Comprehensive Methodology for the Evaluation of Clay Expansiveness. A Case Study

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Abstract. Although expansive soils have been widely studied by different authors, just a few have made methodological proposals to comprehensively evaluate this phenomenon. The aim of this paper is to suggest a methodology which will serve as guideline to evaluate soil expansiveness in a comprehensive way. This methodology comprises a series of tasks: identification and classification of potentially expansive soils, estimation of the active zone depth, quantification of the expansion, calculation of heave and choice of a solution.

Key words: clay expansiveness, expansive soil, geotechnical investigation.

1. Introduction

Problems related to soils expansiveness are common worldwide (Jiménez Salas *et al.*, 1981 and Das, 2000), and Cuba is no exception (Monzón, 1976 and Delgado & Quevedo, 2002). The results of soil expansiveness (uplifting, cracking and failure in roads, light buildings, canals and dams) cause extensive damage and economic losses.

In Cuba, the volume of research and publications on the topic is disproportionately small, if compared to the number of problems caused by the phenomena associated to this type of soils. The importance of phenomena typical of expansive clays motivated this research, whose main objective is to apply a methodology to evaluate clay expansiveness in a comprehensive way so as to identify and quantify the problem and find a solution to it.

This methodology comprises the following tasks:

I) Identification and classification of the problem in order to determine whether potentially expansive clays exist, and if they do what degree of attention needs to be paid to them.

II) Estimation of the active zone depth to analyze the dynamic suction profile and predictable moisture changes in the expansive soil.

III) Quantification of the expansion to assess numerical values of deformational properties when the soils are sufficiently prone to volume change.

IV) Heave prediction to determine the potential vertical movement.

V) Choice of a solution to compare and assess alternative design solutions.

It is important to point out that this methodology is based on the premise that volume changes are the result of the soil's effective stress, as a consequence of either internal or external causes. From this point of view, the expansive nature of soils, mainly determined by the content of expansive laminar structure clay mineral, is a necessary condition, but not sufficient, for the phenomenon to occur.

2. Assessment of the Results Obtained with the Application of the Methodology. Case Study: Town of Crecencio Valdés

The main objective of this investigation was to evaluate the potential soil expansiveness in an area of the town of Crecencio Valdés, Cuba, where a day care center and a school were to be built (Delgado & Quevedo, 2002). This investigation was carried out in two stages: one, of preliminary investigation and the second, of detailed investigation. During the first stage, tasks I and II listed above were accomplished, while tasks III and IV were accomplished during the second stage. Task V was implemented during the project stage.

The experimental area is located in the northern coastal plains of the central region of Cuba, only 8 kilometers away from the coast, and less than a kilometer away from the right bank of the Sagua la Chica River.

2.1. Geotechnical investigations. Preliminary investigation stage

General geological and geotechnical data, necessary for the designer to preliminarily assess the construction economic and technical convenience, is collected in this stage. This data is based on previously obtained data and on the topographical data. Also, minimum field investigation was carried out in order to determine the soil physical properties and classification, among other characteristics.

Knowledge of the geotechnical soil profile is vital, and it was found to be:

0.0-0.30 m: Topsoil. Dark clayey material with a high content of organic matter;

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Submitted on November 9, 2007; Final Acceptance on June 11, 2008; Discussion open until April 30, 2009.

0.25-1.0 m: Highly plastic clay with little sand and no gravel, dark brown colour, eluvial-deluvial origin. Classification using the U.S.C.S.: CH (Layer 1);

1.0-5.5 m: Plastic sandy clay, brown to yellowish brown, with fine gravel and carbonate. Classification using the U.S.C.S.: CH-MH (Layer 2) Camacho Formation;

> 5.5 m: Limestones and dolomites of the Remedios Group. Water table position: 8.25 m.

2.1.1. Task I. Identification and classification of the expansiveness problem

The first step, and the most important, is observation. Through observation it was possible to detect clays of medium to high plasticity, a relatively high water table and the occurrence of a dry and a wet season. The existence of these characteristics suggests the presence of expansive soils, but so far it is not possible to determine whether they are active.

