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Identifying Collapsible Soils from Seismic Cone (SCPT): A Qualitative Approach

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Abstract: Collapsible soils are unsaturated low-density soils that undergo abrupt settlement when flooded without any increase in the in-situ stress level. The first stage of the site characterization is identifying collapsible soils, since these are problematic soils. Seismic cone testing (SCPT) has been increasingly used for site characterization, because it allows combining stratigraphic logging with the maximum shear modulus (G_0) determination. In this paper, laboratory and in-situ tests carried out at 21 sites with collapsible and non-collapsible soils are interpreted to differentiate between such soils, based on the seismic cone test (SCPT). Collapsible soils have G_0/q_c values greater than 23 and q_{c1} values less than 70, while non-collapsible soils have G_0/q_c values less than 23 and q_{c1} values greater than 70. The investigated collapsible soils have microstructure (bonding/cementation), but the classical approach cannot be sufficient to identify collapsible soils alone. An approach was used to identify collapsible soils based on maximum shear modulus (G_0), normalized cone resistance (q_{c1}), and cone resistance (q_c). The chart G_0/q_c versus q_{c1} and boundaries is an alternative for distinguishing between collapsible and non-collapsible soils in the early stage of site investigation. This qualitative approach should be used in the preliminary investigation phase to select potentially collapsible soils and helps guide the sampling of potentially collapsible soils for laboratory testing. Further SCPT data from different soil types, particularly the collapsible ones, are valuable to adjust or confirm the boundary equations suggested.

Keywords: seismic cone; collapsible soils; small-strain stiffness; G_0/q_c ratio; identification

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1. Introduction

Collapsible soils occur in different countries around the world and cover a wide range of soils (e.g., residual soils, alluvial fans, tropical soils, and loess) [1,2]. These soils are unsaturated, have low water content, low specific gravity, meta-stable structure, relatively low compressibility, strength in the dry state, and are susceptible to significant volumetric variations when there is an increase in water content [1–3]. Moreover, any soil compacted when dry is collapsible [1,3–5]. Soil collapse is a significant geotechnical problem and can cause differential settlements, affecting buildings and civil structures. The first stage of the site characterization is identifying collapsible soils, since these are problematic soils [2,6].

The characterization of collapsible soils is usually carried out based on laboratory tests [7–9]. However, laboratory tests require high-quality, undisturbed soil samples truly representative of the in-situ conditions and are time-consuming and expensive. Plate load [10] and downhole collapse [11] in-situ tests may be carried out to identify collapsible soils; however, these tests are expensive, time-consuming, and not very feasible at great depths. An in-situ test, such as the seismic cone (SCPT), could identify collapsible soil or guide the selection of potential collapsible soils samples for laboratory tests. The SCPT test allows combining stratigraphic logging, estimative of geotechnical parameters, and specific measurement of the maximum shear modulus (G_0), which is a modern approach for site characterization.

The geotechnical soil conditions assessment using the cone penetration test (CPT) is based on cone resistance (q_c) and sleeve resistance (f_s); however, it is not always reliable, because q_c and f_s are not sensitive to stress history, aging, and cementation [12–14]. As a result, it is suggested that cone resistance should be correlated with the small strain stiffness [12,14–17]. The ratio G_0/q_c not only improves soil classification, but provides more accurate estimates of soil parameters from a theoretical perspective [13,15].

Soil stiffness (e.g., maximum shear modulus) is dependent on void ratio, stress level, stress history, and time effects [18,19], while soil strength (e.g., cone resistance) is dependent on void ratio, stress level, soil anisotropy, grain crushability, rate of shearing, and stress history [20,21]. So, the stress-strain response of soils at small- and large-strain levels is known to follow different functions of the same variables, and the G_0/q_c ratio could be interesting in-situ parameter to assessing soil condition, such as collapsible soils.

The aim of this paper is to introduce and discuss a qualitative approach in order to identify collapsible soils by SCPT from the G_0/q_c versus normalized cone penetration resistance (q_{c1}). Empirical equations, the lower, and the upper bounds are suggested to define the collapsible and the non-collapsible soils zones. They were established based on SCPT, CPT, and seismic data (downhole tests) from 21 worldwide sites documented in the literature.

