Revisiting Classical Methods to Identify Collapsible Soils

O.M. Vilar, R.A. Rodrigues

Abstract. The paper revisits and updates two expedite methods used to identify the occurrence of collapsible soils. The methods were proposed by Gibbs (1961) and de Mello (1973) and have in common the assumption that collapsible soils are usually low density soils characterized through the relative compaction at natural condition, ratio between *in situ* dry-density and maximum dry density from Standard Proctor compaction test. The method by Gibbs (1961) considers in addition the soils moisture deviation, the difference between *in situ* moisture content and optimum moisture content. Data from different origin soils of all over the world were used and analyzed and equations were proposed to separate collapsible and non-collapsible soils. The updated procedures, as the original ones, were intended for use during preliminary investigation to identify potentially collapsible soil and to serve as a basis for planning more precise methods of investigation.

Keywords: collapsible soils; identification; unsaturated soil.

1. Introduction

Some soils under constant stress show volume decrease related to an increase of moisture content. This wetting-induced or soil collapse deformation characterizes the so called collapsible soils. Collapse deformation is a typical behavior of non-saturated low dry density soils and has been reported as occurring in many places all over the world in soils of different genesis such as aeolian, alluvial, colluvium and residual soils and even in poorly compacted embankments (Dudley 1970, Clemence & Finbar 1981, Vilar *et al.*, 1981, Vilar and Gaioto 1994).

Soil collapse can take place upon wetting for a wide range of applied stress. As the applied stress is increased, the amount of collapse deformation experienced by an unsaturated soil reaches a maximum and then decreases to a negligible value. The maximum value attained depends on the soil type, density and moisture content. Larger collapse deformations are associated to low densities and low degrees of saturation. (Alonso *et al.*, 1990).

The classical oedometer test with some variation has been extensively used to characterize and to quantify the soil collapse. In one option, the unsaturated specimen is loaded until a load of interest and, after equilibrium of load deformation, is wetted, thus allowing measuring the wetting induced deformation. The other option is the double oedometer test (Jennings & Knight 1957), performed with two similar samples, one unsaturated and the other soaked since the beginning of load application.

Figure 1 illustrates typical results of soil collapse using both options of test. The soil tested is from the Northwest Region of Sao Paulo State, Brazil and has experienced collapse strains after the filling of a reservoir (Vilar & Rodrigues, 2011). This region is covered by sandy soils that are of colluvial nature and reach depths of about 10 m. Their characteristics are: specific gravity, G_s = 2.63, dry density, ρ_d = 1.44 g/cm^3, moisture content, w = 7.2%, void ratio, e = 0.85, porosity, n = 46%, liquid limit, w_L = 18%, plastic limit, w_P = 11%, clay = 15%, silt = 6% and sand = 79% and they typically classifies as SC in the Unified Soil Classification System. Standard Proctor maximum dry density, ρ_{dmax}, is 2.04 g/cm^3, which is associated to optimum moisture content, w_{opt}, of 8.6%. From Fig. 1(a) one can observe the influence of stress on the magnitude of soil collapse and in Fig. 1(b) the differences in unsaturated and soaked sample, attributed to soil collapse.

Field tests also have been performed with procedures similar to the ones used in laboratory tests. The tests used were plate and pile load tests (Cintra 1998), cone penetration tests (Ferreira *et al.*, 1989), and load tests especially designed to measure the phenomenon (Ferreira 1993, Houston *et al.*, 1988).

Suction has long been recognized as a fundamental variable in the understanding of the mechanical behavior of unsaturated soils (Escario & Saez 1973, Fredlund & Morgentern 1977, Alonso *et al.*, 1990). Thus modern options of oedometer tests with controlled suction are currently available, allowing a detailed description of the collapse phenomenon. For instance, Fig. 2 shows compression curves from oedometer tests with controlled suction using the axis translating technique obtained in the same collapsible soil presented in Fig. 1. The tests were performed on samples with initial suction, s, of 200 kPa. After equilibrium of suction, each specimen was loaded under net normal stress of 50, 100, 200 and 400 kPa allowing for the equilibrium of deformation under load. Then the suction was gradually reduced to 0 kPa and the corresponding col-

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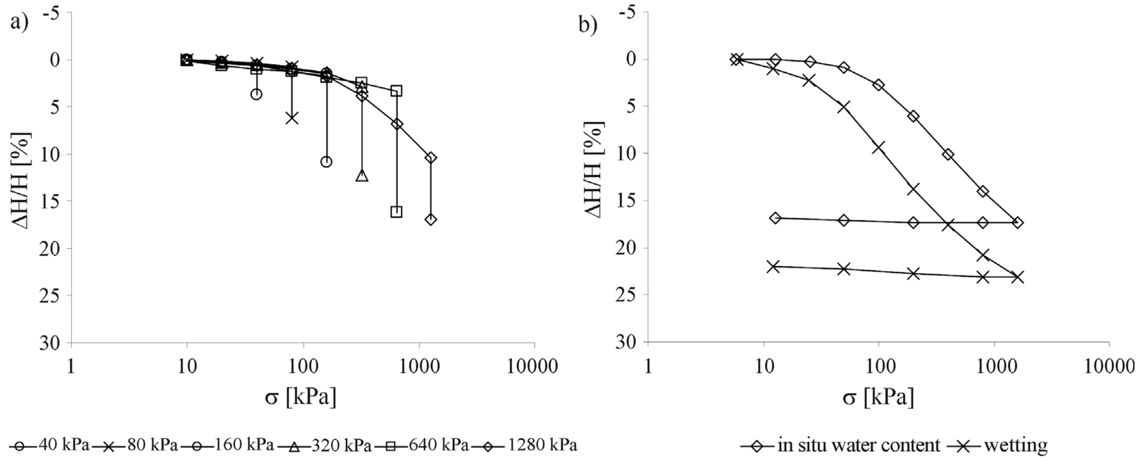

Figure 1 - Conventional oedometer test results from Pereira Barreto-SP. Brazil: (a) single oedometer tests; (b) double oedometer tests.

lapse strains were measured. In Fig. 2 (a) the variation of collapse deformation with net vertical stress, the difference between total stress and pore air pressure, σ - u_a, can be appreciated, indicating the influence of stress on collapse strain. These are small for lower stress, tend to increase with load and then decrease and negligible values of strain can be attained if larger loads are applied.

