Estimation of fines content and plasticity index of soils using Screw Driving Sounding

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ABSTRACT: Many soil properties, such as fines content (*FC*) and plasticity index (*PI*), play key role in the design of geotechnical structures. Instead of soil sampling and laboratory testing, *FC* and *PI* are routinely estimated from, say CPT data. Screw Driving Sounding (SDS) is a new in-situ test in which a machine drills a screw point into the ground in several loading steps while the attached rod is continuously rotated. At every rod rotation, a number of parameters, such as torque, load, penetration speed and rod friction, are measured; these provide robust way of characterizing soil layering. It uses a portable device and is cost-efficient, making it ideal for confined areas. In this paper, soil samples from different sites with available SDS data are obtained and the *FC* and *PI* are determined to formulate a relation between the lab-obtained values and the corresponding SDS parameters. The results indicate that the SDS method has great potential in estimating these properties of soils.

Keywords: field testing; screw driving sounding; empirical correlation; fines content; plasticity index

1. Introduction

Information about the foundation ground properties is essential for the analysis and design of engineering structures. For this purpose, the conventional approach in determining the soil properties is to retrieve samples from the site and test them in the laboratory. Such approach allows a more direct inspection and more accurate measurement of the required soil parameters to be made, leading to more accurate characterization. However, since soil sampling can only be done at representative locations and therefore, laboratory testing and the ensuing characterization can be made only at discrete points. Moreover, there is the difficulty and cost associated with obtaining high-quality undisturbed samples, as well as the effect of possible sampling disturbance during sampling, handling and transport. The more preferred approach, especially by the industry, is to estimate the required soil properties through field testing, where a continuous, or nearly continuous, subsurface profile can be achieved, allowing for full characterization of the site to a desired depth. With the development of better field testing techniques and equipment as well as improved understanding of soil behavior, field investigation techniques are now commonly adopted over the traditional methods of sampling and laboratory testing.

A number of field testing techniques are available to characterize sites, such as the standard penetration tests (SPT), cone penetration tests (CPT) and Swedish weight sounding (SWS). These in-situ tests generally apply specific loading patterns to measure soil properties (mostly, in terms of penetration resistance). In recent years, CPT has become popular worldwide because of its ability to provide continuous profile and, although sampling is not possible, soil type (to be more specific, soil behavior type) can be estimated from the collected data. On the other hand, SWS is used very often in Japan to evaluate the strength profiles of soil deposits for residential house construction because it is highly portable and economical and also gives continuous profile [1-2].

An upgraded version of SWS, referred to as Screw Driving Sounding (SDS), has been recently developed in Japan [3]. The machine commonly used in conducting SWS (see Figure 1(a)) has been modified and improved, including the measurement of the friction on the sounding rod [4]. In performing the test, the SDS machine (see Figure 1(b)) drills a rod, with a screw point attached at the tip, into the ground in several loading steps while the rod is continuously rotated; at the same time, a number of parameters, such as torque, load and speed of penetration, are logged at every rotation of the rod. Because these parameters are continuously measured, the soil profile and strength variation throughout the depth of penetration can be obtained.

Because SDS test is relatively new, there is a need to develop empirical relationships for the purpose of soil characterization. For this purpose, the authors performed

(b)

(a)

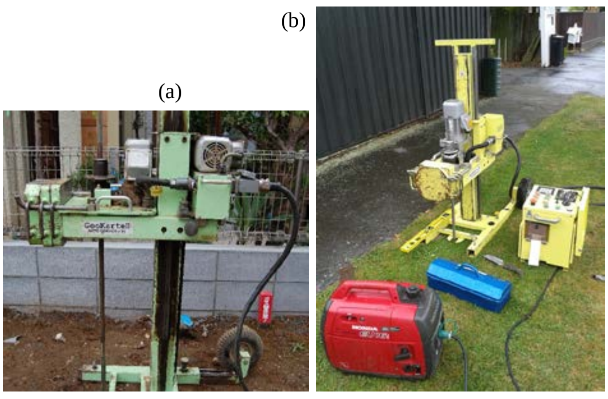


Figure 1. Machines used for (a) SWS test; (and (b) SDS test.

SDS tests at a variety of sites in New Zealand, and they employed the soil database obtained to develop a soil classification chart based on SDS-derived parameters [5]. Moreover, using the data obtained following the 2010–2011 Canterbury Earthquake Sequence, they established a methodology for liquefaction potential evaluation using SDS data [6-7].

In this paper, the results of the SDS tests conducted adjacent to boreholes at different soil types around New Zealand were employed and, together with the laboratory test results on the soil samples obtained from the same sites, correlations were developed for estimating the fines content and plasticity index of the soil directly from the SDS parameters.

2. Principle and Test Procedure

2.1. SDS Test

The SDS set-up consists of a screw point, sounding rods and a machine capable of applying loads and recording measurements. In the test, a static loading system is used and involves applying a load on a rod equipped with a screw-shaped point at the tip. While the rod is being rotated at a constant rate of 25 rpm, static load is applied to the rod at every complete rotation in seven sequential steps: 0.25, 0.38, 0.5, 0.63, 0.75, 0.88, 1kN, i.e. the load is increased at every complete rotation of the rod until a 25 cm penetration is reached. During the test, the following parameters are measured at every complete rotation of the rod: maximum torque ($T_{\max}$), average torque (T_{ave}), minimum torque ($T_{\min}$) on the rod, penetration length (L), penetration velocity (V_{pen}) and number of rotations (N) of the rod. After each 25 cm of penetration, the rod is automatically moved up by 1 cm and then is rotated to measure the rod friction. Then, the measured torque and load during the penetration are corrected to take into account the effects of rod friction; details of the correction procedure are explained by Tanaka et al. [4]. The rod is then moved down 1 cm back to its original position and the process is repeated. The test procedure is schematically illustrated in Figure 2.