2. SCPT in Unusual Soils

Unusual geo-materials (e.g., bonded soils, residual soils, unsaturated soils, collapsible soils, and tailings) present a unique behavior because of the geological and/or pedological formation processes. Bonding (cementation) and structure, cohesive-friction nature, soil suction, and anisotropy derived from relic structures, variable fabric and mineralogy govern the behavior of these soils. In this sense, the methods for interpreting in-situ tests in unusual materials may not be adequate and may lead to unrealistic behavior predictions, since they were developed for the drained and undrained mechanical behavior of sedimentary clays and the drained behavior of reconstituted young sands without microstructure [14,16].

One of the major applications of the SCPT is to define site stratigraphy and classify the soil type based on charts that correlate cone resistance and sleeve friction to soil type [22–24]. However, these charts predict the soil behavior type (SBT) (in-situ mechanical behavior of the soil) and not the soil classification, considering physical characteristics (e.g., plasticity and grain-size distribution) [12]. The different measurements combined into a single sounding offer a powerful means of assessing the mechanical behavior of unusual soils [16,25]. So, emphasis was placed on correlations with mechanical properties based on combining measurements of independent parameters or indices, such as the ratio of the elastic stiffness to ultimate strength (G_0/q_c, G_0/N_{SPT}) and the ratio of q_c/ψ_L, where N_{SPT} is the SPT N value and ψ_L is the pressuremeter limit pressure.

The ratio G_0/q_c is a measurement of the relationship between elastic stiffness and ultimate strength. This ratio increases with age and cementation, mainly due to the stronger effect of age and cementation on G_0 than on q_c, while all other factors (stress history, in-situ stress state, etc.) are constant [13,14,26]. Moreover, the G_0/q_c ratio concept is supported by fundamental mechanics [27,28].

Robertson et al. [29] proposed a chart that plots normalized cone penetration (Q_t) against the ratio of small-strain shear modulus with corrected penetration resistance (G_0/q_c). This chart can be used alongside the traditional CPT classification charts to identify compressible soils, as well as the effect of aging and cementation. Schnaid et al. [16] proposed a chart and boundaries by correlating G_0/q_c versus q_{c1}, a dimensionless normalized cone resistance defined as:

$$q_{c1} = \left(\frac{q_c}{p_a}\right) \cdot \sqrt{\frac{p_a}{\sigma\prime_v}} \quad (1)$$

where p_a = atmospheric pressure and σ'_v = vertical effective stress. This relationship can be used to evaluate the possible effects of compressibility, stress history, degree of cementation, and ageing for a given profile [26].

According to Robertson [12], the relationship between physical characteristics and in-situ behavior is mainly influenced by geologic factors such as age and cementation. Therefore, it is first necessary to identify whether soils have significant aging/cementation (microstructure), since it can influence the in-situ soil behavior and the effectiveness of any classification system based on in-situ tests. Several researchers have discussed that collapsible soils have cemented structures contacts between soils grains, resulting in a dry shear strength to their loose and unstable soil structure [1,2]. For example, the bonding (cementation) in collapsible loess can be attributed to calcium carbonate and clays, where the calcium carbonate is regarded as one of the main bonding materials in loess soils, since it is found not only as film coating on grains but also because of its concentration at grain contacts [1]. The cementing agent on the collapsible soils form the southwestern United States has been observed to be dried clay slurry and salts such as calcium carbonate and calcium sulfate compounds [30,31]. Clay and silt particle aggregation (cementation) in the collapsible tropical soils from Brazil are due to the action of iron and aluminum oxides and hydroxides, typical of lateritic soils [32–34].

Robertson [12], following the work of Schneider and Moss [17], proposed a chart that correlates Q_{tn} versus I_G (small-strain rigidity index = G_0/q_n) to identify the presence of aging/cementation (microstructure) in soils. The modified normalized small-strain rigidity index, K^*_G, assesses the cementation/aging in a given soil, computed as:

$$K^*_G = \left(\frac{G_0}{q_n}\right)(Q_{tn})^{0.75} \quad (2)$$

where $G_0 = \rho.\ (V_s)^2$ (ρ is the soil mass density; V_s is the shear wave velocity), $q_n = q_t - \sigma_v$, is the net cone tip resistance, and Q_{tn} is the normalized cone tip resistance [12]. Schneider and Moss [17] and Robertson [12] showed that most young and uncemented sands (i.e., little or no microstructure) have $100 < K^*_G < 330$, and soils with $K^*_G > 330$ tend to have significant microstructure.