Figure 2 (b) shows collapse strain ($\Delta H_c/H$) that arises as suction is reduced, where ΔH_c is the height variation of the sample after a stage of suction reduction and H is the specimen height. As can be seen in Fig. 2 (b), in the initial stages of suction reduction, the collapse potential is negligible and it tends to increase for the lower values of suction, reaching its maximum as suction approaches zero. Wetting-drying cycles such as those that seasonally takes place in natural soil did not introduce additional deformation, as expected.

Figure 2(c) shows the influence of suction on confined compression curves. Each pair of curves of saturated specimen (s = 0 kPa) and at a known suction can be considered as a double oedometer test and allows observing the influence of suction (or moisture content) on collapse strain, calculated from the difference in void ratio at a known load.

The improvement of collapsible soil testing, both in laboratory and in the field, has been accompanied by the development of analytical models, such as the Barcelona Basic Model (Alonso *et al.*, 1990), which allows to reproduce the unsaturated soil behavior, including soil collapse, in a more comprehensive way. However, it is necessary to recognize that the use of more elaborated testing techniques and sophisticated constitutive models demand expertise, are time consuming and expensive. In many applications, the geotechnical engineer needs simple and straightforward methods to deal with some particular problem, especially in the preliminary parts of the project. This is the case of identification or detection of the occurrence of collapsible soils in a particular site or large areas, as those traversed by linear works such as canals and roads. In this context, this paper revisits and updates former criteria proposed by Gibbs (Gibbs, 1961; USBR, 1998) and by de Mello (1973), analyzing the density characteristics of soils and data from soil collapse occurrences registered in many parts of the world.

2. Some Features of Methods to Identify Collapsible Soils

The existing methods for identifying collapsible soils generally rest on some basic principles: a) regional methods developed from empirical concepts and expedite tests, such as the methods based on consistency limits and physical indexes (Denisov 1951, Feda 1966, Sultan 1971, Gibbs & Bara 1967); b) methods based on oedometer tests with wetting (Denisov 1951, Jennings & Knight 1957, Reginatto & Ferrero 1973) and c) methods based on field test, as the cone penetration test (Ferreira *et al.*, 1989) or specially designed plate load tests (Houston *et al.*, 1988, Ferreira 1993).

Many of the empirical criteria incorporate the idea of a low density soil, expressed in different direct and indirect ways. Gibbs (1961) has used a criterion based on the relationship between in-place dry density and laboratory maximum dry density and the moisture content deficiency, as shown on Fig. 3(a). If the relative compaction and moisture content difference plot below and to the right of the limit line, significant wetting induced deformation must be expected even under low pressures, and treatment of in-place materials may be required. This criterion was developed on the basis of the Bureau of Reclamation experience, considering soils classified as ML, CL, ML-CL, SM and MH and the loads within the range applicable for small dams (USBR 1998). This option comes to the heart of the problem since it encompasses soil looseness and moisture deficiency, however it has not, apparently, gained acceptance of geotechnical community and it is hardly referenced in the literature issued after 1970.

In the same report Gibbs (1961) has proposed an alternative method for fine grained soils that was based on in

