Characterization of Expansive Soils in Southwest Brazilian Amazon – a case study in a Flexible Pavement

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ABSTRACT: The main geotechnical aspects of the southwestern Brazilian Amazon are the absence of rocks for the gravel used in pavements and the widespread occurrence of expansive soils along the natural subgrade, associated with high rainfall indices. Although most of the local soils are rich in 2:1 clay minerals and amorphous material originated from Andes volcanic ashes – which is quite different from other Amazonian soils - the state-of-art about the behavior of these materials is still very limited. The high volumetric variation of these expansive soils has been causing financial losses and making transportation by land difficult in the region. This paper presents the case study of a pavement built over expansive subsoil at Rio Branco city. Physical, hydraulic and mineralogical characterizations of samples are presented. The results showed the high swelling potential of the samples and the importance of developing a methodology for the control of expansive soils for building over these materials.

Keywords: Expansive soils; Amazonia; Flexible pavement.

1. Introduction

Expansive soils are those clayey unsaturated soils which have the capacity to undergo volumetric changes when subject to change in suction or water content, commonly associated with seasonal moisture fluctuations. As a result, these soils shrink and swell in response to alternate dry and wet conditions, inducing vertical movement of soils which is detrimental for lightweight structures, such as buildings or highway pavements.

The mechanism of swelling in expansive clays is complex and depends on a wide range of variables such as soils characteristics, initial moisture content, climate, vegetation, in situ density, slope of the site and changes brought about by man's action [1]. Nelson and Miller [2] divide these factors in three different groups: the soil characteristics, the environmental factors and the state of stress.

Considering the mineralogical properties that cause soil volume changes, clay minerals which typically associated are smectite or vermiculite [3]. Illites and Kaolinites are infrequently expansive but can cause volume changes when particle sizes are less than a few tenths of a micron [2].

Expansive soils causes are found throughout the world, causing huge loss every year. The problem is global, but in general regions with more arid climates have more severe expansive soils problems [4]. In Brazil, the problem has been founded in many places, especially in the Northeast region [5].

The city of Rio Branco, located at Southwestern Brazilian Amazon, has been facing problems associated with expansive soils in its roads year by year. Nevertheless, there are few studies focused on understanding the local soils in pavement subject.

The objective of this paper is to present results of a typical Acre expansive soil as well as to use a simple approach for predicting vertical movements using unsaturated soils approach.

2. Background

2.1. Literature review

Unsaturated soil theories have shown significant development over the past four decades. A significant part of these theories was dedicated to study expansive soils, a problematic soil which impact structures and challenge geotechnical engineers around the world.

Due the need of supporting lightly loaded structures, it is important to provide tools for practitioners to reliably estimate the volume change behavior of expansive soils in field [6].

Essentially, when studying a potential heave problem, the engineer must evaluate the present state of stress in the soil profile and determine suitable physical properties for predicting future behavior [4].

The focus of the majority of prediction methods has been towards estimating the maximum heave potential which occurs when soil attains the saturation condition, usually through linear relationship between void ratio (vertical strain) and net normal stress or soil suction. Likewise, 1-D heave are the most common analysis methods. [6, 7].

Nevertheless, a limitation of these methods is in not providing information of soil movement in the field over time. This led researchers to develop several methods considering the soil suction fluctuation as a function of time within the active zone depth; or connecting the soil state variables to volume change movement of the soil. Adem et al. [6] classify this group in three categories: (i) consolidation theory-based methods that use the matric suction and the net stress as state variables; (ii) water content-based methods that use the soil water content as a state variable; and (iii) suction-based methods that use the matric suction as a state variable.

Regarding the classification based in 1-D heave prediction methods, Vanapalli et al. [8] list three categories of techniques or procedures: (i) Empirical methods that use soil classification parameters; (ii) Oedometer test methods that take into account the loading and unloading sequence, surcharge pressure, sample disturbance and apparatus compressibility for reliable determination of the swelling pressure; and (iii) soil suction methods where the influence of suction is taken into account through the use of different parameters.