Collapsible soils are unusual materials with high porosity and relatively high small-strain stiffness because of interparticle bonding (cementation/bonding). The large strains induced by cone penetration destroy this interparticle bonding, resulting in low q_c [35,36]. Hence, the ratio between the G_0 and a specific in-situ test parameter, such as q_c, could be used to select and identify collapsible soils. This relationship has already been used to estimate the state parameter (ψ) [14], soil classification [13], and soil liquefaction in granular geomaterials [15].

Another important factor influencing the collapsible soil behavior is the soil suction [3,8,32,37]. Soil collapse occurs when there are decrease in suction and there are increase in stress above the preconsolidation pressure of the unsaturated soil [38]. The extent of collapse deformations (or settlements) depends on the intensity of the applied load and on the suction values before and after wetting [1]. Soil suction raises both small-strain and medium to- large-strain in-situ test parameters in unsaturated soils [39–41]. So, it was assumed the hypothesis that soil suction similarly affects the in-situ test parameters and the ratio between elastic stiffness and medium-to large- strain stiffness to qualitatively identify collapsible soils.

3. Description of Sites and Tests

SCPT, CPT, and seismic data were collected from 21 sites (Table 1). The investigated soils consist of silty clay, silts, and sands. Table 1 presents the main information of each site, such as the collapsible behavior, and the thickness of the collapsible horizon. The collapsible behavior was set based on authors' experimental data and information from the literature.

Table 1. Site designation, collapsible behavior, and references for the soils from each site.

Site	Behavior	Collapsible Horizon Thickness	References
Unesp 1	Collapsible	9.0	[42]
USP 1	Collapsible	8.0	[42]
Unicamp 1	Collapsible	10.0	[43]
UnB 1	Collapsible	9.0	[44,45]
UEL	Collapsible	12.0	[46,47]
Belgrade 1	Collapsible	10.0	[48]
Belgrade 2	Collapsible	10.0	[48]
Argentina loess	Collapsible	7.0	[49]
Ilha Solteira	Collapsible	8.0	[50]
Pereira Barreto	Collapsible	7.0	[2]
Unesp 2	Non-collapsible	-	[42]
USP 2	Non-collapsible	-	[42]
Unicamp 2	Non-collapsible	-	[43]
UnB 2	Non-collapsible	-	[44,45]
Dudley, MO	Non-collapsible	-	Paul Mayne's site [1]
Memphis, TN1	Non-collapsible	-	Paul Mayne's site [1]
Memphis, TN2	Non-collapsible	-	Paul Mayne's site [1]
FEUP	Non-collapsible	-	[51]
Lublin	Non-collapsible	-	[52]
Shenton Park	Non-collapsible	-	[53]
Perth CBD	Non-collapsible	-	[53]
Ledge Point	Non-collapsible	-	[53]
UFSC	Non-collapsible	-	[54]
Texas AandM	Non-collapsible	-	[53,55]
Dyke Road	Non-collapsible	-	[56]

[1] "https://geosystems.ce.gatech.edu/Faculty/Mayne/Research/index.html. (accessed on 14 January 2023)"

Table 2 summarizes some geotechnical parameters for the soils from all test sites, such as the liquid limit (w_{LL}), plasticity index (*PI*), ground water level (*GWL*), in-situ dry unit weight (γ_d), and USCS classification. The w_{LL} of the soils varied between 12% and 68%, the *PI* varied from *NP* (non-plastic) to 23%, and γ_d was 10.60–18.70 kN/m^3.

Table 2. Typical values of the liquid limits (w_{LL}), plasticity index (*PI*), ground water level (*GWL*), in-situ dry unit weight (γ_d) and USCS classification for the soils from each site.

Site	w_{LL} (%)	*PI* (%)	*GWL* (m)	γ_d (kN/m^3)	USCS
Unesp 1	23	- *	Below 20.0	14.9	SM
USP 1	30	12	Below 9.0	14.1	SC
Unicamp 1	55	16	14.0	11.2	MH
UnB 1	42	12	13.0	12.2	ML
UEL	61	16	15.0	10.6	MH
Belgrade 1	30	10	10.0	13.4	CL
Belgrade 2	30	10	10.0	13.4	CL
Argentina loess	25	5	11.0	13.0	SM
Ilha Solteira	20	11	9.0	15.8	SC

