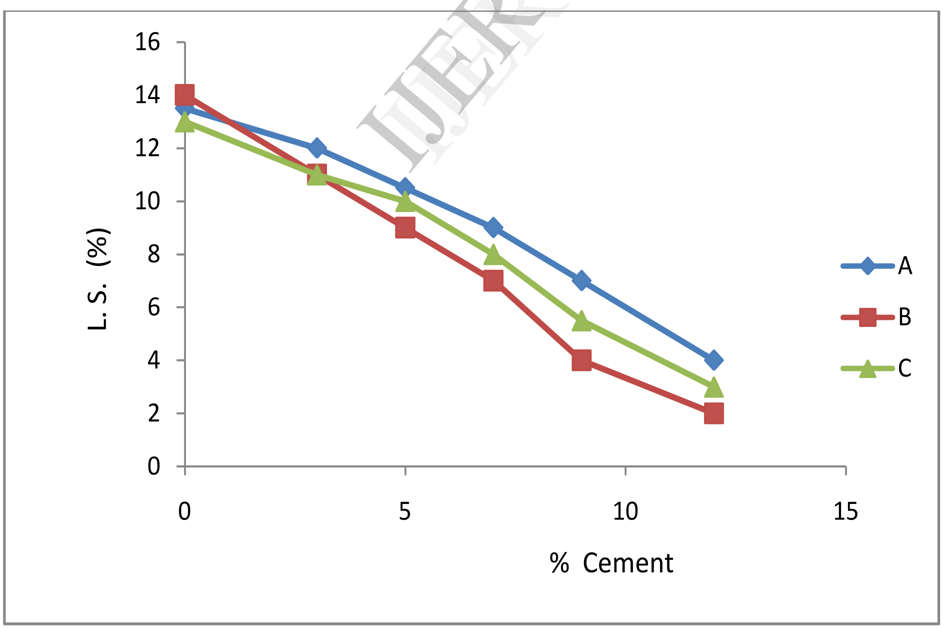

Figure 4: Linear Shrinkage of Cement – Stabilized Soil Samples

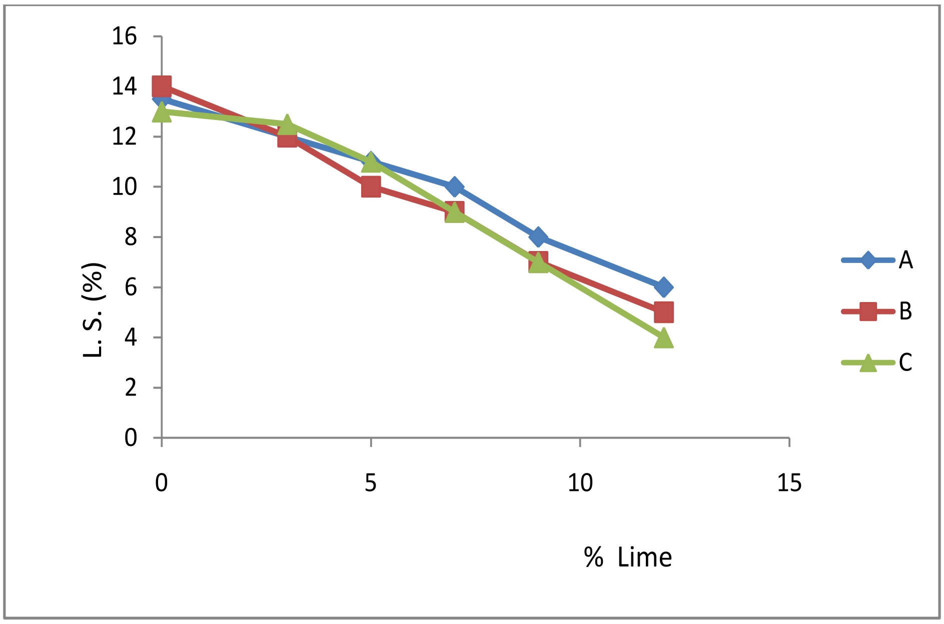

Figure 5: Linear Shrinkage of Lime – Stabilized Soil Samples

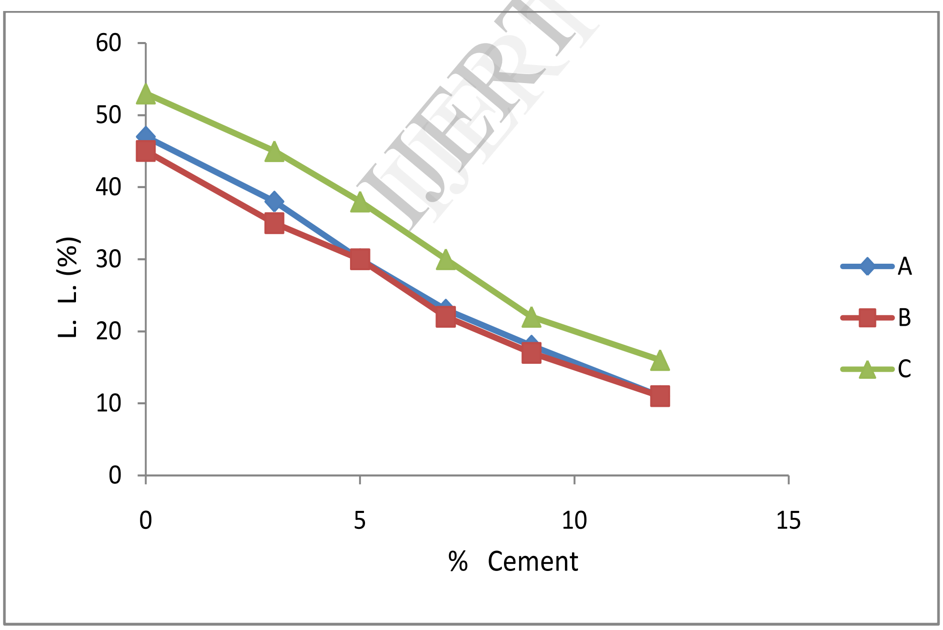

Figure 6: Dependence of the Liquid Limit of the Soils on the Cement Content