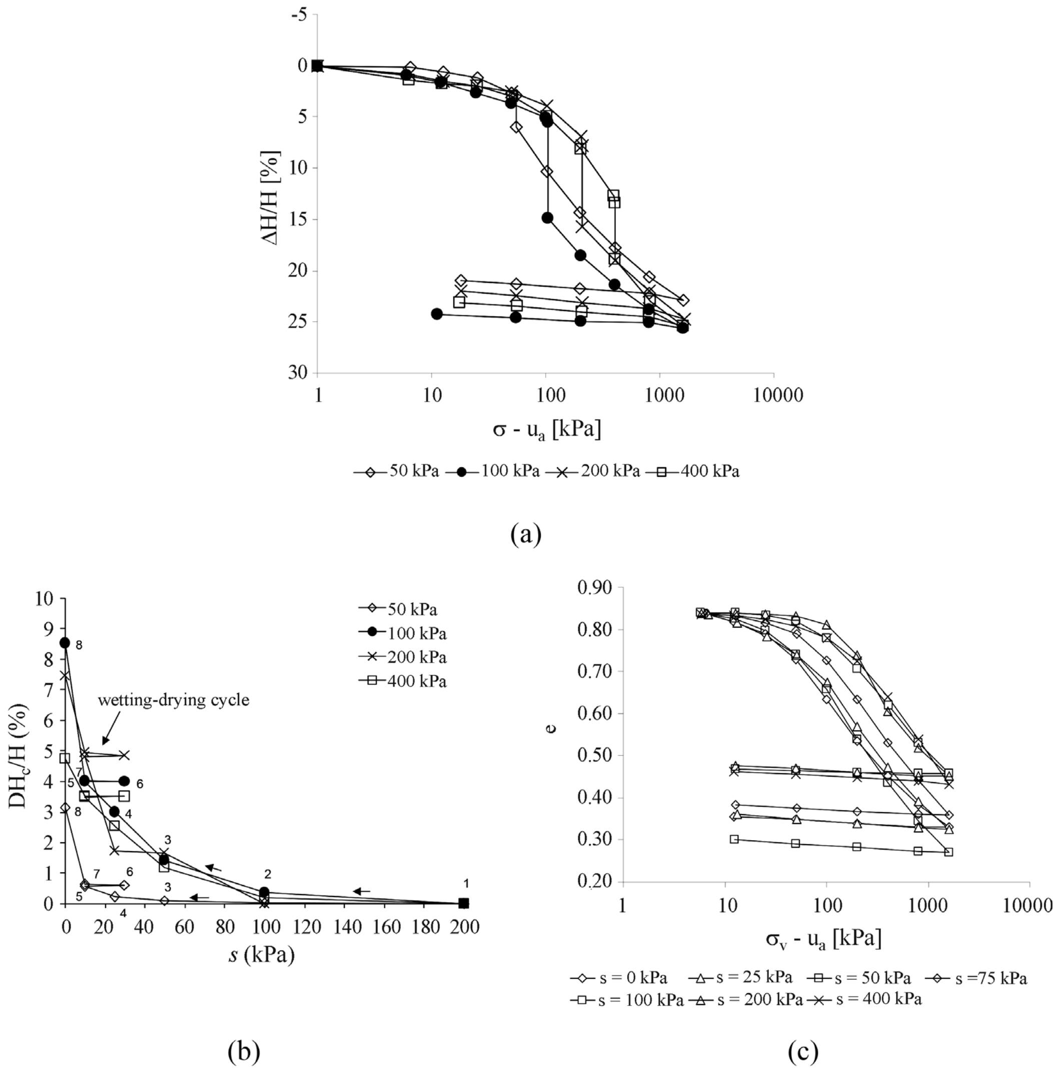

Figure 2 - Suction controlled oedometer test results from Pereira Barreto-SP. Brazil: (a) tests with initial suction of 200 kPa; (b) collapse potential during suction decrease and wetting-drying cycle; (c) compression curves for different suction (Rodrigues & Vilar 2006; Vilar & Rodrigues 2011).

situ dry unit weight and liquid limit, as sketched in Fig. 3(b) taken from Gibbs & Bara (1967). This method, from now on called the Gibbs and Bara method, rests on the soil void ratio expressed through in-situ dry density and liquid limit. Case I shows a soil whose volume of voids is larger than that required to hold the volume of water needed to reach the liquid limit. Saturation will result in moisture content in excess of the liquid limit, consistency will be low and the potential for collapse would be high. In this case, if collapse did not occur, the soil would be in a very sensitive condition (Knodel 1981). If voids volume is less than that required to hold the moisture content at the liquid limit, as shown by Case III, the soil at saturation will remain in the plastic state and have greater resistance against particle shifting and only settle as a normal result of loading. The extensive and successful use of this method is shown in Gibbs & Bara (1967) and Knodel (1981). Similar concepts are usually embodied in other proposal, usually incorporating some local correction factors trying to better match field observation and criteria result (Denisov 1951, Feda 1966).

De Mello (1973) has considered for colluvium soils similar to those illustrated in Fig. 1, that the condition for collapse deformation to occur is that the relationship between in-place dry density and laboratory maximum dry density should be lower than 80%. In fact, Gibbs & Bara (1967) method applied to the soil of Fig. 1 lead to a collapsible soil as the void ratio capable of retaining the moisture content associated to the liquid limit is 0.48, which is lower than natural void ratio (0.85). In addition, the relationship between in-place dry density and laboratory maximum dry density (Standard Proctor) is 76%, which is lower than the value proposed by de Mello (1973), thus also, classifying this as a collapsible soil.