It's important to note the time consuming and high cost of carrying out a direct measurement of soil suction in both laboratory or field is often excessive. It has encouraged the pursuit of new means of implementing unsaturated soil mechanics into routine geotechnical engineering practice. Fredlund et al. [9], for instance, described a procedure for predicting the soil water characteristic curve from the grain-size distribution. Ito et al. [10] modeled a two-dimensional soil-atmosphere from climate data such as wind speed, net radiation, precipitation, and park watering, which were found to be the major contributors to the changes in matric suction.

2.2. Constitutive relationships for volume change behavior

The volume change constitutive relationship of an unsaturated soil links the deformation state variables to the stress state variables. The deformations state variables are the changes in total volume (i.e. soil structure) and the changes in water volume [6]. The two independet stress state variable are the net normal stress ($\sigma - u_a$) and the matric suction ($u_a - u_w$) (where, u_a = the pore-air pressure, u_w = the pore-water pressure, and the σ = the total stress).

These stress state variables provides a smooth transition when going from an unsaturated condition to a saturated condition. When the dregree of saturation approaches 100 percent, $u_a = u_w$. The term ($u_a - u_w$) is zero and ($\sigma - u_a$) become equal to ($\sigma - u_w$), the effective stress term well known in saturated soil mechanics.

2.3. Fredlund (1983) Method

Fredlund et al. [11] proposed a method for predicting 1-D heave in expansive soils involving the use of Eq. (1) and the constant volume swell (CVS) oedometer test.

$$\Delta H = C_S \frac{H}{1+e_0} log \left\{ \frac{P_f}{P'_S} \right\} \quad (1)$$

Where H_i = thickness of the i_{th} layer, $P_f = (\sigma_y \pm \Delta\sigma_y - u_{wf})$ = final stress state, P'_s = corrected swelling pressure, C_s = swelling index, σ_y = total overburden pressure, $\Delta\sigma_y$ = change in total stress, u_{wf} = final pore-water pressure, and e_0 = initial void ratio.

The CVS test involves innundating a sample subjected to a token load, which is tipically 7 kPa. As the sample attempts to swell, the applied load is increased to maintain the sample at a constant volume until there is no futher tendency for swelling. This point is referred to as "uncorrected swelling pressure" (P_s). The sample is then further loaded and unloaded in the conventional way of a consolidation test, as shown in Fig. 1 [4].

The CVS test provides two main measurements; namely, corrected swelling pressure, P'_s and the swelling index. P'_s is obtained from an empirical construction proposed by Casagrande which takes in account the effect of sampling disturbance. C_s is an index parameter equal the slope of the rebound curve determined from the CVS test.

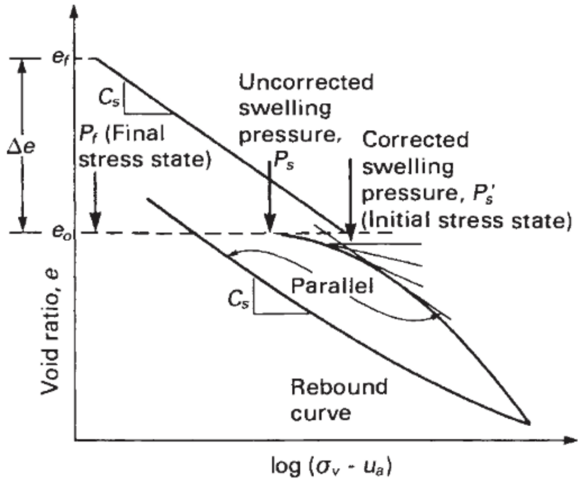

Figure 1. One-dimensional oedometer test results showing effect of sampling disturbance [12].