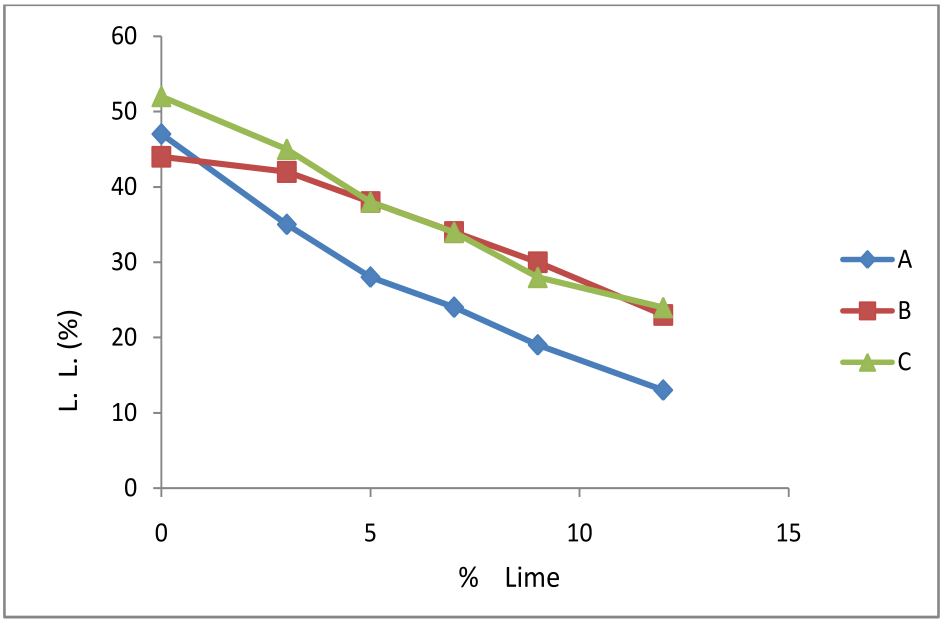

Figure 7: Dependence of the Liquid Limit of the Soils on the Lime Content

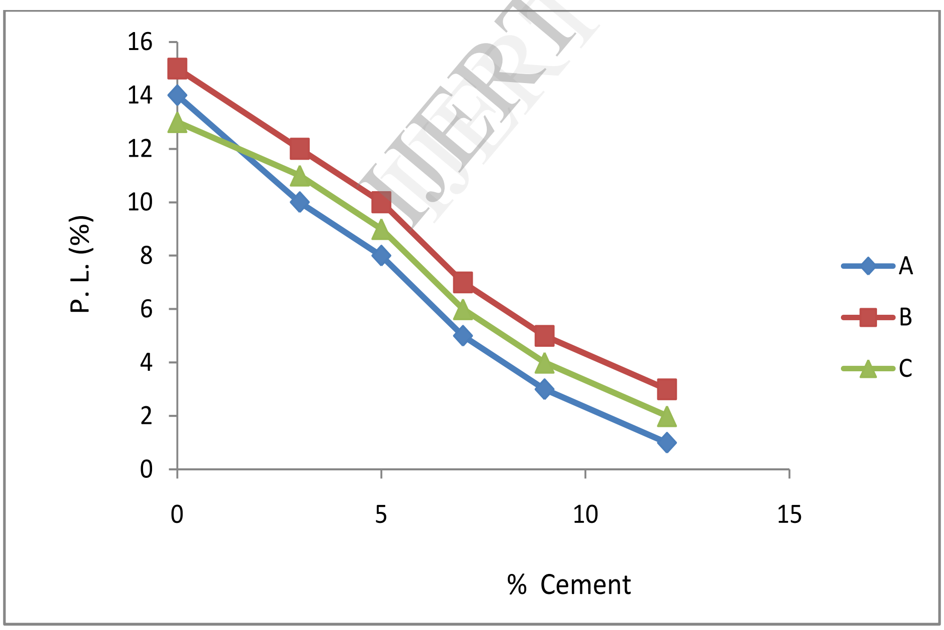

Figure 8: Plastic Limit of Cement – Stabilized Soil Samples

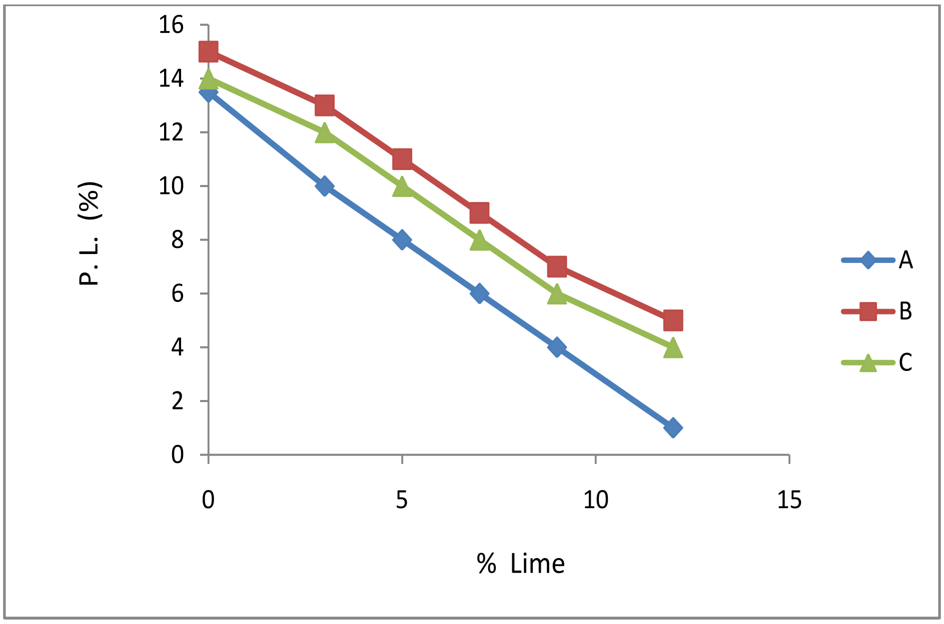

Figure 9: Plastic Limit of Lime – Stabilized Soil Samples