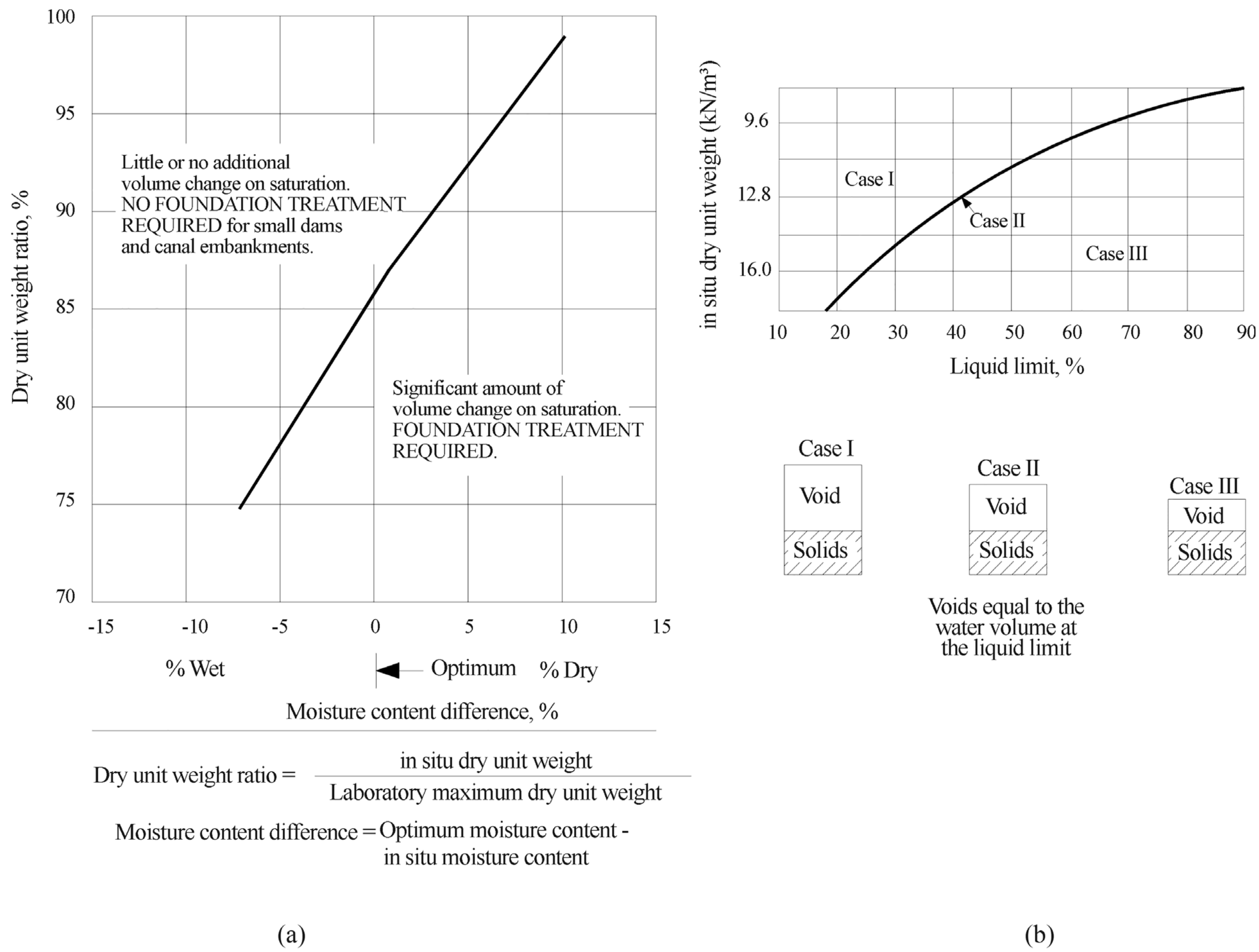

Figure 3 - Criteria for identifying collapsible soils. (a) based on relative compaction and moisture content difference; (b) based on dry unit weight and liquid limit (USBR, 1998).

3. Updating an Expedite Methods to Identify Potentially Collapsible Soils

In spite of the successful application of the Gibbs and Bara method, it is worth to recognize that it is not able to identify non-plastic soils and, in some instances, fail to identify some collapsible soils as can be confirmed checking some of the data of Table 1 presented in the Appendix, for instance soils 12, 14, 36, 63, and 67. Thus, based on the general concepts and physical characteristics of collapsible soils reported in the literature, it seems consistent to pursue a criterion to identify collapsible soils, considering in a direct way the usual looseness of collapsible soil, through compaction parameters. In this sense, the authors recover the proposals by Gibbs (1961) and de Mello (1973) that are updated with some data of collapsible soils from various parts of the world. Those proposals are reanalyzed and the possible range of values at which a soil should be considered as collapsible is enlarged. The proposed methods take into account the relative compaction of natural soil, relating the in situ dry unit weight (ρ_d) with maximum dry density (ρ_{dmax}) of soil as given by a Standard Proctor Test (ASTM D 698). Using these parameters it is possible to determine the natural relative compaction (RC_n) of the soil.

$$RC_n = \frac{\rho_d}{\rho_{d\max}} \times 100(\%) \quad (1)$$

To define the critical condition for collapse, it was assumed that soil density was of highest importance because, for this type of deformation to occur, soils must be sufficiently loose so that they are capable of collapsing when their particle-to-particle bond is weakened by wetting or suction decrease.

However, the collapse also depends on the moisture content of the soil or on the degree of saturation. Although the soils must first be in a critically low-density condition for a collapse of structure to occur, the existing moisture content or the associated suction must be also considered an important part in the analysis. Soils which were already at an *in situ* high degree of saturation were expected to be minimally affected by additional wetting or suction decrease, as they will compress under load leaving small room for collapse to takes place. Soils drier than optimum moisture content are capable to sustain appreciable loads with little compression due to the rigidity introduced by soil suction and were considered prone to collapse as indicated by the many results available (Escario and Saez 1973, Vilar and Gaioto 1994). To take into account the influence of moisture, the method by Gibbs (1961) also considers the moisture content deviation (Δw), the difference between the actual soil moisture content (w) and the Standard Proctor optimum moisture content (w_{opt}).

$$\Delta w = w - w_{opt} \tag{2}$$

The value of Δw is negative when the soil is drier than optimum moisture content and positive when the soil is wetter than optimum moisture content.

As the original criteria analyzed do not consider load and the available information about load and collapse is limited, this variable was not considered in this paper.

To evaluate the boundary between collapsible and non collapsible soils different sets of test results on natural specimens of collapsible and non-collapsible soils were used. These data are gathered in Table 1 and as many of the original papers did not present all the soil characteristics data, some values were calculated or even assumed, when needed. In assuming data, especially for the Standard Compaction Tests, practical Tables based on Universal Classification System were used such as Table 2 in Appendix, adapted from USBR (1998). In the case of calculated or assumed values, the data are inserted in Table 1 in italic. In the same table, soils said as non-collapsible are identified by an asterisk.

Figure 4 shows the plot of natural relative compaction (RC_n) and moisture content deficiency of the soil.

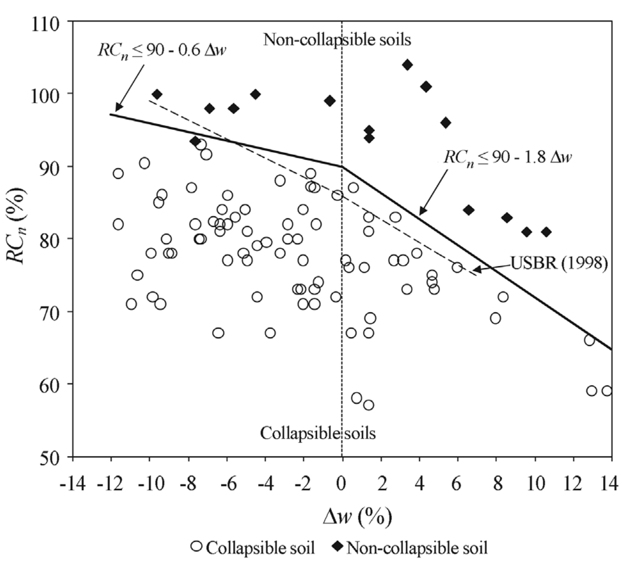

Figure 4 - Relative compaction RC_n versus moisture content deviation of collapsible soils.