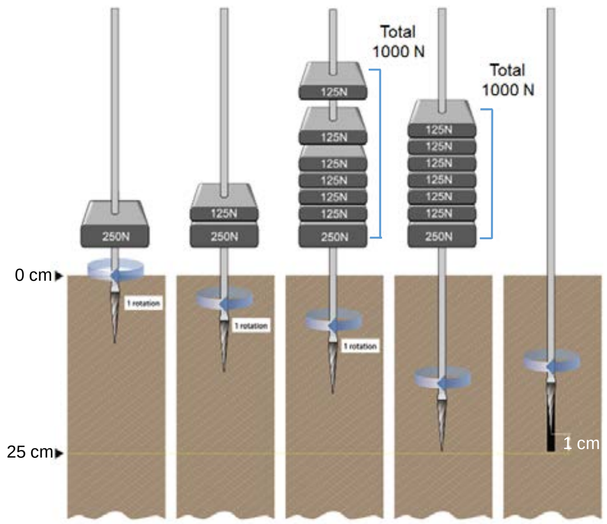


Figure 2. Schematic of the SDS test procedure.

A typical set of SDS test results, as well as the CPT strength profile for a site in Christchurch is shown in Figure 3. In the figure, the torque, T, and load, W, are corrected for the effect of rod friction, while V_{pen} is the penetration velocity. The parameter c_p" is the coefficient of plastic potential while $Ave\delta T$ is the average change in torque for every 25 cm of penetration; these are defined as follows:

$$c_p" = \frac{1}{n}\sum_{i=1}^{n}\left(\frac{N_{SD}D}{\pi T / WD}\right)_i \tag{1}$$

$$Ave\delta T = \frac{1}{n-1}\sum_{i=1}^{n-1}\left(T_{i+1} - T_i\right) \tag{2}$$

In the above equations, N_{SD} is the number of half-turns of the rod per 25 cm penetration, D is the diameter of screw point, and n is the number of loading. The parameters q_c and f_s represent CPT cone tip resistance and sleeve friction, respectively.

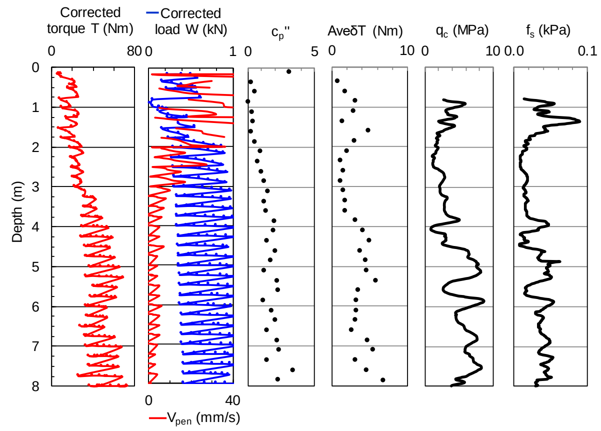


Figure 3. Typical set of SDS results, and their comparison with CPT data, at a site in Christchurch.

Further details of the SDS method have been reported by Suemasa et al. [3] and Tanaka et al. [4], while some results of SDS application in New Zealand have been reported by Mirjafari et al. [8-9] and Orense et al. [10-11]. Some of the developed correlations are discussed in the next section.

2.2. Previously developed correlations

2.2.1. Soil classification chart

Considering the SDS tests performed adjacent to sites where borehole logs are available, attempts were made to determine which SDS parameter(s) would correlate well with the type of soil (determined visually from the borehole logs) found at each depth; hence, it was possible to come up with a soil classification chart based purely on SDS parameters obtained during the tests. For this purpose, various SDS parameters (expressed in terms of measured torque, load, energy, etc.) were investigated to examine which of these best correlate with the appropriate soil types. Based on observation using NZ experience, the parameter c_p" and $Ave\delta T$ correlate well

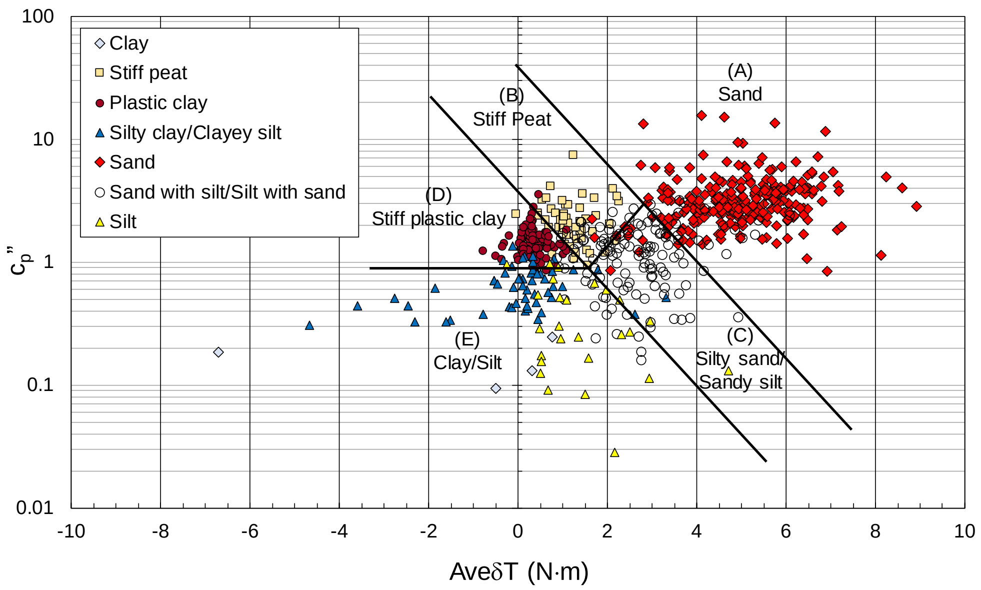


Figure 4. Soil classification chart based on SDS data from NZ database.