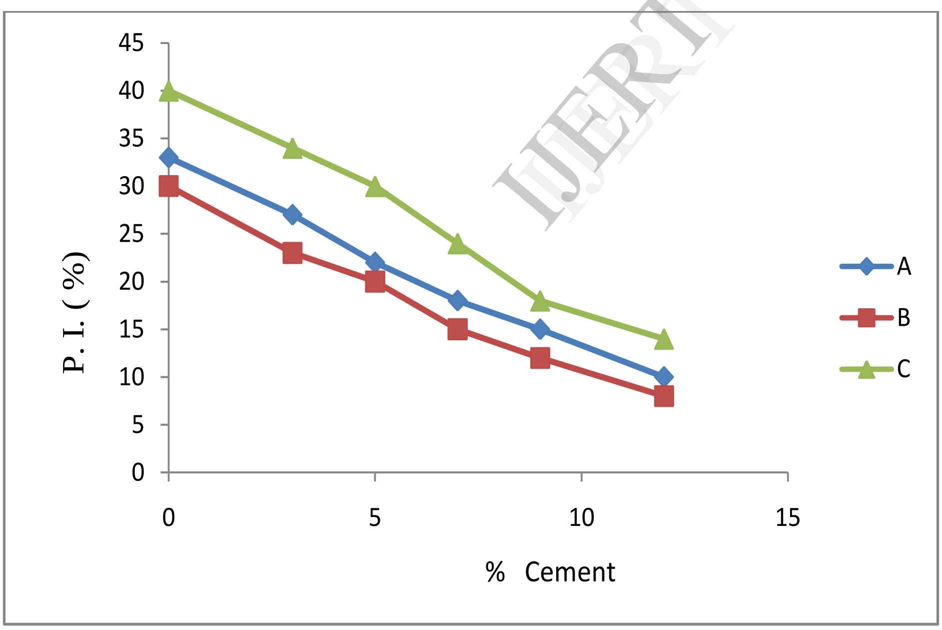

Figure 10: Plasticity Index of Cement – Stabilized Soil samples

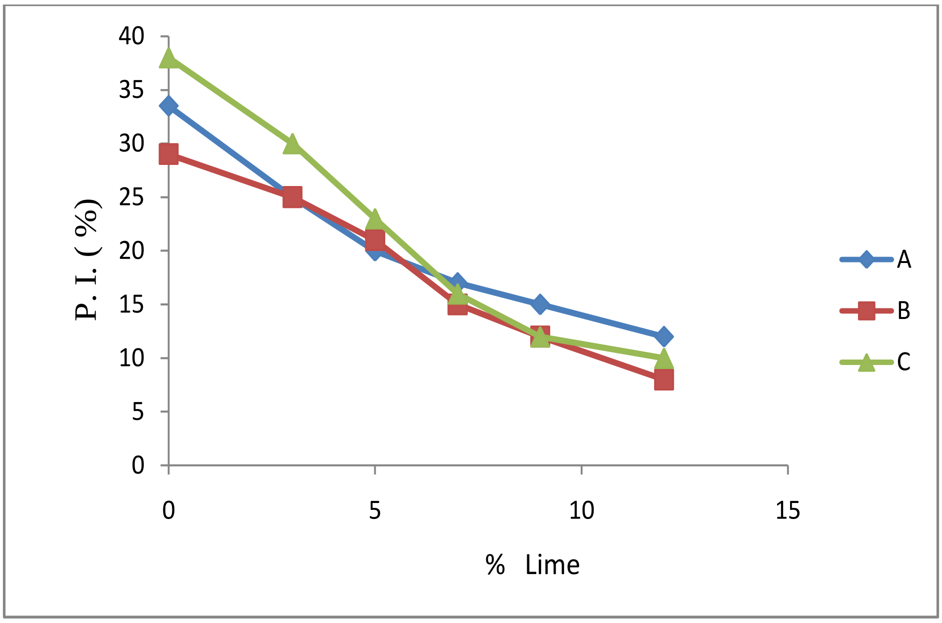

Figure 11: Plasticity Index of Lime – Stabilized Soil samples

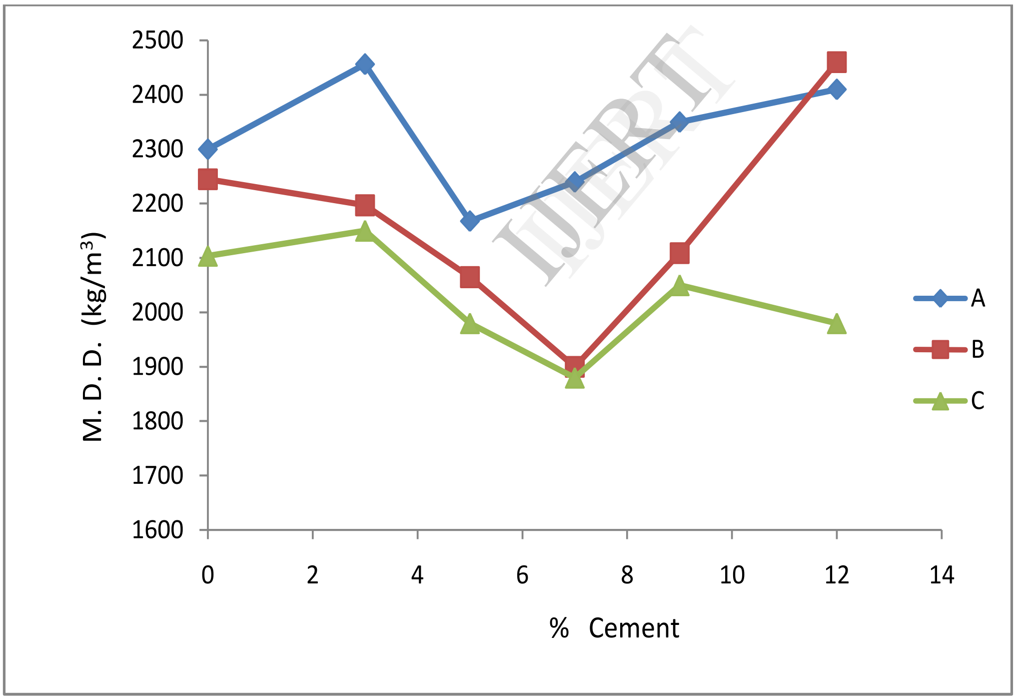

Figure 12: Relationship between the Cement Content and the Maximum Dry Density of Soil Samples