Also, superficial cracking was detected at the end of the dry season. The cracks were 60 cm deep and with a 4-5 cm maximum width, which is a clear evidence of the soil expansiveness when rehydrated.

Finally, the visual survey of the town (where approximately 75% of the houses were moderately or slightly damaged) and the inhabitants' comments sufficed to conclude that expansiveness problems indeed existed.

The second step is to consider the physical tests results as indicators of expansiveness. The physical test showed the existence of two fine soils with clay contents that ranged from 40.5% to 22%, low to medium natural water content mainly near the surface, low to high plasticity, and classified as CH and CH-MH according to the U.S.C.S. All these characteristics were a clear indication of expansiveness. Data obtained in previous investigations showed that the clays were esmectites with cation exchange capacity higher than 53 m.e./100 g, which warrants potential expansion.

Once the potential expansiveness is confirmed, the soil is identified and classified using a specific method for this. In this case a method was designed, which takes into account the characteristics of the Cuban soils. With this method, the natural water content (w_n, %), the liquid limit (w_{LL}, %), the clay content (C, %), the consistency index (I_c) and the dry density (γ_d, kN/m^3) are used to obtain the swelling indices, the final water content (w_f, %), the free swelling index (s_f, %), and the controlled swelling index (s_c, kPa). Table 1 shows the regression equations obtained to predict the soil swelling indices.

Although these equations allow a quantitative assessment of the swelling indices, they are only used in the investigation preliminary stages to classify the relative magnitude of the volumetric change. The following classification is recommended for this purpose (Table 2).

According to this method, layers 1 and 2 have a medium degree of expansion (see Table 3).

Table 1 - Regression equations.

N.	Regresion equation	Unit
1	$w_f = 0.92\, w_n + 16.29$	%
2	$w_f = 0.51\, w_{LL} + 8.85$	%
3	$\log s_f = 0.54 \log C + 0.67\, Ic - 0.39$	%
4	$s_f = 25\, \gamma_d + 14.3\, Ic - 38$	%
5	$\log s_c = 1.21 \log s_f + 0.66$	kPa

Table 2 - Classification of the degree of expansion.

s_f (%)	s_c (kPa)	Degree of expansion
< 4	< 25	
4-10	25-80	Low
10-22	80-200	Medium
> 22	> 200	High

Table 3 - Soil classification according to degree of expansion.

Soil	w_f (%)	s_f (%)	s_c (kPa)	Degree of expansion
Layer 1	41.0	15-17	130	Medium
Layer 2	36.6	10-11	85	Medium

So, by means of the observation, the physical tests and this method it was possible to make an accurate diagnosis of the problem and to determine the level of attention needed.

2.1.2. Task II. Estimation of the active zone depth

The active zone depth was obtained from the relation between natural water content and plastic limit (w_{LP}) versus depth (Z), and consistency index (I_c) and liquidity index (I_L) versus depth. The sampling was carried out at the end of the dry season and results are shown in Table 4.

These results prove that layer 1 is comprised within the active zone and in layer 2 the active zone (Ha) approaches 2.5 m. The indices show a water content deficit ($w_n/w_{LP} < 1.0$; $I_L < 0$ and $I_c > 1$), that is, the soil water content is well below the final water content up to a 2.5 m depth, where the suction profile begins to stabilize.

2.2. Geotechnical investigation. Detailed investigation stage

The results obtained in this stage allow the execution of the construction; therefore, the final data is to be very accurate and precise.

A number of investigation methods were used, the sampling was also carried out at the end of the dry season, and a series of tests were conducted to characterize the soil mechanically and physically.

Table 4 - Relation between depth and w_{ithin} / w_{LP}, I_L and I_c.