Table 2. *Cont.*

Site	w_{LL} (%)	PI (%)	GWL (m)	γ_d (kN/m^3)	USCS
Pereira Barreto	19	8	8.0	16.5	SC
Unesp 2	28	- *	20.0	17.5	SM
USP 2	29	11	12.0	16.0	SC
Unicamp 2	68	23	14.0	12.0	MH
UnB 2	35	10	13.0	14.2	ML
Dudley, MO	- ℵ	- ℵ	6.5	- ℵ	- ℵ
Memphis, TN1	- ℵ	- ℵ	8.0	- ℵ	- ℵ
Memphis, TN2	- ℵ	- ℵ	8.0	- ℵ	- ℵ
FEUP	33	16	9.0	16.8	SM
Lublin	- ℵ	- ℵ	Below 10.0	16.5	SM
Shenton Park	15	- *	7.0	18.7	SP
Perth CBD	18	- *	8.0	17.8	SP
Ledge Point	12	- *	Below 10.0	17.0	SP
UFSC	43	9	Below 15.0	14.0	SM
Texas AandM	18	- *	7.5	18.5	SP
Dyke Road	30	4	7.0	12.8	CL

* nonplastic; ℵ information not Available.

The collapsible behavior was defined based on laboratory tests (simple and double oedometer tests) from the author's database and information from the literature. The collapse potential of at least 2% was used to define the collapsible behavior and the thickness of the collapsible horizon [57]. Figure 1 illustrates representative data for collapsible soils determined by both single-and double-oedometer tests for Unesp, USP, Unicamp, UnB, UEL, Belgrade, Ilha Solteira, and Pereira Barreto. Figure 1a also presents the collapse potential (CP), determined by Jennings and Knight's [8] equation (Equation (3)), for the simple oedometer tests. The collapsible behavior and the thickness of the collapsible horizon for the soils from Argentina loess were assumed following information from the literature.

$$CP = \frac{\Delta e_c}{1 + e_0} \times 100\% \tag{3}$$

where CP is the collapse potential, Δe_c is the variation of the void ratio due to wetting, and e_0 is the initial void ratio.

4. Identify Collapsible Soils from SCPT

Although the SCPT has the potential for site characterization of non-textbook type geomaterial (e.g., unsaturated soils, residual soils, bonded soils, collapsible soils, tailings), the tendency to identify collapsible soil, combining the small-strain stiffness (G_0) and cone resistance (q_c), is still scarce in the literature [49]. The G_0/q_c ratio versus q_{c1} and Q_{tn} versus and I_G are used to identify collapsible soils following earlier proposals of how to identify unusual geomaterials using the relationship between large strain parameters, such as q_c, and G_0.

SCPT testing was conducted at most of these soils. CPT was carried out without the corresponding seismic test at UnB, UEL, Ilha Solteira, and Pereira Barreto. G_0 values were determined by multichannel analysis of surface waves (MASW) at the UnB site. For UEL, Ilha Solteira, and Pereira Barreto sites, the maximum shear modulus was estimated by the Barros and Pinto's [58] correlation, which was developed for use in residual tropical soils for Brazil based on SPT N values.