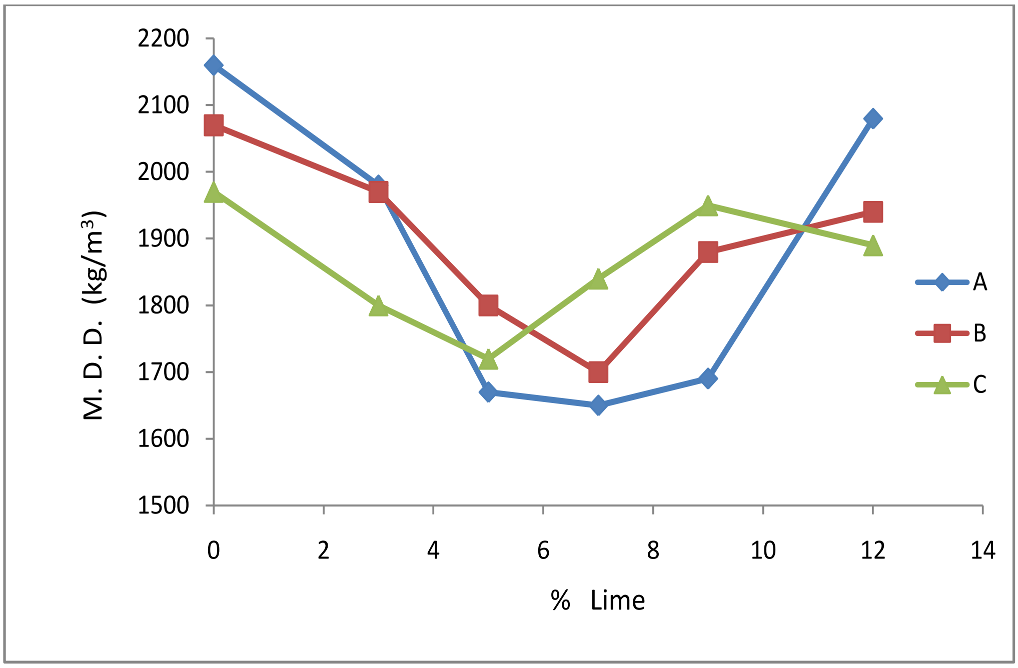

Figure 13: Relationship between the Lime Content and the Maximum Dry Density of Soil Samples

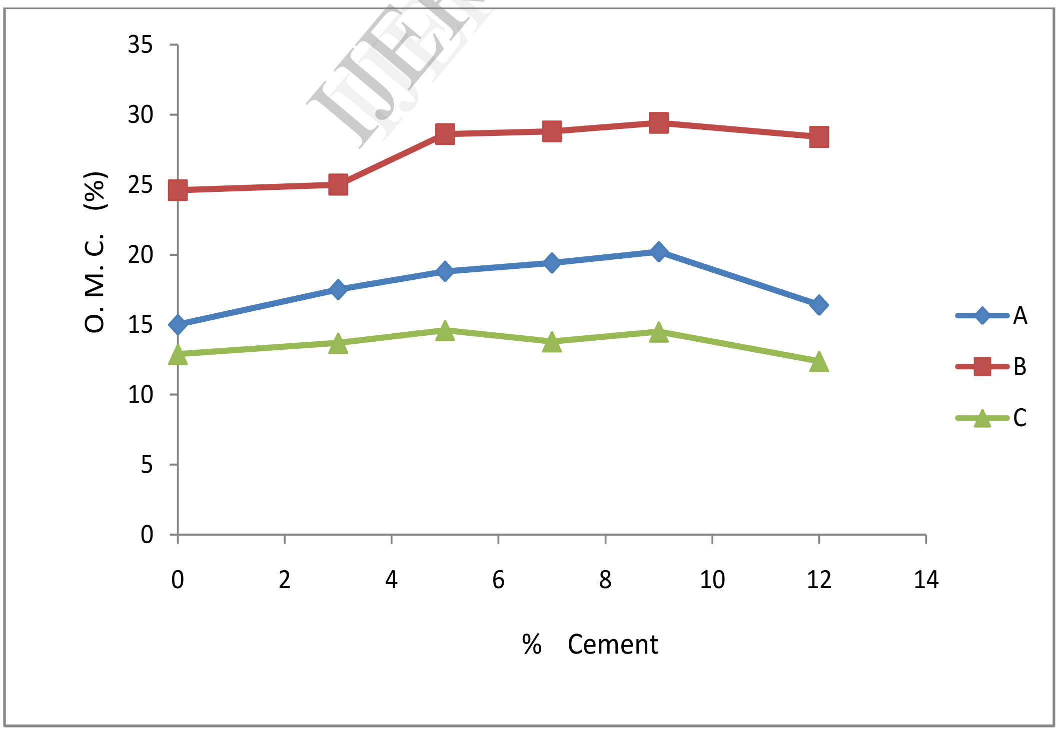

Figure 14: Optimum Moisture Content versus Cement Content of the Stabilized Soil Samples

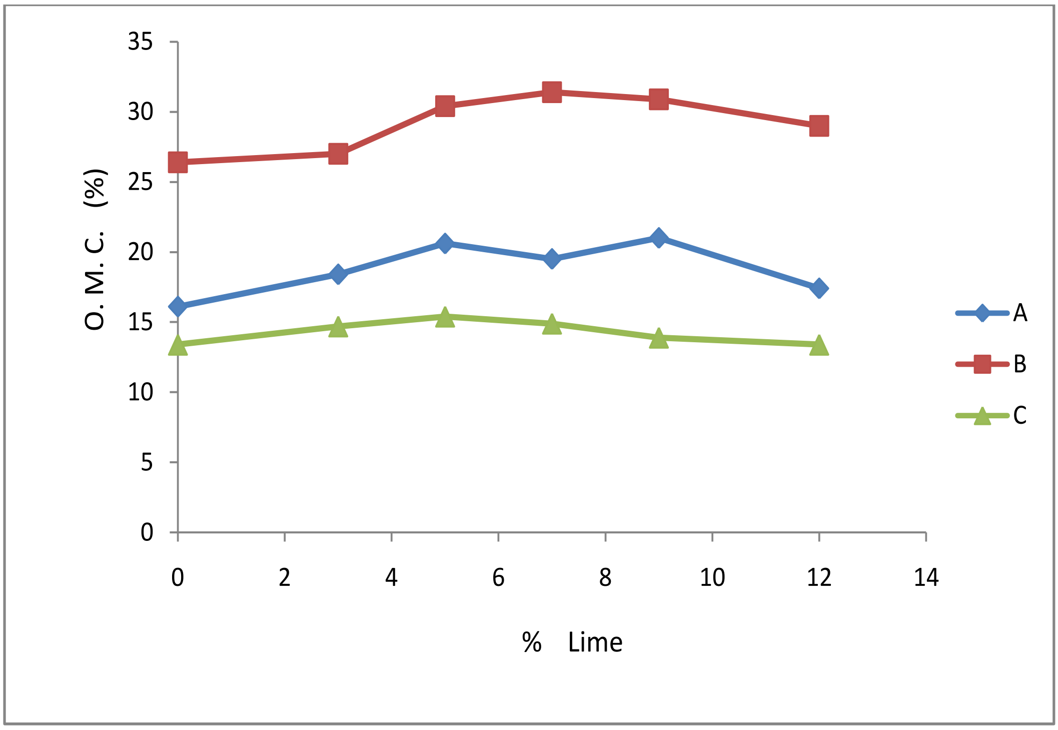

Figure 15: Optimum Moisture Content versus Lime Content of the Stabilized Soil Samples