with the soil type. The soil classification chart obtained based on the NZ soil database is shown in Figure 4 [11].

Note that in the figure, the boundary lines were drawn visually to separate the data such that points representing similar soil types are grouped together. Data points in region (A) are sandy soils which, because of their frictional nature, are expected to have higher $Ave\delta T$ and c_p" values compared to the other soil types. Based on borehole log analysis, sands on the left part of the region are finer than those on the right part. In addition, as c_p" is an indication of the difficulty in penetration, the upper part of region A would be denser than those on the lower part. Region (B) is for stiff peat, which can be found in South Auckland; peat is considered as c-ϕ soil and it is reasonable that it is positioned to the right side of Regions (D) and (E), both of which represent cohesive soils. Region (C) represents sandy silt and silty sands. Soils at the bottom left of the region contain more silt than sand; therefore, this region can be considered as a transition zone from frictional behaviour to frictionless (cohesive) one. Soils in region (D) are highly-plastic stiff clays which have $Ave\delta T$ values < 1.5 and 0.9 < c_p" < 3. Finally, region (E) where c_p" < 0.9 belongs to clayey silt, silty-clay, silt and clay. Note that the available borehole data for clayey soils in the database were scarce and more tests are planned to separate clay and silt. However, it is expected that the upper part of this region would represent stiff clay or silt while the lower part would be for soft clay.

Note that SDS parameters were used represent the mechanical behavior of the soil and therefore the chart may be better called “soil-behavior chart” (similar to CPT-based SBT chart) even if the correlations between these SDS parameters were made with respect to the “actual” soil type, as reported in boring logs. In any case, soil classification criteria based on grain-size distribution and plasticity often relate reasonably well to in-situ soil behavior and, hence, a good agreement between USCS-based classification and SDS-derived classification is expected.

2.2.2. Correlation with SPT

SPT is a very popular test around the world and geotechnical engineers have gained significant experience in designing geotechnical structures using parameters based on SPT correlations. For the SDS, more development based on local experience and field observation is required to derive geotechnical parameters from the obtained results. Thus, there is a need for reliable SDS-SPT correlation so that SDS data can be used in conjunction with available SPT design correlations.

For this purpose, SDS tests were performed at sites where SPT data were available to compare the SDS output and measured SPT *N*-value. The borehole and SPT data used were obtained from the New Zealand Geotechnical Database (NZGD) [12] and the SDS tests were conducted < 2 m away from the pre-selected sites. It was deemed that only those with clean sand layers would be considered, as the data for other types of soil were very limited. Thus, the soil type was first identified through the borehole log and then comparisons were made between SDS and SPT on a depth-by-depth basis.

Based on the analyses of results, it appeared that the SPT *N*-value correlates well with the energy of penetration. Note that the combined effect of the applied load and torque during SDS test can be expressed in terms of energy, i.e., the incremental work done, δE, by the torque and vertical load for a small rotation can be calculated as [3]:

$$\delta E = \pi T \delta n_{ht} + W \delta s_t \tag{3}$$

where δn_{ht} is the number of incremental half turns and δs_t is the incremental settlement caused at a load step. The average specific energy, E_s, is defined as the average of the penetration energy E for different loading steps, divided by the volume of penetration of the screw point:

$$E_s = \frac{1}{n}\sum_{i=1}^{n}\left(\frac{E}{L\cdot\left(\pi D^2/4\right)}\right)_i \tag{4}$$

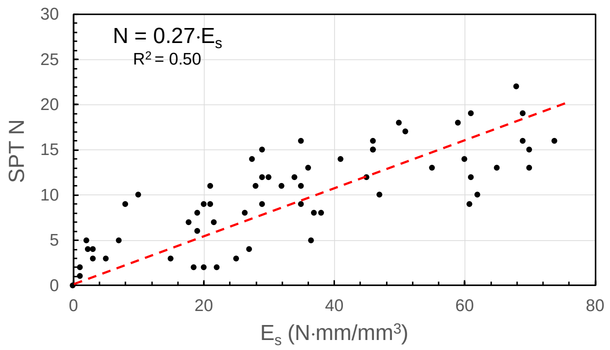


Figure 5. Plot showing the relation between SPT *N*-value and E_s of SDS (clean sand).

Figure 5 illustrates the correlation between SPT N-value and specific energy E_s from SDS for 18 sandy sites in Christchurch [11]. It is observed that while there is scatter in the plots, there is generally a linear relation between SPT *N*-value and specific energy, E_s. As the SPT *N*-value increases, the soil becomes denser and the energy required to penetrate the screw point into the soil increases.