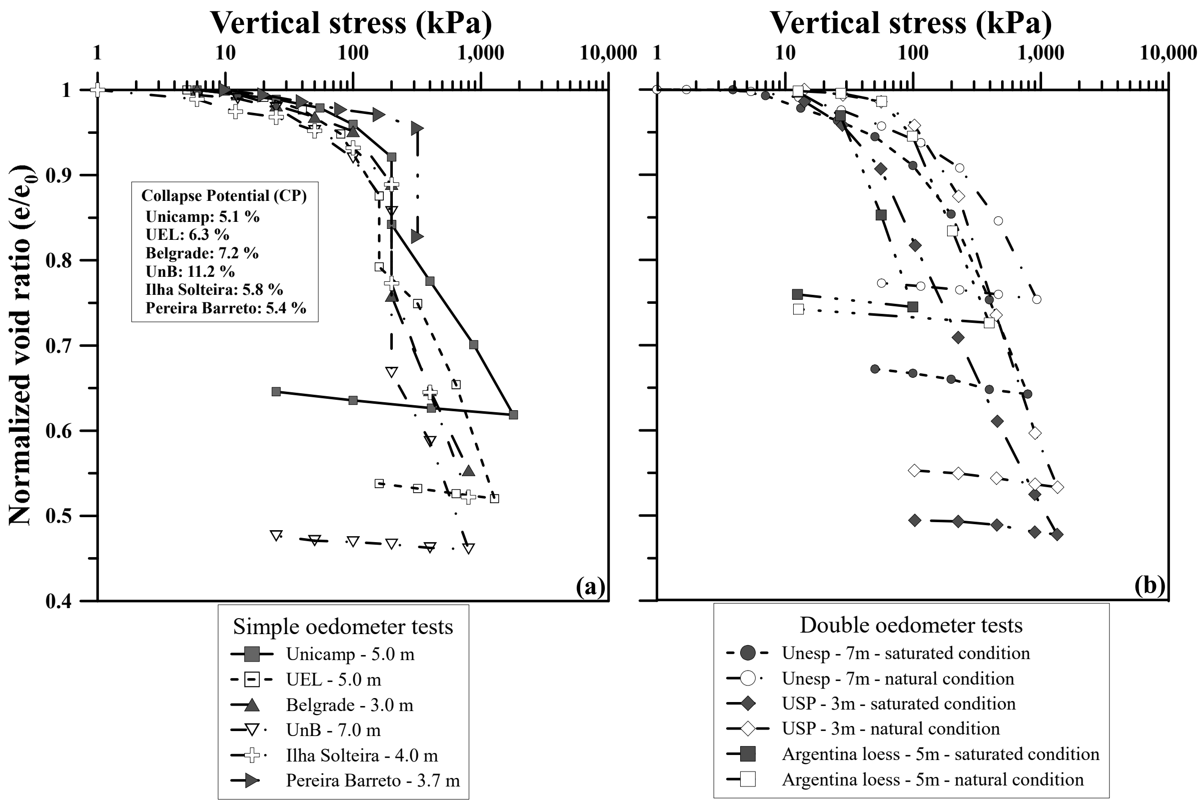

Figure 1. Typical (**a**) single-and (**b**) double-oedometer test data for collapsible soils (adapted from [2,47,48,50,59–63]).

Since the cone resistance (q_c) and V_s are often measured over different depth intervals (e.g., q_c is typically determined at 10 to 50 mm depth intervals), while V_s (and consequently, G_0) is usually determined over 0.5 to 1.0 m depth intervals, qc values was averaged over the V_s depth interval. For instance, when V_s was determined at 1.0 m depth intervals, the associated q_c (and consequently, Q_{tn} and q_{c1}) value was averaged over the same depth interval. Table 3 summarizes the range of Q_{tn}, I_G, G_0/q_c, q_c, and q_{c1}. The q_{c1} values generally range from 6 to 330, while G_0/q_c varies from 3 to 225. It can be observed in this table (Table 3) that the G_0/q_c is higher than 23 and q_{c1} is lower than 70 for collapsible soils. The high G_0/q_c values for the collapsible soils are related to interparticle bonding (cementation/bonding), which is mostly destroyed at medium to high strains induced by cone penetration and results in low q_c and consequently q_{c1}. Hence, the ratio of the elastic stiffness (G_0) to ultimate strength (q_c), can be used to select collapsible soils.

Table 3. Q_{tn}, I_G, G_0/q_c, q_c, and q_{c1} ranges for the soils from all sites.

Site	Q_{tn}	I_G	G_0/q_c	q_c (MPa)	q_{c1}
Unesp 1	32 to 50	26 to 102	25 to 99	1.2 to 4.3	20 to 37
USP 1	8.5 to 14	70 to 210	68 to 185	0.6 to 1.9	8 to 21
Unicamp 1	10 to 139	15 to 51	25 to 50	1.5 to 2.2	16 to 46
UnB 1	30 to 38	35 to 152	30 to 145	0.7 to 3.5	11 to 43
UEL	9 to 18	52 to 140	52 to 133	0.8 to 3.9	14 to 29
Belgrade 1	2 to 17	76 to 280	73 to 221	0.7 to 2.3	6 to 36
Belgrade 2	3.5 to 19	43 to 186	42 to 164	1.0 to 3.0	10 to 44
Argentina loess	60 to 119	22 to 37	23 to 38	2.0 to 7.0	56 to 70
Ilha Solteira	3 to 7.1	100 to 245	70 to 225	0.5 to 2.2	8 to 18.7
Pereira Barreto	10 to 28	27 to 67	24 to 65	1.5 to 4.4	25 to 60
Unesp 2	50 to 72	20 to 54	19 to 37	4.3 to 11.5	38 to 63