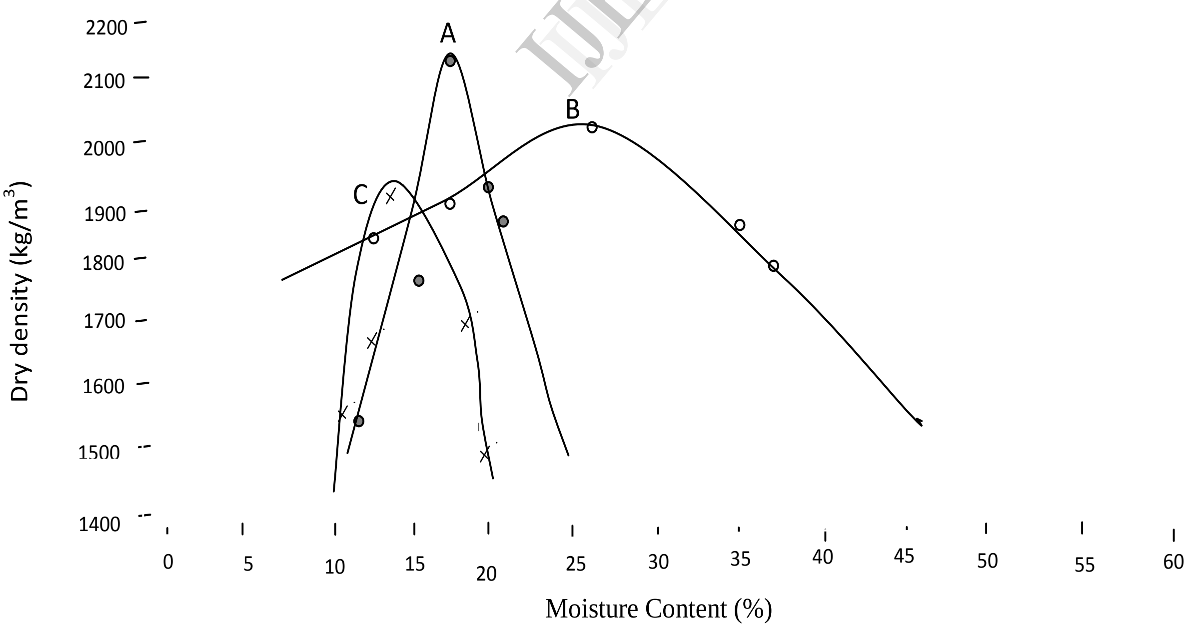

Figure 16: Moisture, Density Relationship for the three Soil Samples.

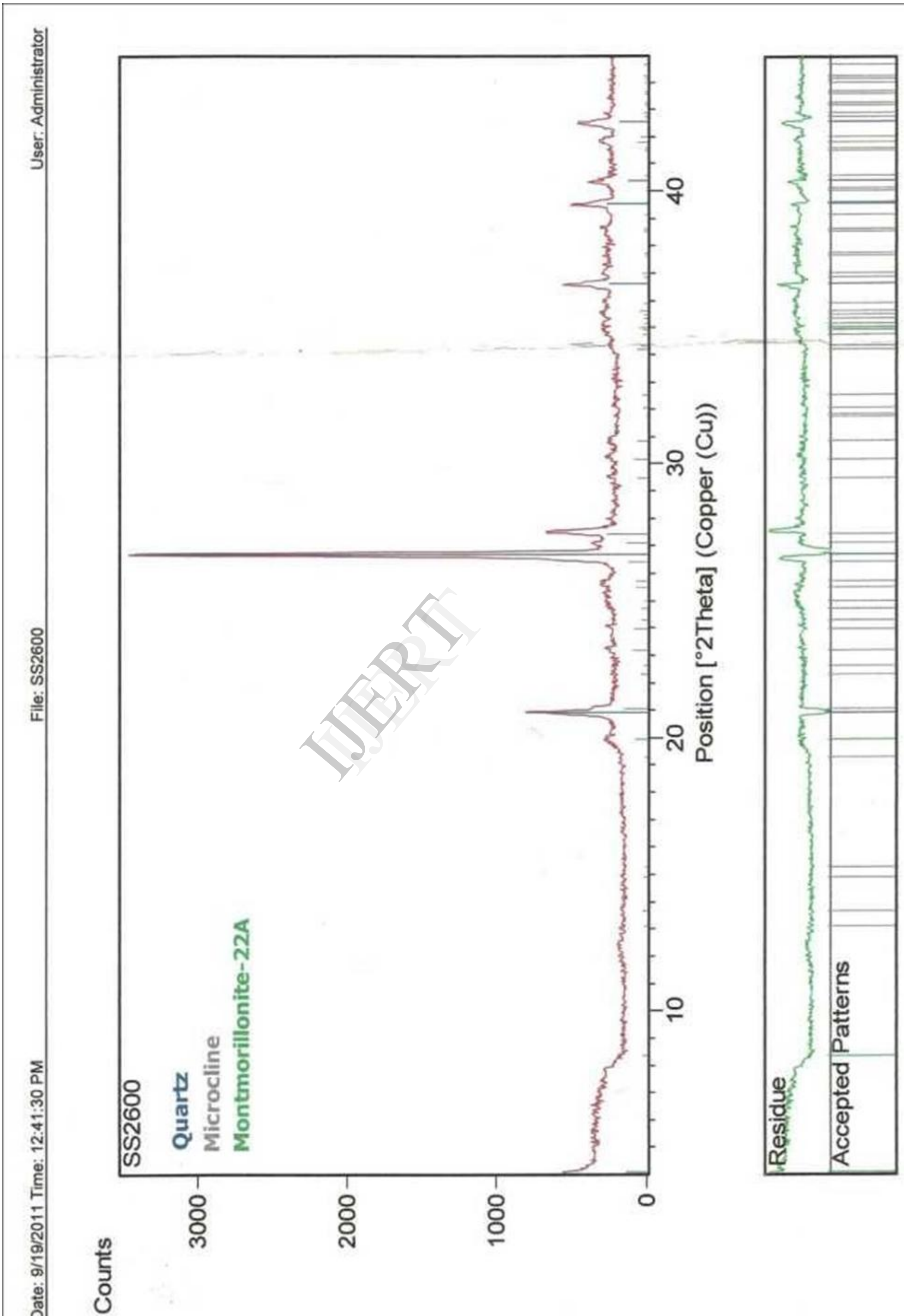

Figure 17: X-ray Diffraction analysis Chart for the sample

LABORATORY TEST RESULTS

The grain size distribution curves for the three soil samples used are shown in Fig 16. About 64 %, 60 %, and 63 % of the sample A, B and C respectively passed No.200 B.S sieve. The corresponding clay fractions are 11%, 19 and14%.