Inspection of Table 1 and Fig. 4 shows that the vast majority of collapsible soils are low dry density and dry soils, that is, they are poorly compacted materials showing dry density lower than 1.50 g/cm^3 and moisture content deficiency. Collapsible behavior was also noticed for soils wetter than optimum moisture content, however always related to loose soils, with natural degree of compaction lower than approximately 85%. In general, collapsible soils showed RC_n lower than 90%, although at least one soil with RC_n = 95% has suffered collapse. In this case the moisture content deviation was of about -8%, suggesting that even denser soils if dry can be subjected to collapse deformation if under large overburden stress. However, in many of these situations related to dense and fairly dry soils, the stress needed to induce collapse strains are very large, beyond the range of practical interest for the Geotechnical Engineer. Very few data of non-collapsible soil has been reported and they show usually RC_n larger than 90% as is the case of the soil from arid climate tested by Carvalho (1994) that has not shown wetting induced strains although they were dry and wetted at stresses as large as 400 kPa or present large moisture content when RC_n is lower than 90% as reported by Arman & Thornton (1973).

The trend shown by the data set in Fig. 4 confirms the original proposition by Gibbs, however also indicates that it should be slightly enlarged since some reported collapsible soils plotted above the boundary line. In this sense, the available information has lead to the proposition of the limit line drawn in Fig. 4 that separates collapsible and non-collapsible soils. If the natural relative compaction and moisture content deficiency plot below the limit line, significant wetting induced deformation should be expected and additional and more directed investigation may be performed to adequately characterize collapsing behavior, which necessarily includes the state of stress acting on the soil. As it is known, the magnitude of collapse strain depends on the stress and usually after reaching a maximum it then decreases to negligible values as the load is increased and the soil is compressed to a denser condition.

The limit lines of Fig. 4 also allows stating a relationship between dry density and moisture content deviation and the following condition should stand for a soil to be considered potentially collapsible:

a) for dry soils ($w < w_{opt}$)

$$100 \cdot \frac{\rho_d}{\rho_{d\,\max}} \le 90 - 0.6\Delta w \tag{3}$$

b) for wet soils ($w > w_{opt}$)

$$100 \cdot \frac{\rho_d}{\rho_{d\,\max}} \le 90 - 1.8\Delta w \tag{4}$$

In these equations, Δw is expressed as percentage. The relationships are valid up to -12% on the dry side and to

+14% on the wet side, since those were the limiting moisture content deviations found in the referenced papers.

As there are very few data above the limit line, caution should be exerted for soils that plot in the vicinity of this limit line. In this case, additional investigation should be performed to confirm collapse behavior, considering the key factors that influence collapse deformation.

Figure 4 allows also suggesting a tentative single index separating collapsing and non-collapsing soils. As can be seen the dry soils, that is, the soils with moisture content lower that optimum moisture content show RC_n lower than 90%, figure that tends to decrease for moisture content above the optimum water content. This index is larger than the value proposed by de Mello (1973) and therefore should replace it.

These expedite methods of collapsible soil identification were checked and updated to support the initial stages of investigation and to orientate, but not to substitute, the more elaborate investigation techniques, that will support the designer on choosing the solution for the problem under analysis. Thus, by performing very simple and conventional measurements, as the in situ unit weight and moisture content and the parameters of Standard Proctor test it is possible to define whether a given soil is potentially collapsible. In situ density can be measured by conventional methods, such as the sand replacement method, among others, while moisture content can be measured through speedy moisture tester or other applicable method, such as the oven method. In some instances, and considering that the proposed method was devised for preliminary investigation, an experienced engineer can use results from other sources and indirectly evaluate the soil characteristics of interest. For instance, soil identification following ASTM D2488 - Standard practice for description and identification of soils (visual-manual procedure) - allows classifying the soil according to the Universal Soil Classification System and obtain average Standard Proctor parameters from practical tables, such as Table 2 in the Appendix.

It must be recognized that the methods are difficult to apply to deeper soil horizons, unless some measurement of soil density and moisture content is available. In this case and indeed in a more comprehensive way, a natural strategy would be using some field test such as the common Standard Penetration Test, searching for low density sandy and clayey soils that are unsaturated. Some SPT profiles of collapsible soils are available, especially from the Center-West and Southeast Brazil (Cavalcante *et al.*, 2007 among others), which shows collapsible soils as being unsaturated loose to medium compact sandy soils and unsaturated soft clayey soils. However, the relationship between SPT and soil collapse must be seen with caution, since SPT is influenced by moisture content or suction. For instance, for collapsible soils developed under arid and semi arid climate, Ferreira (1995) and Souza Neto (2004) have shown that these highly desiccated soils can show high SPT values during dry season, suggesting that their relative density is high. As an example, SPT performed in the dry season in a sandy soil of Petrolândia, Pernambuco, Brazil presents SPT higher than 10 blows. After three hours of flooding the SPT was reduced to about 70% of original value in the shallower portions of the soil profile (Ferreira, 1995) and after the rainy season, some measured values of SPT were reduced from 10 to about 5 blows, showing the influence of moisture content and suction on the number of blows and on the relative density (Souza Neto, 2004). The influence of moisture content or suction on the blow number of SPT has been addressed by other authors, such as Reginatto (1971) and Camapum de Carvalho *et al.* (2001) suggesting that SPT is of limited value in identifying collapsible soils. A possible alternative to overcome this point is to perform field tests with soil at natural moisture content and after flooding, analyzing the differences between both values; however the subject demands additional research in order to establish its efficiency.

Finally, it is worth to say that that both criteria updated in this paper indicate the collapsible nature of the sandy soil of Petrolândia, above referred to, since it shows natural relative compaction (RC_n) lower than 80% and moisture content deviation of about -7% in the dry season (Ferreira, 1995).

4. Conclusion

Collapsible soils are typically low density non-saturated soils that can be of different origins. These soils experience volume reduction or collapse strain when wetted at an almost constant stress, usually larger than the overburden stress. Collapse strains are known to depend on the dry density and moisture content of the soil and on the stress acting on it. An expedite method first proposed by Gibbs (1961) to identify collapsible soils, which considers that collapsible soils are naturally poorly compacted soils, with moisture content deficiency has been updated. The method takes into account both in situ density and moisture content and Standard Proctor compaction parameters of the soil, related through the natural relative compaction (RC_n), relationship between in situ dry density (ρ_d) and maximum dry density (ρ_{dmax}) and moisture content deviation (Δw), the difference between the in situ soil moisture content (w) and optimum moisture content (w_{opt}). Both parameters are related through Eq. 3 for dry soils and Eq. 4 for wet soils, that is soils that show moisture content larger that optimum moisture content.