Depth (m)	Indices			
	w_n (%)	w_{ithin} / w_{LP}	I_L $\frac{w_n \quad w_{LP}}{IP}$	I_c $\frac{w_{LL} \quad w_n}{IP}$
Layer 1				
0.2-0.3	24.28	0.74	-0.25	1.25
0.4-0.6	25.63	0.78	-0.21	1.21
0.6-0.8	25.21	0.77	-0.22	1.22
0.8-1.0	24.97	0.76	-0.23	1.23
1.0-1.2	24.22	0.74	-0.26	1.26
Layer 2				
1.2-1.4	25.28	0.92	-0.08	1.08
1.6-1.8	25.47	0.93	-0.07	1.07
2.0-2.2	27.22	0.99	-0.01	1.01
2.4-2.6	27.61	1.01	0.01	1.01
3.0-3.2	29.59	1.08	0.08	0.92
3.4-3.5	30.63	1.12	0.12	0.88
3.6-3.7	30.99	1.13	0.13	0.87
3.8-3.9	33.19	1.21	0.21	0.79
4.0-4.1	34.59	1.26	0.26	0.74
4.2-4.3	33.43	1.22	0.22	0.78

2.2.1. Task III. Quantification of the expansion

Four tests were carried out in order to determine the swelling indices. These tests were the simple oedometer test modified by Ralph & Magor (1972) and the constant volume swell test (Sullivan & McClelland, 1969). Also, the data obtained from these tests was analyzed as if it was a two-embedded-samples oedometer test (Holtz, 1970).

The data obtained from the modified simple oedometer test are shown in Table 5, whereas the data obtained from the constant volume swell test appears in Table 6. The free swelling values shown in the tables correspond to the overburden pressure, while the probable swelling (s_{prob}) values were obtained under a 35 kPa pressure.

The two-embedded-samples oedometer test was not conducted, however, the data obtained from the modified simple oedometer test and the constant volume swell test were analyzed together as if it was the data obtained from the test proposed by Holtz (1970). The only condition for doing this was to use the same sample in both tests. The values in Table 7 are the ones obtained for the *C* curve[1], which is an approximation to volumetric change values for loaded embedded samples under an intermediate pressure, as required by the two-embedded-samples oedometer test.

Table 5 - Results of modified simple oedometer tests.

Layer	Indices			
	s_f (%)	s_{prob} (%)	w_f (%)	s_c (kPa)
1	14.0	4.8	39.21	142
2	11.9	3.1	36.89	84

Table 6 - Results of constant volume swell tests.

Layer	Indices			
	s_f (%)	s_{prob} (%)	w_f (%)	s_c (kPa)
1	12.4	3.5	37.69	150
2	10.2	2.1	34.90	90

Table 7 - Results of two-embedded-samples oedometer test.

Layer	Indices			
	s_f (%)	s_{prob} (%)	w_f (%)	s_c (kPa)
1	14.0	4.1	39.09	150
2	11.9	2.6	36.03	90

The free swelling values (12-14%) and the controlled swelling values (90-150%) correspond to those predicted in Table 3.

2.2.2. Task IV. Heave prediction

Once the soil expansiveness is quantified, it is necessary to calculate the active zone heave.

The proposed prediction method used the data obtained from the oedometric tests, specially the results of the two-embedded-samples oedometer test. The active zone was discretized using the sublayer method. The calculation was made taking into account a 0.8 m foundation depth, a 4 x 0.4 m superficial foundation beam, a 35 kPa pressure and a 1.7 m active zone depth. The results are shown in Table 8. The values of s_{prob} and Δe (change in void ratio) shown in the table correspond to the pressure acting on the foundation level.

The use of the two-embedded-samples oedometer test results and the method for the prediction of the active zone heave produced a more realistic evaluation of the phenomenon (S_{cal} = 2.19 cm). It is important to point out that it is possible to use the values obtained in the constant volume swell test when it is not possible to interpret the data as if it were a two two-embedded-samples oedometer test.

2.2.3. Task V. Choice of a solution

At this moment the designer is ready to analyze and compare different design alternatives and choose an adequate foundation solution.