Table 3 shows the summary of the Atterberg limit tests carried out. For soil sample A, the liquid limit is 49 %, plastic limit is 14.3 %, plasticity index is 35 %, specific gravity is 2.40 and natural moisture content 13.8 %. For sample B, the liquid limit and plastic limit are 48 % and 15.3 % while plasticity index, specific gravity and natural moisture content are 33 %, 2.41 and 12 % respectively. For sample C it has liquid limit 50 %, plastic limit is 13 %, plasticity index 36.5 %, specific gravity 2.54 and its natural moisture content as 13 %.

From Table 3 the maximum dry density is 2160 kg/m^3 and the optimum moisture content is 16.1 %, for sample A, and for sample B, the corresponding values are 2070 kg/m^3 and 26.4 %. For sample C the corresponding values are 1970 kg/m^3 and 13.4 %.

Quick undrained triaxial test was performed at cell pressure of 100, 200 and 300 kN/m^2. The Mohr diagram is presented in Fig. 4.3. The apparent cohesion c is 10, 32 and 12 kN/m^2 and angle of internal friction φ is 12, 21 and 15^o respectively for soil sample A, B and C.

The pattern on the chart of X –ray diffraction analysis shows that some minerals was detected at 2theta of 20.8, 26.7, 27.3, 36.7, 39.5, 40.3, and 42.5 degrees with its corresponding d- spacing of 4.27, 3.34, 3.26, 2.45, 2.45, 2.28, 2.24, and 2.12 Å , using tables in Pei (1977). The analysis reveals quartz –α low (silicon oxide – SiO_2), and monmorillonite – 22 Å. The CBR of the soil samples for A , B, and C are 15, 13.2 and 15.4 respectively.

DISCUSSION OF RESULTS

Based on the grain size distribution, the soil studied was classified as A - 7 - 5 on the AASHTO classification scheme, equivalent to CL of Unified Soil Classification System. The high group indexes (GI) of 14-15 of the soils rated them as poor quality for subgrade.

In a soil sample, clay fraction is responsible for expansive characteristics. The amount of expansion however depends on clay mineral type and amount. Classification of samples based on the index properties according to O'Neill and Poormoayed (1980) placed the samples as high expansive in the following decreasing order, C, A, and B.

Using only linear shrinkage as a criterion (Altmeyer, 1955) would identify the samples as potentially expansive in the following decreasing sequence B, C and A. According to (Holtze,1969) the samples would be identify as having a high swelling potential in the following increasing order B, A and C on the basis of their plasticity index. On the basis of their shrinkage limit they all fall in to medium swell potential. Using percentage passing sieve No.200 (0.075mm), the order of expansiveness would be samples A, C and B. According to Chen (1988) samples A and C would be identified as having very high degree of expansion since their plasticity index are 35 and 37% respectively while B having 33% is high degree of expansion. From the plastic index proposed by Chen (1988) the soil falls into medium to high swelling potential. Since the dry densities for samples A, B and C are 2160 kg/m^3,2070 kg/m^3, and 1970 kg/m^3, he suggests that it will generally exhibit the characteristic of high swelling potential. The presences of gravel in the samples as shown in the sieve analysis provide strength to the soil. To mix the soil with gravel may improve the strength (MDD). Soil samples are sticky during compaction due to low permeability, this show seepage can occur in this type of soil. The soil show moderate swelling characteristic as a result of free swell, this may be as a result of the acidic nature of the soil. The LL and PI of the soil samples stabilized with lime and cement decrease, which indicate that swell potential is reduced. The lime and cement chemically react with the soil constituents and reduce its activity as well as providing bonding to the soil particle as to reduce the swell potential thereby increase the overall strength of the soil.

The presence of quartz in the sample is confirmed by the characteristic reflection at 4.27 Å of the x-ray diffraction pattern. The sample also contains feldspar which is indicated by the presence of characteristic reflection at 3.26 Å and montmorillonite-22 Å. Also, the presence of microcline is conferred by characteristic reflection at

3.26 ÅThe high quantity of Al_2O_3 – 18.8, SiO_2 – 61.4, K_2O – 1.77, Fe_2O_3 – 2.24, and Na_2O – 1.46 suggest the high quantity of quartz, feldspar in the samples.

ENGINEERING SOLUTIONS TO THE PROBLEMS ASSOCIATED WITH EXPANSIVE SOIL.

The study has established the characteristics of the expansive soil necessary for design to forestall detrimental distortions of structures in the expansive soil in the study area. Expansive soils are generally not amenable to treatments for preparation of structure foundations by such methods as pre – saturation or lime stabilization. The normal problem associated with swelling soils is soil volume changes due to wetting and drying.

Foundation evaluation in problem soils in Mubi and elsewhere, therefore, generally is not based on rigorous mathematical analysis but rather on empirical consideration with the objectives to either avoid or minimize the problem.(Hunt, 1986). Most important in the selection of suitable foundation treatment is the recognition that the potential problem exists, and the evaluation of its magnitude. Foundation design in the expansive soil should ensure:

1 The foundations withstand the impact of heave stresses.
2 The heaves of expansive foundation soil is reduced by various methods, such as:
i. Providing granular bed and granular cover below and around the foundation as shown in Fig. 5.1.
ii. Treatment of the expansive foundation soil with lime.
iii. Provision of a layer of cohesive non-swelling soil (CNS) materials of sufficient thickness over the expansive soil, before laying the foundation in canal lining, etc.

3. Deeper foundations carry the load to the stable zone/outside or below the active zone of seasonal moisture variation as in Fig. 5.2. The active zone is therefore the depth of the soil over which soil – moisture deficiency exists. Few observations in wells in Mubi show that this depth extends to greater than 2 m. The depth is a function of the soil type, soil structure, climate and topography. An increase in soil moisture in this zone can cause significant uplift forces to develop along the pile shaft from surrounding soils unless adequate protection is provided.

Structures resting directly on the expansive soil should be capable of withstanding stresses caused by the heave of the wet soil. Spread and continuous footings may be founded on compacted on free – draining geo-materials after partially excavating the “active” clay where swelling pressures are likely to develop. Continuous footing may be stiffened through heavy reinforcement to reduce structural distortion.

Fig.5.1 is the case of a typical shallow foundation in the expansive soil. The excavation should be advance to depth greater than the foundations (by about 20 cm). Freely - draining cohesionless soil - gravely sand mixtures – is filled and compacted up to the bottom level of the foundation. Reinforced concrete footing is placed at this level, over which the sandcrete wall should be raised. An RCC apron with light reinforcement and about 2 m wide is provided around the building to prevent moisture from directly entering the foundation. The cushion of a granular material below the foundation will absorb the effect of the swelling (the soil breathes) and so reduce its detrimental effects on the RCC foundation. Waffle mats or stiffened slabs can also be used on the expansive soil, and this construction has been successfully used elsewhere on similar soil for residential houses (Hunt, 1986). The waffles are shallow beams which give the mat rigidity and although the mat will rise as the soils swell, it will move as a unit and the structure will not be distorted. Differential movement inevitable between the structure and utilities can be addressed by providing the latter with adequate flexibility.