2.4. Mechanisms of longitudinal crack development in pavements

Zornberg and Gupta [13] describes the following mechanism that leads to the development of longitudinal cracks (Fig. 2) after long term observation in Texas Departament of Transportation (TxDOT):

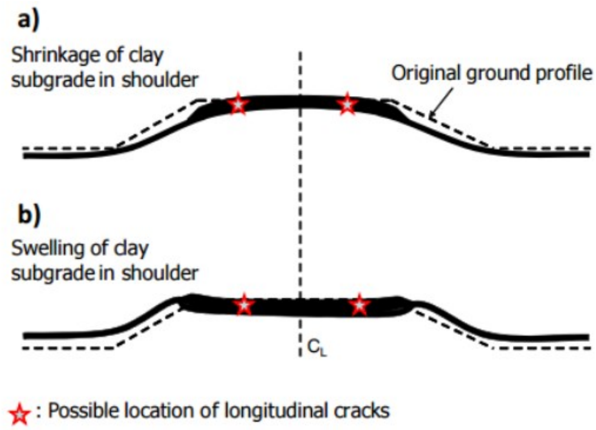

Figure 2. Mechanism of Pavement deflection over expansive clay subgrades [13].

According with Fig. 2a, during the dry season, there is a decrease in the moisture content of the soil in the vicinity of the pavement shoulders. This leads to settlements in the shoulder area, but not in the vicinity of the central line of the pavement, where moisture tend to remain constant. On the other hand, during the wet season, the moisture content in the shoulders increases (Fig. 2b). As a consequence of these cyclic movements, the cracks are developed in the region where the moisture front advancing and retreating from the shoulders reaches its maximum penetration under the pavement.

3. Rio Branco city case study

3.1. General information

The soils of Rio Branco, a Brazilian city in South America (Fig. 3), are mainly composed of sedimentary clay with high plasticity and consequentely a high expansive potential, which is very commom for most part of surrounding region.

Figure 3. Location of the study area.

According with studies prepared by the Brazilian Company of Mineral Resources (CPRM) in Rio Branco city, mineralogical characterization showed that montimorilonite is the predominant clay mineral in almost all samples analysed [14].

An important point about the magnitude of volume change in soils is its high dependency on environmental conditions. Rio Branco has one period of dry and warm climate, between July and August; and other with very wet with high rainfall indices, between October and April. In addiction, the region has a large number of freshwater rivers and streams, which are usually seasonally flooded.

Some of traditional solutions for pavements built over expansive soils envolves the use of crushed stone aggregates both as moisture barriers and as a backfill material after removing some layers of expansive soils. However, as a rule in many cities of Brazilian Amazon, the cost of these materials are very expensive due the long transport distances from suitable rock deposits. The cost of gravel (m^3) at Acre can be as high as 4 times, for exemple, the cost of gravel at Rio de Janeiro.

3.2. Local site investigation

The site studied in this paper is a booming real estate area located at the vicinity of Rio Branco city. Firstly, it was verified the occurrence of longitudinal cracks in many asphalt-paved roads after ending of dry season, but still without opening traffic (Fig. 4).

Figure 4. Typical Longitudinal crack in the site study.

It was also oberved in the ground surface of the pavement shoulders the wide spreading of cracking in roughly polygonal patterns (Fig. 5). The cracks have approximatelly 1 inch of width and excavations have shown that cracks were up to 2 feet deep 2 feets of deep. It is caused by desiccation that occurs in dry season and also is influenced to the presence of root fibers that penetrate the soil profile.

Figure 5. Occurrence of cracking in roughly polygonal patterns at the studied site.

Attempts to evaluate the groundwater level showed that cracking affected the infiltration of moisture insofar as accelerate the infiltration of surface precipitation. In other words, it affects the fluctuation of water in subgrade, what influence the mechanism described in

2.4. Plus, it can affect directly the pavement structure if vertical moisture barriers are not used.

It is essential the early identification of expansive soils during the preliminary stages of a project. There are many criteria available to identify and characterize these soils, but the commom identification schemes are based on standard classification results, such as grain size analysis and Atterberg limits.