The sources of the scatter in the data points can be traced to the difference in penetration mechanism and the depth of recording points. Obviously, the penetration in SPT is dynamic, while in SDS, it is static. Also, while SDS recorded the parameters continuously along the depth (and E_s was averaged every 25 cm of penetration), the SPT *N*-value was measured every 1 or 1.5 m. In dense to medium dense sands, E_s may represent the properties of soil for a layer with thickness of even less than 25 cm (due to difficulty in penetration); at some depths, the average of two successive 25 cm penetration was used in SDS to compare with the SPT *N*-value, and some outliers belonging to transition layers, identified visually by observing the trends of the two tests, were removed from the plot.

As mentioned, this correlation is only applicable to clean sands and more data is required to extend this relation to other types of soils. In addition, the maximum load and torque that the SDS machine can apply are limited; thus, in stiff soils, the machine cannot penetrate and the rod just simply rotates without significant penetration and therefore the measured energy does not represent the actual stiffness of soil. Generally, based on experience, SDS can penetrate in soil layers with SPT *N*-value < 20.

2.2.3. Correlation with CPT

In order to develop correlation between CPT and SDS parameters, SDS tests were conducted < 2 m from sites with available CPT data. As in establishing the SPT correlation, the SDS parameters and the CPT cone tip resistance, q_c, at similar depths were compared. Among the SDS parameters, the specific energy, E_s, best correlated with q_c.

By comparing the SDS and CPT data for 60 sites, it was found that the correlation between the outputs from these two tests is highly dependent on the fines content and the stiffness of the soil (c_p" value). The fines content were estimated from the SDS data using the previously developed SDS-based chart [7], with three levels of *FC* considered: low, medium and high. For soils with high fines content, *FC*, the correlation was different for stiff plastic soils and soft plastic soils. As shown in the soil classification chart in Figure 9, c_p"=0.9 is the threshold

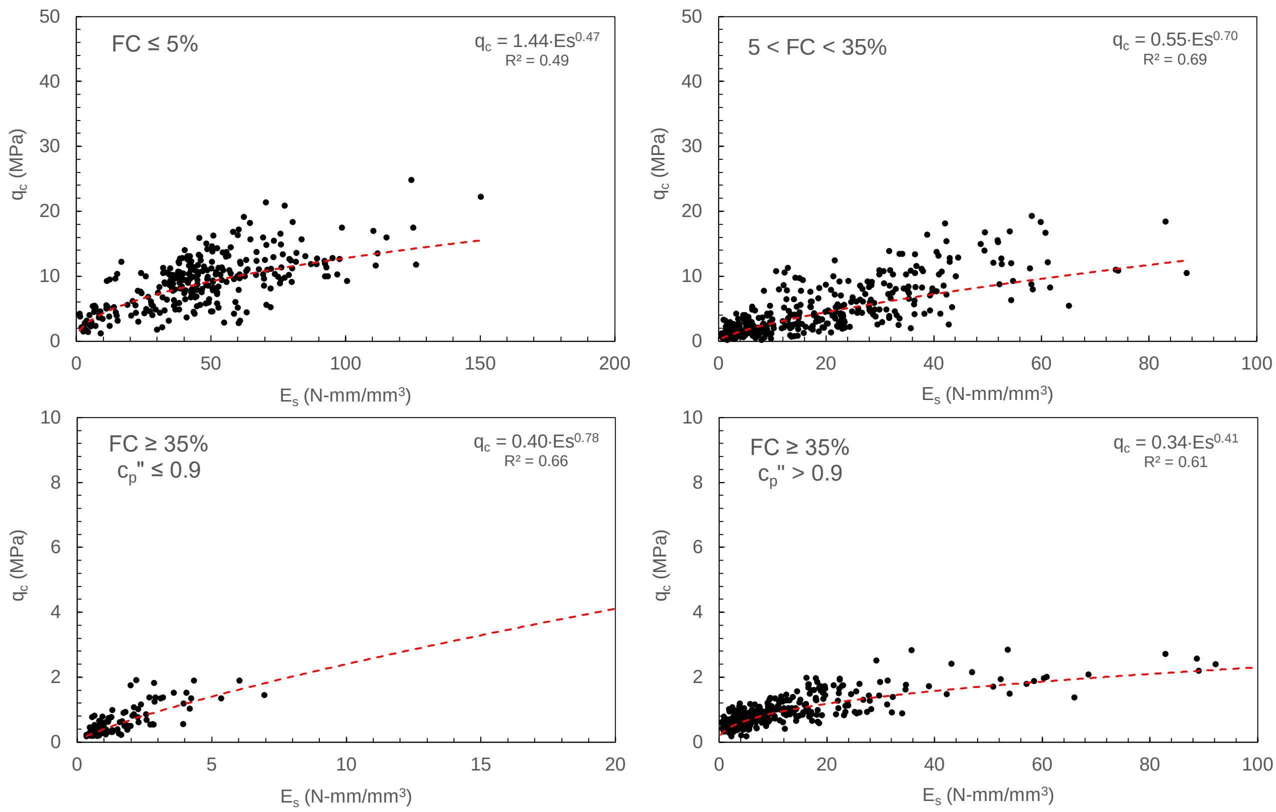


Figure 6. Plots showing the relation between SDS E_s and CPT q_c.

separating stiff and soft soils. Thus, for the purpose of developing the correlation, the tested soils were divided into 4 groups as follows: (a) $FC \leq 5\%$; (b) $5\% < FC < 35\%$; (c) $FC \geq 35\%$ and $c_p'' > 0.9$; and (d) $FC \geq 35\%$ and $c_p'' < 0.9$.