In few cases in Mubi (study area), swelling is reduced if the swelling soils are excavated from below the floor area and replaced with crushed stone or gravel. This allows the soil to “breathe” and when the soils expand, they squeeze into the gravel voids and their swelling pressures are reduced. Free water should be prevented from entering the gravel and saturating the soils. Paving the area around the structure would prevent rainfall infiltration and helps to reduce the problem.

Deep Foundations.

The safest solution to the foundation problem in the study area obviously would be to extend foundation through the active zone to the inactive zone, not subjected to seasonal moisture fluctuation. Bored piles extending through the active zone for support are the most successful foundation solution in expansive materials under many conditions. (Hunt, 1986). Protection against swelling uplift could be provided by:

1. placing sufficient steel in the pile to resist tension
2. painting casing left in place with a graphite-based lubricant
3. Isolating the pier from the clay.

Heave beneath floor

Reduction of heave beneath floors can be achieved by excavating the expansive soil from beneath and replacing it with gravel or crushed stone. Water should not be allowed to enter the granular fill and saturate the soil. The area around structure should be paved to prevent rainfall infiltration and so reduce the problem. However, supporting the floor structurally, providing space (air gap) between the floor and ground to permit expansion (see Fig.5.1) appears to be the safest solution.

Pavement over expansive soils

Measures for pavement construction over the expansive soils would include:

1. Excavating a portion of the "active zone" soil and replacing with granular materials like crushed stone as for floor. Alternatively replace with the excavated soil compacted on the wet side of the optimum moisture, especially where granular materials are uneconomical to obtain.
2. Prevent surface water from getting into the pavement and provide adequate under-drainage, and
3. Buried pipes and culverts should be given adequate treatment.

Though the use of lime on expansive soil is beneficial, the problems associated with mixing the lime with the soil and the long period for effective stabilization practically precludes the use of this technique on expansive soil.

Provision of Deeper Foundation

The most successful foundation solution in expansive soil would be the use of bored piles bored through the "active zone" into the inactive zone for support i.e the use of bore piles

Under – reamed piles are performing well in the expansive soil foundation. Typical piles with single under – ream that have been successfully used in expansive soil are shown in Fig. 5.2 the bulbs or under – reams of such single under – ream is placed below the "active zone" (below the zone of season moisture content fluctuation). The pile is extended into the stable zone by sufficient depth to take care of the uplift pressure exerted on the pile shaft in the unstable or active zone.

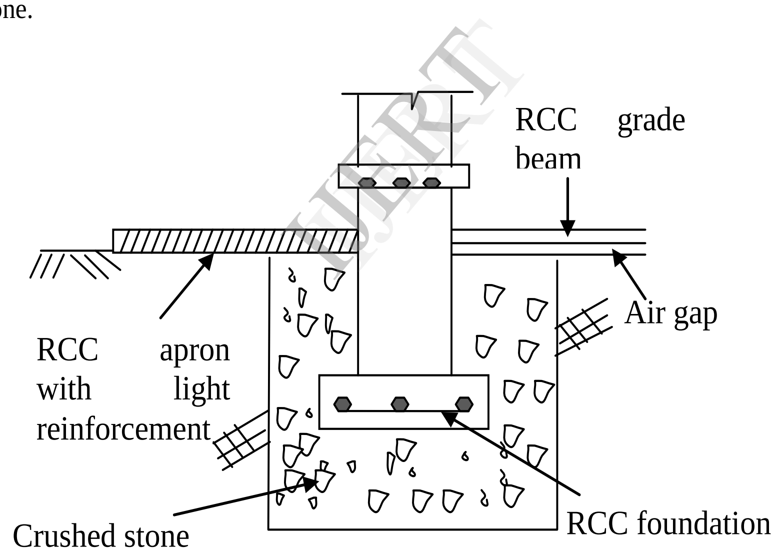

Figure 17: Typical shallow foundation in the expansive soil

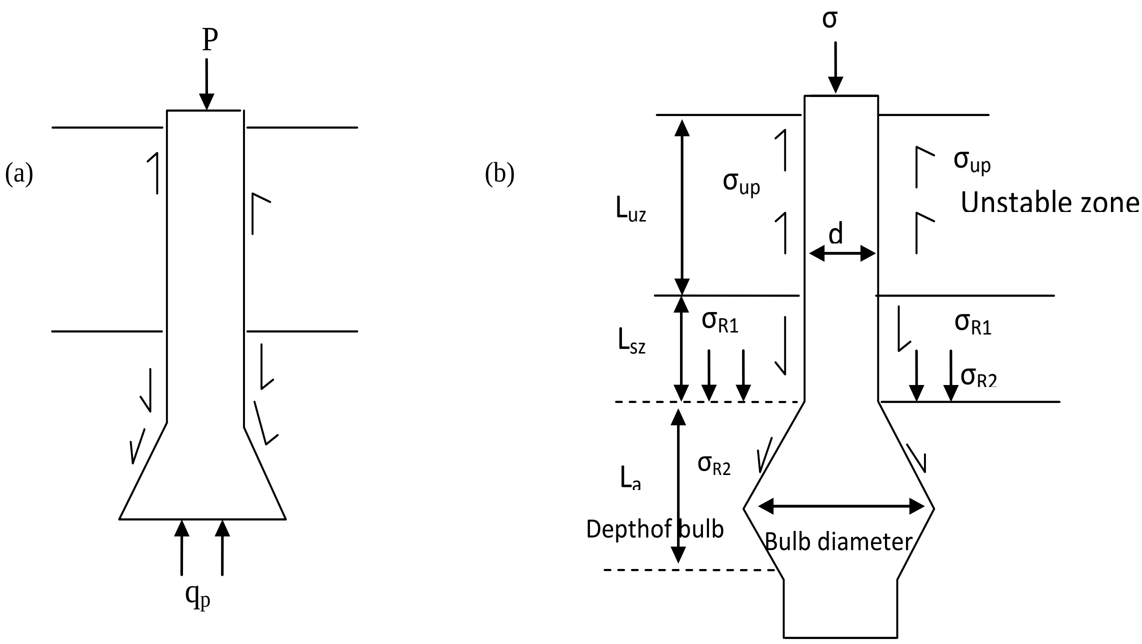

Figure 18: Typical single under – reamed piles for use in the expansive soil.

Design of the under – reamed pile

To design the under – reamed pile shown in Fig.5.2b, the following procedure were taken.