The Fig. 6 show the site where disturbed and undisturbed samples were collected, in approximately 1 meter of depth from the surface in an expansive clay layer about 2m. After excavating, as the exposed profile loses wet, huge blocks were easily displacement only by hands, indicating the effect of existing cracking and possible problems associated with soil erosion in case of slopes areas, which is also a commom problem in amazon region.

Figure 6. Collecting of disturbed and undisturbed soil samples at the local site.

3.3. Heave prediction studies

In order to predict the future ground movements, an unidimensional oedometer test was carry out at the Soil Laboratory at Federal University of Acre (UFAC). The test was carried out on a sample of 10 cm in diameter and 3 cm in height, as shown Fig. 7.

The test followed the Fredlund et al. [11] procedures, presented in section 2.3. The main objectives were to obtain the corrected swelling pressure, P'_s; and the swelling index, C_s.

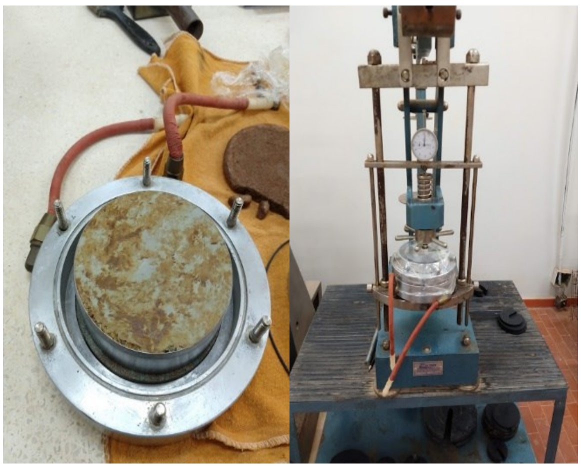

Figure 7. Determination of swelling properties by Constant volume oedometer test procedure.

Since most of heave will occur near ground surface where there is the biggest difference between the corrected swelling pressure and the total overburden pressure, we can assume that heave tends to become zero in a depht where there is no difference between them. The depth where it occur can be defined as the active depht of swelling, which is predict by Eq. (2) according with [12].

$$H = \frac{P'_S}{\rho g} \quad (2)$$

Where H = active depht of swelling, ρ = total density of the soil (assumed to remain constant with depth), and g = gravitational acceleration.

According with 1-D predicting heave method by [11], it is important to know the initial state of stress and predicting the future ground movements in order to carry out a volume change analysis. Once the required laboratory information about both swelling soil properties and in situ state of stress were obtained, it is necessary to estimate the final state of stress, which is related to the designed pavement and also the changes in pore-water pressure conditions.

Considering the wide spreading of crackings and the high rainfall indices at the studied area, it was assumed that the final pore-water pressure go to zero, creating a hydrostatic condition. It is a realistic assumption, once the ground water table rises to ground surface often at the local site.

For the case study, the soil swelling properties were considered along the entire predicted active depht. The expansive clay material was subdivided into three increasing layer with, respectly, 20%, 30% and 50% of tickness. The amount of heave in each layer is computed considering the stress state changes at the midle of the layer. The heave in each sublayer is calculated with Eq. 1, and the total heave from the entire expansive soil layer, ΔH, is equal to the sum of the heave amounts calculated for each of the subdivided layers (Eq. 3):

$$\Delta H = \sum \Delta h_i \quad (3)$$

Considering the ground surface covered with asphalt (impermeable), the initial stress state P_0 will be equal to the corrected swelling pressure and the final stress state P_f will be the overbuden pressure. Similar assumptions can be found in [12].