Figure 6 illustrates the relationships between E_s and q_c for different fines content [13]. Again, it is observed that as the cone penetration resistance increases, the specific energy required to penetrate the screw point in SDS increases. In denser soils with high q_c values, the SDS machine cannot penetrate the screw point into the ground and the rod just simply rotates without significant penetration, leading to higher E_s. Based on experience, SDS can generally penetrate in soil layers with q_c values < 15 MPa.

Again, data scatter is seen in the plots, due to the difference in penetration mechanism (static penetration for CPT and static + rotational penetration in SDS) and the uncertainty associated with depth measurements (CPT was measured every 20 cm while SDS was recorded every 25 cm). While care was taken when selecting data for analyses, zones in the transition between layers were problematic. Also, many of the CPT and SDS tests were performed with a time gap between them and it is possible that soil conditions, such as water table, may have changed during this gap which could have affected the penetration resistance of soil.

Attempts were also made to correlate SDS parameters with the CPT sleeve friction, f_s. In order to facilitate the correlation, f_s was combined with q_c in terms of the parameter P:

$$P = \frac{\log q_c}{\log F_r + 10} \tag{5}$$

where $F_r = (f_s/(q_c - \sigma_{v0})) \times 100\%$ and σ_{v0} is the total vertical pressure [14]. By analysing the SDS results obtained, it was determined that P correlates well with the corrected torque, T. A typical depth profile comparison between P and T is given in Figure 7, where a good agreement is obtained.

The correlation between P and the average torque, T_{ave}, is shown in Figure 8. It can be seen from the figure that with the relatively high value of the coefficient of determination, R^2, there is good correlation between the

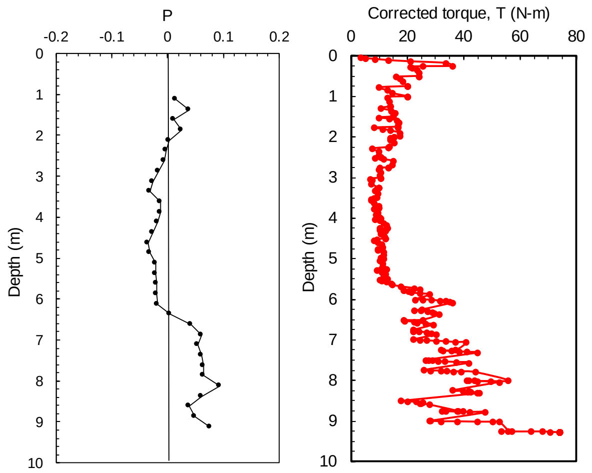


Figure 7. Typical variation with depth of: (a) P from CPT test; and (b) Corrected T from SDS test.

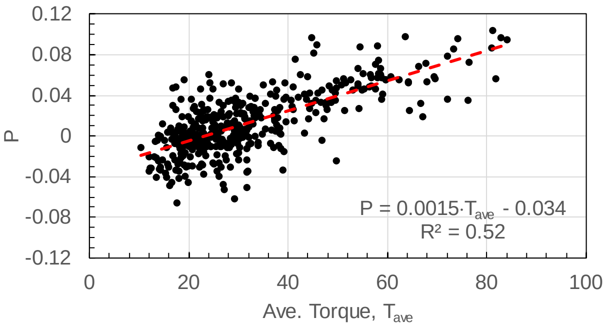


Figure 8. Correlation between P and T_{ave}.

two parameters. Thus, f_s at any given point can be estimated from T_{ave} and P correlation, and then through Eq. (5).

Finally, SDS parameters were correlated with the soil behavior type index, I_c, which is essentially the radius of the concentric circles delineating boundaries between soil types in the soil behavior chart [15]. Figure 9 shows the correlation between I_c and $Ave\delta T$; again, good correlation can be observed.

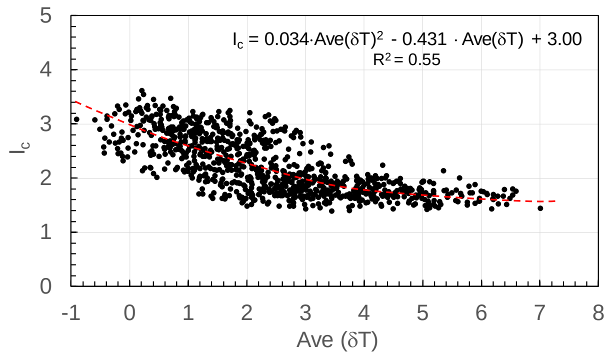


Figure 9. Correlation between I_c and $Ave\delta T$.

2.2.4. Correlation with shear strength parameters

In order to estimate the shear strength parameters of the soil directly from SDS parameters, undisturbed soil samples of cohesive clays and plastic silts were obtained using push tubes at fine-grained sites in Auckland where SDS tests have been performed; for SDS sites in Christchurch, disturbed sand samples were obtained by block sampling. For each sample, a series of monotonic triaxial compression tests were performed, i.e. for sandy soils taken from Christchurch which were mostly clean sands, consolidated drained (CD) triaxial compression tests were conducted on reconstituted samples taken from boreholes adjacent to SDS test locations, with their in-situ density replicated based on estimates from the CPT cone resistance. On the other hand, for the clayey samples taken from SDS sites in Auckland, several series of consolidated undrained (CU) triaxial tests and unconsolidated undrained (UU) triaxial tests were performed on undisturbed soil samples. The shear strength parameters obtained were then correlated to the SDS parameters recorded at the appropriate depths were the samples were obtained [16].