Table 3. *Cont.*

Site	Q_{tn}	I_G	G_0/q_c	q_c (MPa)	q_{c1}
USP 2	10 to 18	65 to 120	60 to 118	2.0 to 4.1	13 to 23
Unicamp 2	6 to 15	19 to 59	18 to 52	1.8 to 5.0	12 to 31
UnB 2	38 to 48	19 to 30	18 to 30	3.5 to 7.0	32 to 50
Dudley, MO	55 to 103	6 to 40	6 to 42	3.2 to 15.2	47 to 133
Memphis, TN1	10 to 207	10 to 65	3 to 50	2.4 to 8.2	10 to 255
Memphis, TN2	34 to 158	5 to 27	3 to 25	3.6 to 29.8	20 to 203
FEUP	40 to 83	27 to 43	28 to 42	3.4 to 6.3	45 to 72
Lublin	30 to 50	13 to 30	13 to 30	4.5 to 9.5	65 to 110
Shenton Park	25 to 48	16 to 35	17 to 35	3.6 to 10.0	59 to 103
Perth CBD	28 to 210	3 to 22	3 to 23	4.2 to 20.2	65 to 180
Ledge Point	25 to 140	5 to 22	5 to 25	3.8 to 28.0	50 to 330
UFSC	28 to 130	7 to 40	7 to 41	4.0 to 12.4	40 to 262
Texas AandM	40 to 95	10 to 16	10 to 15	6.5 to 10.2	80 to 191
Dyke Road	5 to 70	10 to 46	10 to 42	1.0 to 4.0	10 to 90

Table 3 also presents the range of cone resistance (q_c). The q_c value is lower than 4 MPa for collapsible soils. However, the relationship between cone resistance and soil collapse must be approached with caution, since in-situ tests, such as the cone resistance test, is influenced by moisture content or suction [39,41,64,65]. For instance, Ferreira [66] and Souza Neto [67] have shown that the highly desiccated collapsible soils developed under arid and semi-arid climates present high SPT N values during the dry season, suggesting that the soil has a high relative density. Devincenzi and Canicio [68] presented CPT data carried out in collapsible loose silts before and after wetting. The authors observed that the cone resistance decreases 38% upon saturation. Such behavior was also verified by Rocha [42] in collapsible sandy soils.

Therefore, a possible alternative to identify collapsible soils from in-situ tests (e.g., seismic cone) are combining the measurements from independent tests, such as the ratio of the elastic stiffness, to ultimate strength, such as G_0/q_c vs. q_{c1} and I_G vs. Q_{tn}.

Figure 2 plots the collapsible (Figure 2a) and non-collapsible (Figure 2b) soils data (Table 3) on the normalized rigidity index chart ($Q_{tn} - I_G$). The two datasets plot in significantly different regions, as shown in Figure 2. The collapsible data points have K^*_G > 330, indicating that microstructure is present for the collapsible soils, whereas the great majority of non-collapsible soil dataset falls in the range of 100 < K^*_G < 330, indicating little or no microstructure for these soils. Bauru 2, USP 2, and FEUP sites (non-collapsible soil) presented microstructure. This behavior occurs because Bauru 2 and USP 2 are saprolitic tropical soils, which presents layering and fissures, or bonding (cementation) related to the parent rock [43,69]. The FEUP site has a residual soil that exhibit some bond structure [70,71]. The presented results shows that the proposed method by Robertson [12] to identify soil with microstructure can be used as a first indicator of the presence of collapsible soils, since all investigated collapsible soils presented microstructure. However, this approach requires careful consideration, as non-collapsible soils also can present microstructure.

The dimensionless log-log chart of G_0/q_c and q_{c1} (Figure 3) is another approach to identifying collapsible soils by setting the bounded regions that differentiate collapsible and non-collapsible soils. The line in the chart that separates the region of collapsible soil and non-collapsible soil (upper non-collapsible/lower collapsible bound) was given by Equation (2). Most of the collapsible soils set falls in the region that is different from the zone occupied by most of the non-collapsible soils (Figure 3).