4. Results and discussions

4.1. Subgrade Soil Properties

The initial subgrade soil properties obtained from disturbed samples are summarized in table 01:

Table 1. Subgrade Soil Properties from disturbed samples

Depth (cm)	Liquid Limit (%)	Plasticity index (%)	Passing #200 (%)	Fine clay (% < 2μ)
100	65	29	99	56

4.2. Soil expansivity classification

Based in the sample values presented in Table 1, the Fig. 8 show the soil expansivity prediction according with Van Der Merwe chart [15]. This classification relate Plastic Index and Clay fraction, what results in a high expansivity for the analysed soil.

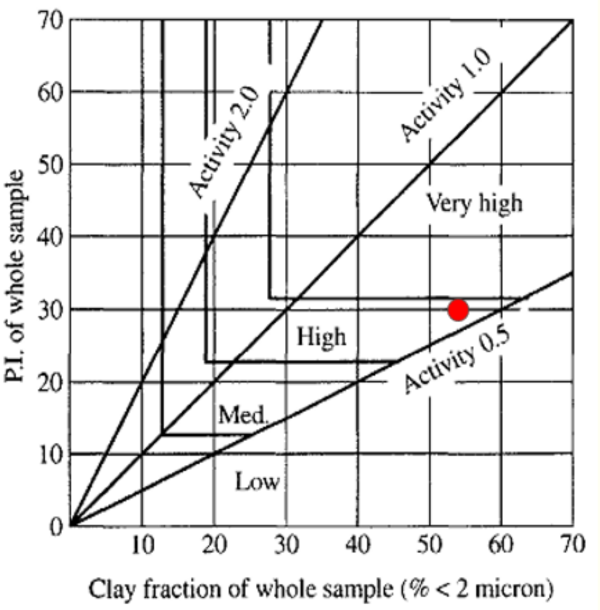

Figure 8. Soil expansivity classification based on Van Der Merwe chart [15].

4.3. Consolidation/swelling curve parameters

The result of the CVS oedometer test allowed to quantify the initial state of stress along the swelling clay profile (Fig. 9). It was obtained a corrected swelling pressure (P'_s) of 90 kPa, which is 4 times greater than the conventional value, P_s, highlighting the importance of accounting for the effect of sampling disturbance. In addiction, from the slope rebound curve was obtained a swelling index (C_s) of 0.02.

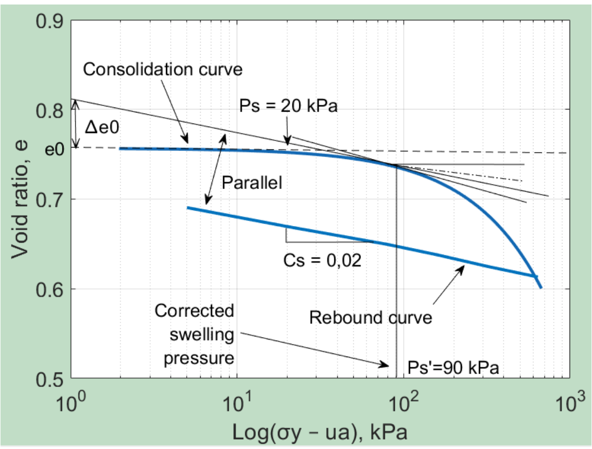

Figure 9. Swelling parameters from oedometer test for Rio Branco clay sample.

Table 2 shows the geotechnical characteristics of the sample; and Table 3 summarize the swelling parameters obtained from the consolidation/swelling curve.

Table 2. Subgrade Soil Properties from undisturbed samples.

Total unit weight (kN/m³)	Initial void ratio e_0	Natural water content (%)
19.7	0,77	25.6

Table 3. Subgrade Soil Properties from constant volume oedometer test.

Swelling index C_s	Uncorrected swelling pressure P_s (kPa)	Corrected swelling pressure P'_s (kPa)	Final pore water-pressure (kPa)	Active dephth of swelling H (m) – Eq. 2
0,02	20	90	0	0.5

The stress state situation is showed in Fig. 10. The Table 4 presents the calculation of the total heave assuming no change in total stress due to excavation or placement of fill.