By analyzing different SDS parameters, it was found that the average torque, T_{ave}, which represents the average torque every 25 cm depth of penetration considering different loading steps, has the best correlation with the peak effective friction angle, ϕ'. Figure 10 shows the relationship between T_{ave} and ϕ', where the best fit line is also indicated. From the figure, increasing the friction angle would result in higher T (and vice versa). In general, the peak friction angle increases as the relative density of the soil increases (i.e. denser soil shows higher friction angle); hence, it is reasonable to expect a good correlation between the peak friction angle and the amount of torque required to penetrate the layer during the SDS test.

The scatter in the plot can be due to the effect of fines content, *FC*. It is well known that the amount of fines affect the friction angle of soils and the proposed relationship can be considered as a general trend representing samples with different fines contents. Thus, to obtain a better correlation that incorporates the effect of *FC*, more tests need to be performed which would enable to have different correlations for several ranges of *FC*.

Based on the results of CU and UU tests, a correlation was developed between the undrained strength, C_u, and the average torque from SDS test, and this is shown in Figure 11, together with the best fit line. From the figure, there appears to be a good correlation between the undrained shear strength of soil and the average torque from the SDS test. Similar to the trend observed for friction angle, the required torque to penetrate a strong soil layer (i.e. one with high undrained shear strength) is also high. Note that the point at $T_{ave} \approx 30$ N·m has significant effect on the observed trend. Hence, the proposed relationship can be further improved by performing more tests to increase the number of data points.

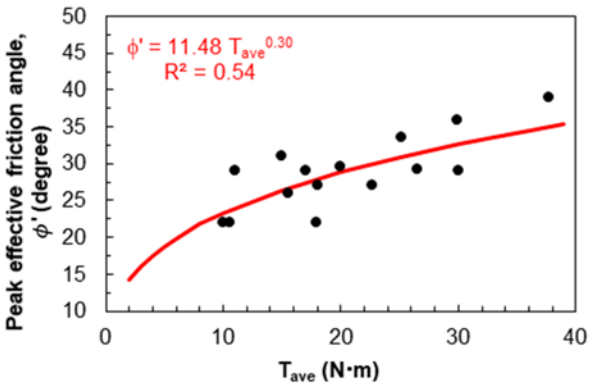


Figure 10. Relationship between average torque from SDS and peak friction angle.

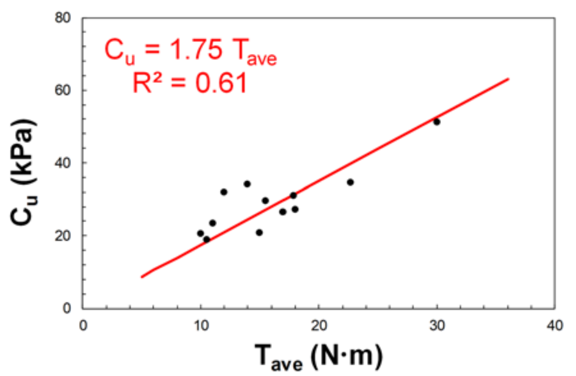


Figure 11. Relationship between average torque from SDS and undrained shear strength.

3. Recently developed correlations

Next, correlations between basic index properties of soils, such as fines content (*FC*) and plasticity index (*PI*), which play significant role in the design of geotechnical structures are performed. In routine field testing, these are estimated from field-obtained parameters, say the CPT data. Hence, it is considered important to develop correlations between SDS-derived parameters and these index properties in order to improve the use of SDS method.

3.1. Sources of samples and laboratory tests

In an earlier study, Mirjafari t al. [17] attempted to develop correlation between *FC* and SDS parameters using disturbed samples from 6 different sites in Christchurch and sieve analysis was performed on 115 samples. The particle size distribution for the samples was obtained by the method of wet sieving described by NZS 4402.2.8 [18] and fines content (*FC*) was defined as the percentage by weight passing through a 63 μm sieve. While a better than average correlation was obtained, the database used had predominantly sandy soil (*FC*<15%) and clayey soil (*FC*>95%) and data in-between was sparse.

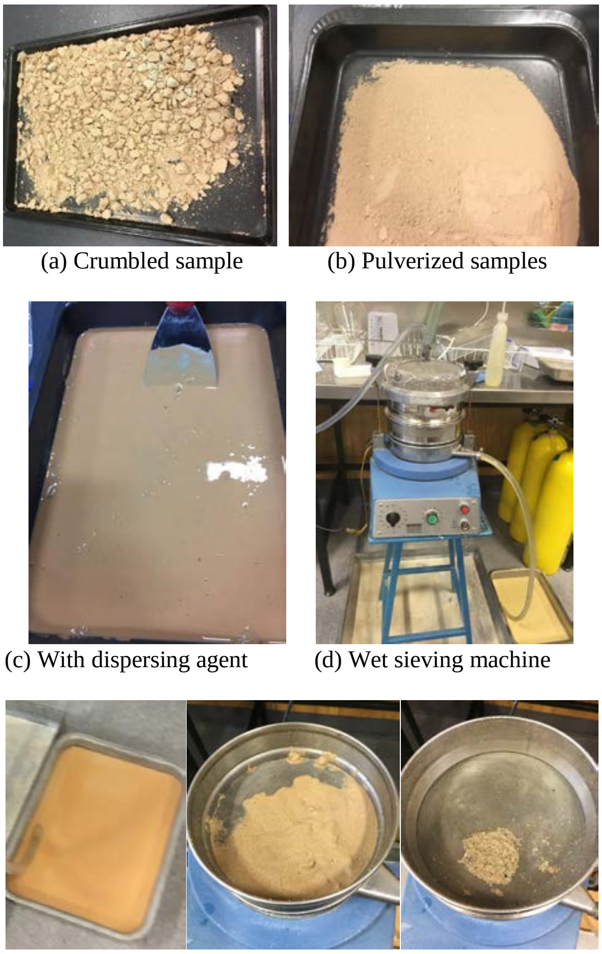
For this purpose, some of the undisturbed soils samples obtained using push tubes at sites in Auckland where SDS tests have been performed were used; these samples were utilized in determining the shear strength parameters through several series of consolidated undrained (CU) triaxial tests and unconsolidated undrained (UU)