The data gathered also allowed a simplified criterion that takes into account only dry density. In this option, soils that show negative moisture deviation and natural relative compaction (RC_n) lower than 90% are classified as collapsible. This index tends to reduce as moisture content increases above optimum moisture content.

The updated methods were devised for preliminary studies and to optimize more complete investigation analysis, not to substitute them, remembering that besides dry density and moisture content, the collapse strains are also function of stress whose influence is not addressed in this paper.

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Appendix

Table 1 - Data from collapsible soil.

Soil	Clay (%)	Silt (%)	Sand (%)	w_L (%)	w_P (%)	Class. USCS	n (%)	S_r (%)	ρ_d (g/cm^3)	w (%)	ρ_{dmax} (g/cm^3)	w_{opt} (%)	Δw (%)	RC_n (%)	Reference
1	32	16	52	27	13	SC	42	37	1.551	10.1	2.125	12.4	-2.3	73	Benvenuto (1983)
2	27	7	66	23	16	SC	50	41	1.510	-	1.936	12.4	-	78	Ferreira *et al.* (1989)
3	2	8	89	NL	NP	SW	40	13	1.590	3.4	1.980	10.8	-7.4	80	Ferreira (1993)
4	3	19	76	18	13	SM	42	6	1.544	1.5	1.970	11.4	-9.9	78	Ferreira (1993)
5	50	30	20	50	25	CL	66	45	0.900	32.8	1.768	16.4	16.4	51	Ferreira *et al.* (1989)
6	50	37	13	80	46	MH	65	-	0.931	-	1.372	33.1	-	68	Ferreira *et al.* (1989)
7	61	6	33	54	27	CH	61	42	1.030	25.3	1.531	24.8	0.5	67	Ferreira *et al.* (1989)
8	42	15	43	41	19	CL	54	38	1.270	16.1	1.768	16.4	-0.3	72	Ferreira *et al.* (1989)
9	45	20	35	37	26	ML	63	39	1.130	21.6	1.645	20.1	1.5	69	Ferreira *et al.* (1989)
10	41	24	25	48	35	ML	65	55	1.080	33.0	1.645	20.1	12.9	66	Ferreira *et al.* (1989)
11	15	35	50	25	15	SC	46	40	1.440	12.8	1.906	12.4	0.4	76	Ferreira *et al.* (1989)
12	46	12	42	58	27	CH	54	60	1.240	26.2	1.531	24.8	1.4	81	Ferreira *et al.* (1989)
13	45	5	50	58	34	SM	59	50	1.120	26.2	1.906	12.4	13.8	59	Ferreira *et al.* (1989)
14	60	15	25	70	30	CH	61	56	1.050	32.8	1.531	24.8	8.0	69	Ferreira *et al.* (1989)
15	-	-	-	50	10	CL	54	33	1.250	14.4	1.768	16.4	-2.0	71	Ferreira *et al.* (1989)
16	32	23	44	28	18	SC	46	34	1.414	10.7	1.802	13.9	-3.2	78	Carvalho (1994)
17	27	14	59	NL	NP	SP	51	3	1.305	1.1	1.827	10.5	-9.4	71	Cardoso *et al.* (1998)
18	56	3	34	36	25	CL	46	41	1.388	13.2	1.620	19.1	-5.9	86	Carvalho (1994)
19(*)	58	6	33	38	25	CL	42	45	1.469	12.6	1.574	20.2	-7.6	93	Carvalho (1994)
20	39	27	33	32	20	CL	44	34	1.467	9.9	1.753	14.9	-5.0	84	Carvalho (1994)
21	17	51	32	28	18	CL	45	27	1.430	8.7	1.742	15.0	-6.3	82	Carvalho (1994)
22(*)	44	39	17	36	22	CL	34	57	1.740	10.8	1.768	16.4	-5.6	98	Carvalho (1994)
23(*)	5	24	71	NL	NP	SP	31	36	1.831	6.0	1.827	10.5	-4.5	100	Carvalho (1994)

Table 1 - Cont.