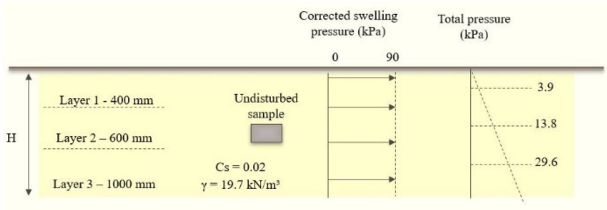

Figure 10. Stress state profile considering final pore-water pressure equal to zero and no change in total stress.

Table 4. Total heave calculation for Rio Branco clay sample assuming no change in total stress.

Layer nº	Initial overburden pressure σ_v (kPa)	$P_f = \sigma_v \pm \Delta_v - u_{wf}$ (kPa)	Δ_{hi} (mm)
1	3.9	3.9	6.1
2	13.8	13.8	5.5
3	29.6	29.6	5.5
Total heave ΔH (mm) =			**17.1**

The calculations in Table 4 show the amount of heave in each layer when its suction change to the predict or expected value. The calculations also show that a total heave of 17.1 mm is likely to occur. Moreover, aproximatelly 36% of the total heave occurs in the upper quarter of the clay strata.

As a solution, the need of removal 0.90 m of swelling clay to be replaced with an inert material was evaluated. The unit weigh of the inert material was assumed as being equal to 18.0 kN/m³, correspondenting to the future pavement. Both assumptions are according with local pavement practice.

It is also assumed that the independent process of excavation and replacement do not allow sufficient time for equilibrium to be established, wich means that the soil responds only to the net changes in stress. It is also a realistic assumption for local pavement practice over expansive soils, since after excavation is commom to apply a thin layer of an inert material, thus avoiding the losing of moisture in the subgrade.

The stress state situation after excavation is showed in Fig. 11. Table 5 presents the calculation of total heave assuming change in total stress due to excavation and placement of fill.

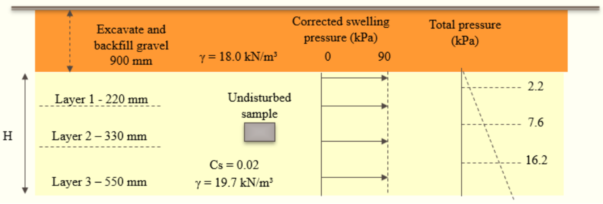

Figure 11. Stress state situation considering final pore-water pressure equal to zero and change of total stress due to excavation and placement of fill.

Table 5. Total heave calculation for Rio Branco clay sample assuming change of total stress due to excavation and placement of fill.

Layer nº	Initial overburden pressure σ_v (kPa)	$P_f = \sigma_v \pm \Delta_v - u_{wf}$ (kPa)	Δ_{hi} (mm)
1	19.9	18.4	1.7
2	25.3	23.8	2.2
3	34.0	32.5	2.8
Total heave ΔH (mm) =			**6.6**

Considering these calculations, the procedure of replacement with an inert material carried out at the site, would mean only 7mm of heave. This low value does not explain the development of longitudinal cracks show in Figure 4, which could be related to the mechanism proposed by Zornberg and Gupta [13], thus due to moisture fluctuations at shoulders areas, inducing differential movements.

5. Conclusions

The behavior of expansive soils are described by predictions methods based on maximum heave potential for saturated condition, which does not provide the understanding of field behavior with time.

Geotechnical tests were carried out on a soil of Amazonia basin, where expansive behavior of subsoil was suspected due to the crack pattern on the local pavement.

Total stress change due to fill replacement carried out at the site and heave calculation was conducted based on geotechnical test. The low value of the calculated heave is not enough to explain the cracks on the pavement. It seems that moisture fluctuations at the shoulders induced differential movements aggravated by the cracks on the ground surface of the shoulder area, as shown in Figure 5.

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