Figure 12. Some of the samples used in the index property determination.

triaxial tests [16]. Some of these samples are shown in Figure 12.

3.1.1. Fines content determination

As mentioned earlier, the fines content was determined following the guidelines of the New Zealand Standard for the particle size distribution using wet sieving [18]. Samples of about 10 cm high were cut from samples extracted from the tube at pre-determined depth (corresponding to the location where SDS test was conducted). The samples were crumbled and allowed to air dry for 1 to 2 days. Then, the material was further pulverized using a suitable pestle and mortar in a way that it did not crush the particles but simply break down the larger crumb. A solution of 0.2% Sodium hexametaphosphate was used as dispersing agent and the mixture was stirred frequently for a period of at least 1 hour. Then, the material was washed through two sets of sieves, a 600 µm one nested in a 63 µm sieve, with the wash water containing all the fines (silts and clays under 63 µm) collected. Washing was continued until the sieves were clean and the water passing the 63 µm sieve was visually clear. The material retained in the sieves was carefully collected in trays to oven-dry and to determine the weight retained and the fines content calculated. These procedures are summarized in Figure 13. In total, 30 samples were tested for their *FC*, and these were added to the original 115



(a) Crumbled sample (b) Pulverized samples

(c) With dispersing agent (d) Wet sieving machine

(e) Oven-dried samples in each sieve

Figure 13. Determination of fines content

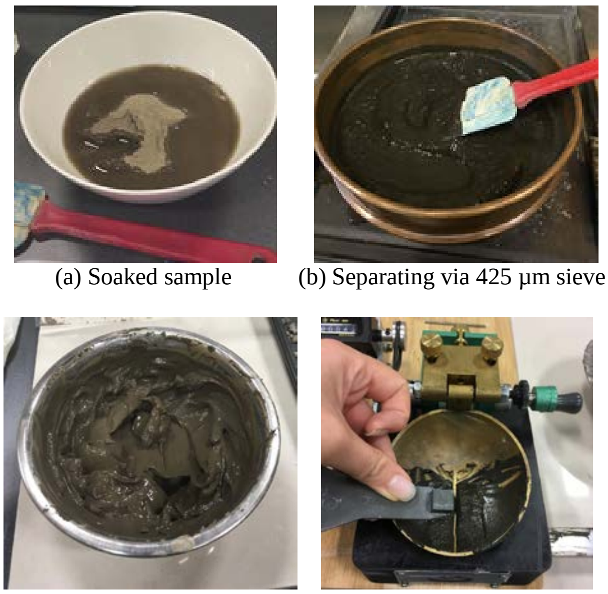
samples, and their correlation with the appropriate SDS parameter was examined.

3.1.2. Plasticity index determination

The plastic index was computed following the guidelines of the New Zealand Standard [19-21].

For the plastic limit test, samples of about 10 cm height were crumbled and pulverized, following the same method as in the previous section. After soaking the powderized material in water for about 2 hours, the wet soil was run through the 425 µm sieve until most of the material passed. The resulting mixture was very smooth and workable. The sample was covered and left overnight at constant temperature. The following day, the sample was remixed for between 5 to 10 minutes after which a subsample was taken and spread into the cup of the liquid limit device. A groove was created with a single stroke using the standard grooving tool. The test was performed by rotating the handle of the liquid limit device at 2 rev/s and the number of counts necessary to cause two parts of the soil to come in contact within a length of 13 mm was recorded. The procedure was repeated 3 consecutive times for the same water content, keeping the difference between two consecutive blows $\leq$ 3. Then, a sub-sample of the specimen was taken from the cup to compute the corresponding water content. The procedure is summarized in Figure 14. At least 5 different measurements were taken per sample with different water contents, trying to have at least 2 measurements under 25 blows and 2 above 25 blows. The liquid limit, *LL*, is considered as the water content of the specimen when it requires 25 blows to cause the two parts of the soil to come in contact at a length of 13 mm. This is done by plotting the water content a ordinate against the corresponding number of blows in a logarithmic scale (known as the flow curve) and fitting a straight line.

To compute the plastic limit, a sub-sample (about 30 grams) of the same liquid limit specimen was continued



(a) Soaked sample (b) Separating via 425 µm sieve

(c) Resulting soil mixture (d) Measuring the contact length

Figure 14. Determination of liquid limit

to be air-dried by mixing them in a mixing plate, until the material became sufficiently plastic to be shaped in a ball. Then, the ball was rolled between the palms until slight cracks appeared on the surface. The sample was then divided in two halves, which were divided into 4 sub-samples. Each sub-sample was rolled between the palm of the hand and the glass plate until it reached about 3 mm diameter, then it was molded between the fingers to further dry the rolls. The plastic limit, *PL*, is the water content that the sample has when it breaks while being rolled to 3 mm diameter and it shears in both longitudinal and transverse directions. In total, two measurements of plastic limit were taken per sample, each including the 4 sub-samples in which the initial ball was divided. Then, the plastic limit was computed as the average of the two measurements. The procedure for determining *PL* is summarized in Figure 15.