1 The *C* curve is the fitting curve related to swelling pressure values obtained from the constant volume swell test and the free swelling values obtained from the modified simple oedometer test.

Table 8 - Active zone heave.

Test	Parameters and calculation of heave		
	s_{prob} (%)	Δe	Calculation method $h \dfrac{e}{1 \quad e_0}$ (cm)
Modified simple oedometer	4.8 3.1	0.091 0.064	2.61
Constant volume oedometer	3.5 2.1	0.066 0.044	1.79
Two-embedded-samples oedometer test	4.1 2.6	0.078 0.055	2.19

The analysis of design alternatives should begin with a comparison of the calculated heave (S_{cal}) and the angular distortion (tg p) with the limit values. Allowable limit (S_{allow}) values were obtained by Quevedo *et al.* (2001), and were reduced as suggested by the SNIP (1986). In order to adjust heave values to permissible values, the foundation depth was increased. Also, because of the soil profile homogeneity, similar acting loads, short spans, and safety measures to prevent the active zone wetting, together with the estimated foundation depth, angular distortion values were kept low, as required by the SNIP. Table 9 shows the results.

It is confirmed that the construction values adjust to allowable deformations, but lighter elements directly placed on the soil may be affected by heaving. Therefore, the solution was to act on the soil to avoid angular distortion and, consequently, damage to these elements.

The most important change made to the original project was to use a superficial foundation collar beam. Its depth was increased in 0.2 m (reaching 0.8 m), its width was reduced from 0,6 m to 0.45 m and compacted coarse aggregate with a 0.5 m width was used. Also, the excavation was carried out very fast and protecting the soil from desiccation and a 0.2 m sand and gravel layer was placed in all the construction area. Reinforced concrete beams were placed on block walls to stiffen them and the walls were reinforced in maximum stress points. Other measures were placing an asphaltic concrete perimeter, having the same width of the active zone (2.5 m), substituting the patio with a garden with a patio with lining, and using additional drainage solutions like placing the construction at a different height, using buried flexible connexions and piping (at 0.8 m) to keep them away from areas prone to volumetric changes.

Additional measures were also taken, which implied keeping the construction away from trees, forbidding the planting of trees at a distance shorter than 1.5 times the height of an adult tree, eliminating gardens to avoid water content variations, and keeping drainage systems at least 20 m away from the building.

Table 9 - Comparison of allowable deformations.

Calculated values		Limit values	
S_{cal} (cm)	tg p	S_{allow} (cm)	tg p
2.19	< 0.0001	10(0.25) = 2.50	0.002(0.5) = 0.001

3. Conclusions

• Soil expansiveness evaluation implies the implementation of tasks, which should be carried out in different stages, but should not be overlooked. The methodology applied in this case study proved to be effective.

• A comprehensive analysis of different physical properties of the soil is a reasonable indication of expansion, so the proposed identification and classification method provides realistic results.

• The modified simple oedometer test, the constant volume swell test, as well as the two-embedded-samples oedometer test can help to solve the problem of evaluating soil expansiveness.

• The active zone heave prediction using the results of oedometer tests and the discretization of the active layer using the sublayer method provide realistic evaluation of the phenomenon.

• The results obtained in this investigation showed that the proposed methodology can be used for the study of Cuban expansive soils, and it is flexible and precise enough to be generalized to other regions of the country.

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List of Symbols

w_n: natural water content (%)
w_{LL}: liquid limit (%)
w_{LP}: plastic limit (%)
IP: plastic index
I_L: liquidity index
I_c: consistency index
C: clay content (%)
γ_d: dry density (kN/m^3)
w_f: final water content (%)
s_f: free swelling index (%)
s_c: controlled swelling index (kPa)
Z: depth (m)
s_{prob}: probable swelling (%)
Δe: change in void ratio
Ha: active zone (cm)
h: heave (cm)
S_{cal}: calculated heave (cm)
tg p: angular distortion
S_{allow}: allowable limit (cm)