Soil	Clay (%)	Silt (%)	Sand (%)	w_L (%)	w_P (%)	Class. USCS	n (%)	S_r (%)	ρ_d (g/cm³)	w (%)	ρ_{dmax} (g/cm³)	w_{opt} (%)	Δw (%)	RC_n (%)	Reference
24	15	6	79	18	11	SC	46	22	1.440	7.2	2.040	8.6	-1.4	71	Rodrigues and Vilar (2006)
25	16	6	78	-	-	SC	36	4	1.684	0.8	2.100	8.1	-7.3	80	Mahler and Mendonça (1994)
26	43	12	45	37	25	ML	67	30	0.930	21.5	1.645	20.1	1.4	57	Conciani (1997)
27	39	51	10	45	32	ML	64	50	0.971	33.1	1.645	20.1	13.0	59	Conciani (1997)
28	67	19	13	52	35	MH	64	41	1.072	24.3	1.372	33.1	-8.8	78	Monacci *et al.* (1997)
29	39	48	13	66	42	MH	60	53	1.152	28.1	1.372	33.1	-5.0	84	Monacci *et al.* (1997)
30	24	35	41	28	18	SC	49	42	1.370	14.8	1.715	17.6	-2.8	80	Ferreira *et al.* (1998)
31	7	22	70	23	14	SC	41	19	1.620	5.2	1.965	11.1	-5.9	82	Ferreira *et al.* (1998)
32	48	39	13	51	38	MH	67	43	0.953	30.4	1.650	29.6	0.8	58	Collares and Vilar (1998)
33	13	22	65	23	14	SC	43	22	1.540	6.1	1.906	12.4	-6.3	81	Costa Jr. (2001)
34	74	16	10	57	42	MH	66	49	1.018	37.8	1.372	33.1	4.7	74	Gutierrez *et al.* (2009)
35	66	28	6	61	42	MH	62	65	1.133	35.9	1.372	33.1	2.8	83	Gutierrez *et al.* (2009)
36	27	8	65	25	15	SC	46	30	1.440	9.6	1.960	11.7	-2.1	73	Rodrigues and Lollo (2004)
37	clayey sand (0-6 m) clayey silt (6-12 m)			41	23	SC	51	56	1.374	20.8	1.906	12.4	8.4	72	Ferreira *et al.* (2004)
38a	7	2	91	NL	NP	SP-SM	39	4.6	1.628	1.08	1.800	11.3	-10.2	90.4	Souza Neto (2004)
38b	9	3	88	14	NP	SP-SM	39	6.9	1.618	1.68	1.880	11.0	-9.3	86.0	Souza Neto (2004)
38c	14	2	84	16	12	SM	39	8.7	1.615	2.11	1.970	9.7	-7.6	82.0	Souza Neto (2004)
38d	15	0	82	17	15	SM	37	9.8	1.673	2.15	2.030	8.8	-6.6	82.4	Souza Neto (2004)
38e	16	5	79	19	13	SM/SC	31	19.1	1.833	3.18	2.000	10.2	-7.0	91.7	Souza Neto (2004)
39	-	-	-	-	-	-	-	-	-	-	-	-	-11.6	89.0	Ferreira (2015)
40	-	-	-	-	-	-	-	-	-	-	-	-	-1.6	87.2	Ferreira (2015)
41	-	-	-	-	-	-	-	-	-	-	-	-	-3.9	79.5	Ferreira (2015)
42	10-15	82-87	3	NL	NP	SM	48	5	1.409	1.7	1.877	12.3	-10.6	75	Lutenegger and Saber (1988)

Table 1 - Cont.

Soil	Clay (%)	Silt (%)	Sand (%)	w_L (%)	w_P (%)	Class. USCS	n (%)	S_r (%)	ρ_d (g/cm^3)	w (%)	ρ_{dmax} (g/cm^3)	w_{opt} (%)	Δw (%)	RC_n (%)	Reference
43	10-15	82-87	3	33	NP	SM	51	4	1.329	1.4	1.877	12.3	-10.9	71	Lutenegger and Saber (1988)
44	10-15	82-87	3	33	NP	SM	50	7	1.358	2.5	1.877	12.3	-9.8	72	Lutenegger and Saber (1988)
45	11	68	-	31	24	ML	47	23	1.343	8.5	1.645	20.1	-11.6	82	Klukanová and Frankovská (1998)
46	7	38	-	24	21	SM	38	25	1.738	5.0	1.877	12.3	-7.3	93	Klukanová and Frankovská (1998)
47	6	26	-	25	20	SM-SC	40	39	1.656	9.1	1.877	12.3	-3.2	88	Klukanová and Frankovská (1998)
48(*)	29	50	-	43	25	CL	36	45	1.738	9.5	1.768	16.4	-6.9	98	Klukanová and Frankovská (1998)
49(*)	15	62	-	27	16	CL	31	37	1.775	6.8	1.768	16.4	-9.6	100	Klukanová and Frankovská (1998)
50	15	38	39	29	16	CL	44	60	1.535	17.0	1.768	16.4	0.6	87	Feda (1966)
51	17	42	41	26	16	CL	42	56	1.574	14.8	1.768	16.4	-1.6	89	Feda (1966)
52(*)	30	42	28	39	21	CL	39	76	1.664	17.8	1.768	16.4	1.4	94	Feda (1966)
53	20	30-40	40-50	18	13	SM-SC	46	27	1.430	8.5	1.940	9.7	-1.2	74	Phien-wej at al (1992)
54	16	21	63	21	17	SM-SC	-	-	1.254	8.3	1.874	12.0	-3.7	67	Alawaji (2001)
55	19	35	45	23	17	SM-SC	-	-	1.339	18.8	1.835	14.0	4.8	73	Alawaji (2001)
56	67% passing 200 sieve			-	-	-	46		1.433	9	1.868	11.0	-2.0	77	Houston *et al.* (1988)
57	18	72	10	28	22	CL - ML	48	53	1.390	18.1	1.645	20.1	-2.0	84	Delage *et al.* (2005)
58	-	-	-	39	16	CL	49	29	1.367	10.5	1.768	16.4	-5.9	77	Jennings and Knight (1957)
59	8	72	20	30	20	-	-	-	1.160	10.0	1.720	16.4	-6.4	67	Clevenger (1956)
60	12	60	28	-	-	SM	48	25	1.564	6.8	1.877	12.3	-5.5	83	Mustafaev *et al.* (1974)
61	18	-	-	-	-	SM	46	33	1.500	10.0	1.877	12.3	-2.3	80	Reznik (1992)
62	78	-	-	64	46	MH	60	44	1.100	24.0	1.372	33.1	-9.1	80	Foss (1973)

Table 1 - Cont.