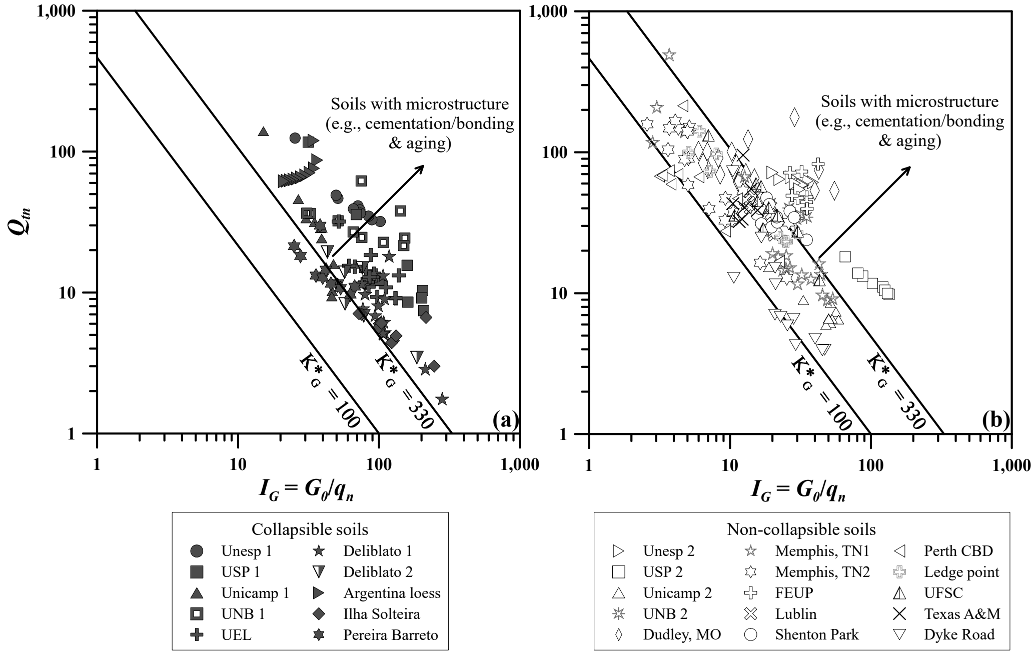

Figure 2. Q_{tn} versus I_G chart to identify collapsible (**a**) and non-collapsible (**b**) soils with microstructure.