Finally, the Plastic Index, *PI*, was computed as the difference between *LL* and *PL*. In total, 30 samples were tested for their *PI* and their correlation with the appropriate SDS parameter (taken at the same depth as the sample) was evaluated.

(a) Sample rolled in a ball

(b) Division into sub-samples

(c) Rolled thread of soil

Figure 15. Determination of liquid limit

3.2. Correlation between FC and SDS

Various SDS parameters corresponding to the depth where samples were tested (sieve analysis) were analysed to check which ones best correlate with the fines content, *FC*. From the analysis, it appears that both $Ave\delta T$ and c_p'' correlate well with *FC*. Looking at the classification chart shown in Figure 4, clayey soils plot on the lower left part of the chart, while sandy soils plot on the upper right part. Using multiple linear regression, the correlation between *FC* and the two SDS parameters is given by:

$$FC = 0.28Ave\delta T^3 - 0.17c_p''^3 - 1.75Ave\delta T^2 + 3.77c_p''^2 - 11.97Ave\delta T - 22.18c_p'' + 100.4 \quad (6)$$

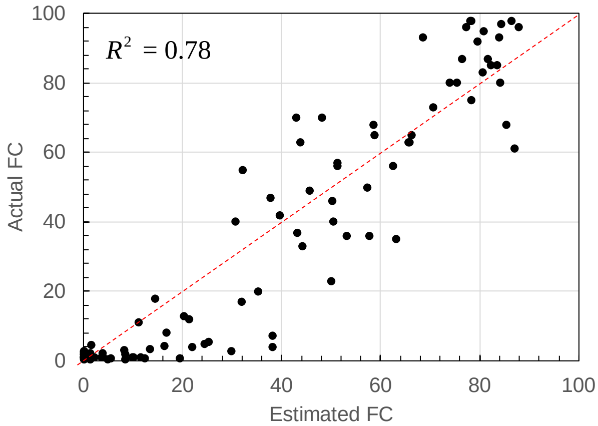


Figure 16. Comparison between actual and estimated *FC*.

A comparison between actual *FC* (from laboratory tests) and estimated *FC* (using Eq. 6) is shown in Figure 16. It can be observed that there is good agreement between the two values, with R^2=0.78.

3.3. Correlation between PI and SDS

Similarly, a correlation was attempted between the SDS parameters monitored at the sampling depth and the corresponding *PI* obtained in the laboratory for the sample. Only plastic samples were considered (i.e. $Ave\delta T$ < 1). Preliminary analysis using Artificial Neural Network (ANN) indicated that T_{ave} has the highest correlation with *PI* (importance level = 0.49) while c_p'' and V_{pen} were next (importance level = 0.21). note that in ANN, the "relative importance" (or strength of association) of a specific explanatory variable for a specific response variable is determined by identifying all weighted connections between the nodes of interest. That is, all weights connecting the specific input node that pass through the hidden layer to the specific response variable are identified, tallied and scaled relative to all other inputs. Hence, the "importance level" of a certain explanatory variable is indicative of its relation with the response variable (i.e. higher importance means stronger relationship, whether negative or positive, while relative importance close to zero indicates no substantial importance) when compared with the other variables being considered. In essence, "importance level" provides similar information as the correlation coefficient *R* (which is an indication of the true relationship between the input and output variables).

Figure 17 plots the relation between *PI* and T_{ave}, where it can be seen that, while the scatter is significant (with R^2=0.20), the value of *PI* generally decreases as T_{ave} increases. Further investigation is planned to clarify the best set of SDS parameters that can be used to estimate the *PI* of soils.

4. Concluding remarks

Instead of obtaining soil samples from the target sites and performing laboratory tests, it was proposed to estimate the fines content, *FC*, and plasticity index, *PI*, directly from the parameters obtained from Screw Driving Sounding (SDS), a

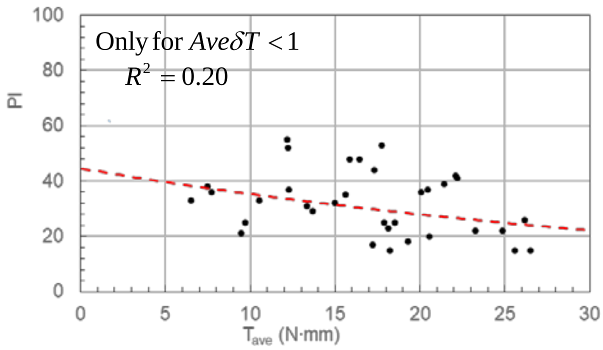


Figure 17. Relation between *PI* and T_{ave}.

recently developed in-situ test in which a machine drills continuously a screw point into the ground under several loading steps. For this purpose, samples were obtained from sites where SDS tests have been performed and *FC* and *PI* were determined using laboratory-based methods. These were then correlated with the SDS results, where analyses indicated that the $Ave\delta T$ and c_p" best correlated with *FC* while T_{ave} appeared to correlate with *PI*. While the number of soil samples tested was limited, good correlations were obtained, indicating that SDS test can be used to estimate these index properties and, in addition to the other empirical correlations already proposed, could provide a robust way of characterizing soil stratigraphy.

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