Soil	Clay (%)	Silt (%)	Sand (%)	w_L (%)	w_P (%)	Class. USCS	n (%)	S_r (%)	ρ_d (g/cm^3)	w (%)	ρ_{dmax} (g/cm^3)	w_{opt} (%)	Δw (%)	RC_n (%)	Reference
63	< 30	> 60	10	31	16	CL	44	53	1.540	15.0	1.768	16.4	-1.4	87	Zur and Wiseman (1973)
64	-	-	-	-	-	CL	51	30	1.275	12.0	1.768	16.4	-4.4	72	Beles *et al.* (1969)
65	-	-	-	45	-	CL-SC	45	48	1.442	15.0	1.877	12.3	2.7	77	Gibbs and Bara (1967)
66	silt (0.06 - 0.01 mm)			32	22	CL	47		1.404	-	1.768	16.4	-	79	Derbyshire and Mellors (1988)
67	18	95% passing 200 sieve		37	19	CL	48	24	1.380	8.5	1.772	17.5	-9.0	78	Sultan (1971)
68	18	95% passing 200 sieve		43	20	CL	43	38	1.482	11.3	1.772	17.5	-6.2	84	Sultan (1971)
69	18	95% passing 200 sieve		67	28	CH	45	40	1.432	12.6	1.772	17.5	-4.9	81	Sultan (1971)
70	-	-	-	23	17	CL-ML	55	35	1.280	15.0	1.645	20.1	-5.1	78	Reginatto and Ferrero (1973)
71	20	17	43	25	14	SC	-	-	1.280	-	1.810	-	-	71	Dudley (1970)
72	12	16	72	21	12	SC	-	-	1.632	-	1.984	-	-	82	Dudley (1970)
73	10	19	38	20	16	GW-SW	37	34	1.708	7.4	2.226	12.3	-4.9	77	Dudley (1970)
74	5	85	10	NL	NP	SM	69	28	0.881	22.0	1.049	-	-	84	Dudley (1970)
75	2	68	30	NL	NP	SM	58	25	1.041	14.0	1.302	-	-	80	Dudley (1970)
76	7	84	9	33	25	ML	56	36	1.186	17.0	1.516	-	-	78	Dudley (1970)
77	10	70	20	-	-	ML	-	-	1.522	22.0	1.778	-	-	86	Arman and Thornton (1973)
78	18	70	12	-	-	ML	-	-	1.234	18.0	1.626	-	-	76	Arman and Thornton (1973)
79	10	70	20	-	-	ML	-	-	1.200	21.3	1.606	16.6	4.7	75	Arman and Thornton (1973)
80	10	70	20	-	-	ML	-	-	1.216	22.6	1.606	16.6	6.0	76	Arman and Thornton (1973)
81	10	70	20	-	-	ML	-	-	1.312	15.3	1.606	16.6	-1.3	82	Arman and Thornton (1973)
82	10	70	20	-	-	ML	-	-	1.264	12.2	1.606	16.6	-4.4	79	Arman and Thornton (1973)
83	10	70	20	-	-	ML	-	-	1.248	20.5	1.606	16.6	3.9	78	Arman and Thornton (1973)

Table 1 - Cont.

Soil	Clay (%)	Silt (%)	Sand (%)	w_L (%)	w_P (%)	Class. USCS	n (%)	S_r (%)	ρ_d (g/cm^3)	w (%)	ρ_{dmax} (g/cm^3)	w_{opt} (%)	Δw (%)	RC_n (%)	Reference
84(*)	10	70	20	-	-	ML	-	-	1.520	18.0	1.606	16.6	1.4	95	Arman and Thornton (1973)
85(*)	10	70	20	-	-	ML	-	-	1.616	21.0	1.606	16.6	4.4	101	Arman and Thornton (1973)
86(*)	10	70	20	-	-	ML	-	-	1.664	20.0	1.606	16.6	3.4	104	Arman and Thornton (1973)
87(*)	10	70	20	-	-	ML	-	-	1.584	16.0	1.606	16.6	-0.6	99	Arman and Thornton (1973)
88	10	70	20	-	-	ML	-	-	1.168	20.0	1.606	16.6	3.4	73	Arman and Thornton (1973)
89	10	70	20	-	-	ML	-	-	1.328	18.0	1.606	16.6	1.4	83	Arman and Thornton (1973)
90(*)	10	70	20	-	-	ML	-	-	1.536	22.0	1.606	16.6	5.4	96	Arman and Thornton (1973)
91	10	70	20	-	-	ML	-	-	1.072	18.0	1.606	16.6	1.4	67	Arman and Thornton (1973)
92	18	70	12	-	-	ML	-	-	1.232	18.0	1.624	16.8	1.2	76	Arman and Thornton (1973)
93	18	70	12	-	-	ML	-	-	1.328	14.0	1.624	16.8	-2.8	82	Arman and Thornton (1973)
94	18	70	12	-	-	ML	-	-	1.248	17.0	1.624	16.8	0.2	77	Arman and Thornton (1973)
95	18	70	12	-	-	ML	-	-	1.248	20.0	1.624	16.8	3.2	77	Arman and Thornton (1973)
96(*)	12	68	20	-	-	ML	-	-	1.408	23.0	1.728	13.4	9.6	81	Arman and Thornton (1973)
97(*)	12	68	20	-	-	ML	-	-	1.456	20.0	1.728	13.4	6.6	84	Arman and Thornton (1973)
98(*)	12	68	20	-	-	ML	-	-	1.440	22.0	1.728	13.4	8.6	83	Arman and Thornton (1973)
99(*)	12	68	20	-	-	ML	-	-	1.392	24.0	1.728	13.4	10.6	81	Arman and Thornton (1973)

ρ_s is solid density, n is porosity, ρ_d is dry density, w is moisture content, ρ_{dmax} is Standard Proctor maximum dry density, w_{opt} is Standard Proctor optimum moisture content, Δw is moisture deviation related to the Standard Proctor optimum moisture content and RC_n is relative compaction, w_L is liquid limit, w_P is plasticity limit. (*) soils tested as non-collapsible.

Table 2 - Average engineering properties of compacted soils (adapted from USBR 1998, Earth Manual).

Soil GROUP NAMe	USCS soil type	Compaction	
		$\gamma_{d\,max}$ (kN/m^3)	w_{opt} (%)
Well-graded gravel	GW	19.89	11.4
Poorly graded gravel	GP	19.07	12.2
Silty gravel	GM	18.19	15.7
Clayey gravel	GC	18.54	14.2
Well-graded sands	SW	20.19	9.1
Poorly graded sands	SP	18.27	10.5
Silty sands	SM	18.77	12.3
Clayey sands	SC	19.06	12.4
Silt	ML	16.45	20.1
Lean clay	CL	17.68	16.4
Elastic silt	MH	13.72	33.1
Fat clay	CH	15.31	24.8