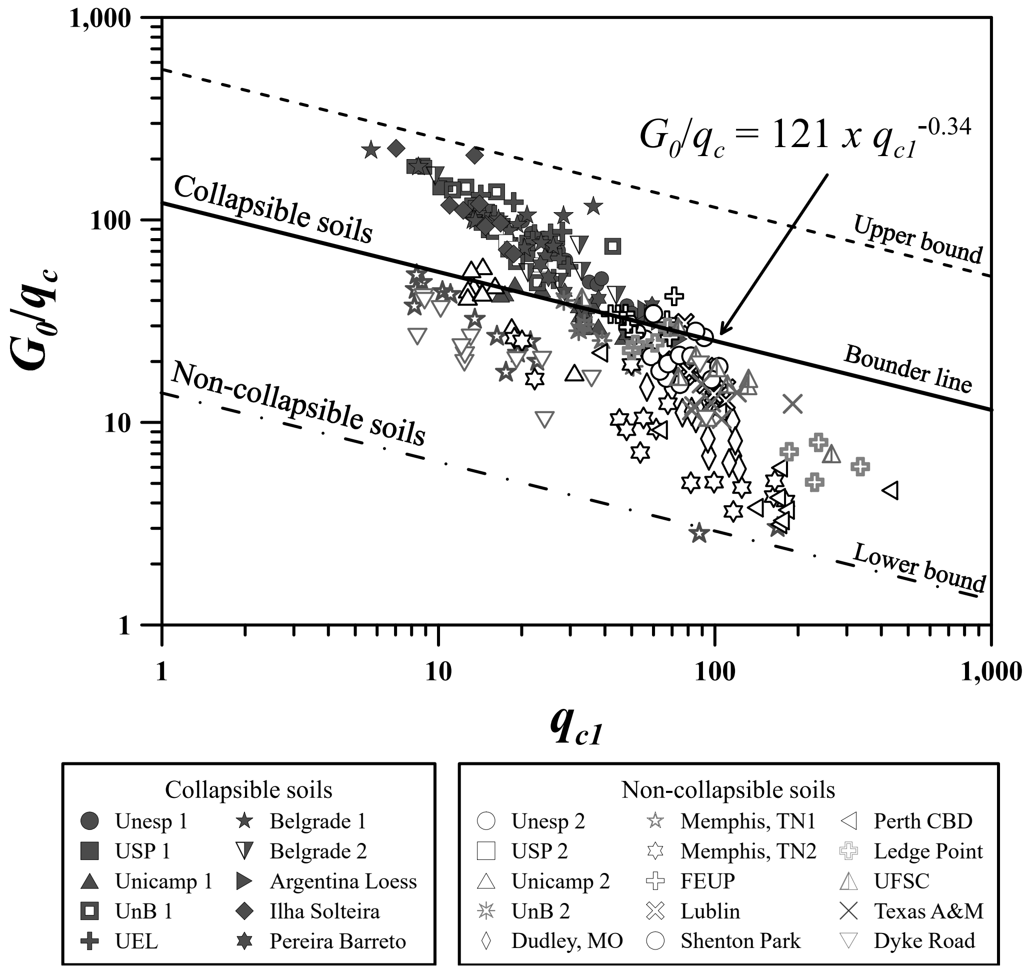

Figure 3. Upper and lower boundaries to separate collapsible from non-collapsible soils in the G_0/q_c versus q_{c1} chart and the data for all sites.

The line separating two regions between collapsible and non-collapsible soils (upper non-collapsible/lower collapsible soil) was defined using Equation (4):

$$\frac{G_0}{q_c} = A \times q_{c1}^{-B} \tag{4}$$

where A and B are constants that depends on the soil type.

The boundary equations are represented as follows:

Lower non-collapsible bound:

$$\frac{G_0}{q_c} = 15 \times q_{c1}^{-0.34} \tag{5}$$

Upper non-collapsible/lower collapsible bound:

$$\frac{G_0}{q_c} = 121 \times q_{c1}^{-0.34} \tag{6}$$

Upper collapsible bound:

$$\frac{G_0}{q_c} = 550 \times q_{c1}^{-0.34} \tag{7}$$

The upper non-collapsible/lower collapsible bound in Figure 3 was empirically determined considering the given database. The same slope was used for the upper collapsible and the lower non-collapsible soil bounds. Schnaid and Yu [14] and Schnaid et al. [16] demonstrated boundaries with the same slope in a plot of G_0/q_c vs. q_{c1} to define a region representative of unaged uncemented soils based on large laboratory calibration chamber tests and centrifuge tests, as well as by in-situ tests. Some datasets below or above the upper and lower limits or some dates are not identified concerning the collapsibility behavior (Figure 3). This can occur due to the soil compressibility, horizontal stress, fabric anisotropy, and some degree of cementation and aging, as well as the direction of propagation of shear waves. Shear wave velocity is related to the direction of propagation and polarization and can also be affected by the type of test to calculate the maximum shear modulus [14,26].

As a consequence of this data analysis, both I_G vs. Q_{tn} and G_0/q_c vs. q_{c1} can be used to identify collapsible soils. Despite the fact that the charts can be used separately, it is strongly suggested to use both I_G vs. Q_{tn} and G_0/q_c vs. q_{c1} for a redundant classification, with the required input data coming from a similar origins test.

The qualitative approach, based on the dataset, indicates that it is possible to use the SCPT to identify collapsible soils in the early stages of the site characterization. It is an interesting approach, since undisturbed samples and laboratory tests are not required. It is an approach to help identify the presence of collapsible soils in the preliminary design phase and to guide, not replace, the appropriate techniques and methods for the characterization and identification of collapsible soils.

5. Conclusions

G_0/q_c vs. q_{c1} and Q_{tn} vs. I_G data points obtained by SCPT, CPT, and seismic tests (down-hole) from 21 sites were collected, reviewed, and interpreted to identify collapsible and non-collapsible soils. Q_{tn} vs. I_G data can be used as a first indicator of the presence of collapsible soils, since all investigated collapsible soils presented microstructure. However, this approach requires careful consideration, as non-collapsible (e.g., Unesp 2, USP 2, and FEUP) soils can also present microstructure. The G_0/q_c ratio decreases when q_{c1} increases, and the opposite is true for non-collapsible soils. Hence, the G_0/q_c vs. q_{c1} chart and the suggested equations were used to distinguish between collapsible and non-collapsible soils. The collapsible soils presented G_0/q_c values greater than 23 and q_{c1} values less than 70, while non-collapsible soils had G_0/q_c values less than 23 and q_{c1} values greater than 70.

SCPT has been widely used for site characterization and is an interesting in-situ test to help identify collapsible soils. It is useful for a qualitative evaluation in the early phase of site investigation, especially when reliable soil samples are difficult to recover. Further SCPT data from different soil types, particularly the collapsible ones, are valuable to adjust or confirm the boundary equations suggested.

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