Parameters needed are : L_{uz} = length of pile shaft of diameter d in the top active zone.

$L_{sz}$is the length of the shaft in stable (inactive zone) and L_{db} is the depth of the bulb of width d_u. Let Ϭ be the vertical load (weight of the structure).

When the soil is wetted, as during the wet season, the soil expands. The expansion is wholly or partly prevented by the rough surface of the shaft of the pile of length L_1. Due to the effect of heaving an upward force is caused on the surface of the shaft (this will try to pull the shaft up and out of its position. The pull, however is resisted by the downward dead load Ϭ acting on the pile top and the resisting force from the shaft length L_2 (i.e. $Ϭ_{R1}$) and the bulb of diameter d_u (i.e. $Ϭ_{R2}$) within the inactive zone.

The stability of the pile under the action of the above forces is considered for these two cases in order to work out the worst case.

(a) When no downward load acts on the pile top. A factor of safety of 1.2 is considered sufficient for this case, and
(b) When Ϭ is also acting on the under – reamed piled top. A factor of safety of 2.0 is applied in this case.

The disturbing force $Ϭ_{up}$ (heave or uplift force) is given by:

$$\mathbf{Q_{up}} = \prod \mathbf{d.\,L_{uz}c_a} ----------------(\mathbf{1})$$

where$\boldsymbol{c_a}$ = unit adhesion between the soil and shaft. This is taken to be equals to the shear strength of the clay if the shaft is rough.

the total resisting force σ_R is the sum of the resisting forces, σ_{R1} due to skin friction on the shaft length L_2 and σ_{R2} due to the reaction provided by the soil above the bulb within annular surface of the area of $\frac{\Pi}{4}(\mathbf{du^2 - d^2})$, so we have that:

$$\mathbf{Q_{R1} = \prod dL_{sz}c_a} ------------------(\mathbf{2})$$

$$\mathbf{Q_{R2} = q_d \frac{\prod}{4}(du^2 - d^2)} ----------------(\mathbf{3})$$

whereq_d = c_b.N_c = 6c_b for all practical purposes; c_b = cohesive strength of the soil at the bulb level and N_c = 6.

$$\mathbf{Q_{R2} = 6c_b \frac{\prod}{4}(du^2 - d^2)} --------------(\mathbf{4})$$

For the two cases of stability, we have:

(a) No pile load **Q** and F.S. = 1.2

$$\frac{Q_R}{Q_{up}} = F.S. = 1.2$$

i.e.$\boldsymbol{Q_{up}} = {}^{\boldsymbol{Q_R}}/_{\mathbf{1.2}}$

$$= \frac{\mathbf{1}}{\mathbf{1.2}}\lfloor\mathbf{Q_R}\rfloor = \frac{\mathbf{1}}{\mathbf{1.2}}[\mathbf{Q_{R1} + Q_{R2}}]$$

$$\mathbf{\prod dL_{uz}c_a - Q = \frac{1}{1.2}\left|\prod dL_{sz}c_a + 6c_b\frac{\prod}{4}(du^2 - d^2)\right|} ---------- (\mathbf{5})$$

(b) When top pile load Q acts and F.S. = 2.0

In the case:

$\frac{Q_R}{Q_{up} - Q} = 2$and so

$$Q_{up} - Q = \frac{1}{2}|Q_R| = \frac{1}{2}|Q_{R1} + Q_{R2}| ---------- (6)$$

or$\mathbf{\prod dL_{uz}c_a - Q = \frac{1}{2}\left|\prod dL_{sz}c_a + 6c_b\frac{\Pi}{4}(du^2 - d^2)\right|} ----- (\mathbf{7})$

Eliminating L_{uz} term using Eqns (5.13) and (5.15) together , we obtain

$$\mathbf{Q + \frac{1}{2}\left|\prod dL_{sz}c_a + 6c_b\frac{\prod}{4}(du^2 - d^2)\right|}$$

$$= \frac{\mathbf{1}}{\mathbf{1.2}}\left|\prod \boldsymbol{dL_{sz}c_a} + \mathbf{6}\boldsymbol{c_b}\frac{\prod}{\mathbf{4}}(\boldsymbol{du^2 - d^2})\right| ------------ (\mathbf{8})$$

Eqn. (5.16) can be used to obtain the values of L_{sz} for the selected values of shaft diameter d and bulb diameter du and using the known values of c_a, c_b and Q while designing the boredpile in the expansive soil.

CONCLUSION

The foundation failures and subsequent cracking associated with the building investigated in Mubi are due to the expansive nature of the soil and softening of the strata underlying the foundations when it absorbs water. The expansive soils owe their characteristics to the presence of swelling clay minerals. It is recognized in the laboratory by their plastic properties and in the field by their deep cracks in roughly polygonal pattern in the ground surface in the dry season as shown in figure

The most obvious way in which expansive soil damage foundations is by uplift as they swell with moisture increase thereby causing damage to the buildings and roads in the form of cracks on the walls, driveways, sidewalk, and distress in floor slabs.

However, researchers have the ability to recognize swelling clay soils and take appropriate measures that can withstand the effect. Potentially swelling soils are prevalent and must be recognized early before construction through the conduct of soil laboratory tests to obtain the geotechnical characteristics.

RECOMMENDATIONS

When the soil is allowed to remain in its natural state or the moisture is applied uniformly the building will not experience severe distress. The following steps should be taken to improve the situation with regard to foundation support:

(i) In order to have uniform settlement, the ground surface should be paved with concrete for a reasonable distance around the foundation.

(ii) All drains around the buildings should be leak-proof and should be made to intercept and divert roof and surface runoff to safe outlet well away and downhill of the buildings.

(iii) The materials could possibly be removed to some certain depth below the foundation and replaced by a stable material such as laterite.

(iv) The best way to avoid damage from expansive soils is to extend building foundations beneath the zone of water content fluctuation. The reason is twofold: first, to provide for sufficient skin friction adhesion below the zone of drying; and, second, to resist upward movement when the surface soils become wet and begin to swell.

(v) Septic tanks, soakaways and other water containing structures should be built away from the building or downhill of the building. It is important to regulate water applied near the foundation.

(vi) Underlying strata could be stabilized with cement or lime to strengthen the soil and provide good base for road construction.

(vii) Proper landscape installation and maintenance practice should be followed.

(viii) It is recommended that the design and construction of all foundations and earthworks be carried out in accordance with the codes of practice such as British Standard Institution's B.S 8004: 1986 code of practice for foundations and B.S.6031: 1981, code of practice for earthworks.

In conclusion, if the steps mentioned above are taken in to consideration, construction, and repair of damages caused by swelling soils can be prevented and the differential movement of the buildings will be reduced to minimum.

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