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Site Characterization of Alluvial Silty Sand Soils by Dynamic In-Situ and Laboratory Tests

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Abstract: Soil is normally heterogeneous and non-uniform material since it is made up of diverse types of matter. Understanding of soil parameters is one of the foremost complex assignments in geotechnical design. A few laboratory tests of soil need to carry out for engineering practices. Research facility tests take too much time to achieve and are excessive moreover. There's another elective way to urge soil parameters direct from the soil field investigation report to spare time and fetched. The objective of this consideration is to explore the relationships of soil parameters with the most commonly utilized soil investigating apparatuses SPT and CPT and other soil parameters that relate to direct applications in designing practice. In this investigate, four sets of in-situ tests were created in order to study the appropriateness of different existing CPT-SPT relationships to the alluvial silty sand soils. The recouped tests from SPT tests contain a calculable sum of fines (f_c) extending from 3.5 to 39.2 percent and mean grain measure (D_{50}) extending from 0.09 to 0.23 mm. Ponder comes about demonstrated that q_c/N ratio as a function of D_{50} and fc-based relationships are ineffectively pertinent to the silty sand soils. This paper recommends that $q/(N_1)_{60}$ ratio as a function of increasing steadily can be way better related in silty sand soils rather than research facility-based D_{50} or f_c relationships. An exertion has moreover been made to perform a comparative investigation for the compelling point of inside contact gotten from CPT, SPT, direct shear tests, and triaxial tests.

Keywords: CPT, SPT, triaxial tests, silty sand soils, direct shear test, site characterization

1. Introduction

The cone penetration test, CPT, has been utilized for numerous a long time as a standard investigation tool, mainly to assess rapidly the soil profile as well as for the estimation of the undrained shear strength in the case of cohesive soils. In spite of the fact that the CPT is picking up popularity, the older in-situ device is known as the standard penetration test, SPT, is still utilized in nearly all geotechnical investigations, during testing boring. Thus, correlations between SPT and CPT are of commonsense interest, considering the extraordinary number of existing information gotten with SPT.

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Fig. 1 - Loation of study area in the Tehran provine and Iran: (a) and; (b) Chitgar Lake in Tehran, Iran

Due to the lack of soil testing amid CPT, in numerous designing projects, it is most common to utilize both CPT and SPT together for subsoil investigation and a combination of gotten soil parameters from these tests are being utilized for design purpose. From that point of view, numerous experimental correlations have been built up between CPT cone tip resistance (q), SPT N-value, and other building soil properties that relate to direct applications in designing practice. Most of the currently accessible relationships apply as it were to perfect soils.

In some cases where the soils are not perfect, the regular relationships are lacking or lead to inconsistent conclusions. In spite of the fact that designers utilize these relationships in practice, it is fundamental to look at the appropriateness of different existing relationships prior to direct application in silty sand soils. The essential objective of this investigation is to look at the appropriateness of different existing CPT-SPT relationships for silty sand soils and to propose conceivable new relationships. The consideration included field investigations, laboratory testing, and a comparative study of the resulting information. For this ponder purpose, four sets of high-quality CPT and SPT tests were carried out up to 25 m at the western shore of the lake in Tehran (Chitgar Lake) as appeared in Fig. 1. In spite of the fact that usually a small-scale inquire about work, it is trusted that this work will give valuable commitment for advance enhancement of locale particular CPT-SPT relationships.

2. Background on SPT-CPT Correlation

In the last few decades, much investigation has taken put to appropriately utilize copious encounters and accessible databases on SPT for a more solid CPT. As a result, a significant number of relationships have been proposed by a few analysts between CPT cone tip resistance (q), SPT N-values, and other designing soil properties. These relationships can be considered in three major groups. Most of the essential relationships considered qc/N as a function of grain characteristics, such as cruel grain estimate (D_{50}) and/or fines substance (f_c). A few other analysts proposed a consistent esteem of q/N for diverse soil sorts in a chart of redressed cone tip resistance (q) versus friction ratio (R_f).

Compiling a number of studies, Robertson et.al (1983) collected a few investigate outputs and presented a relationship of q/N_{60} as a function of cruel grain size (D_{50}). Comparative effort has been made by Been et al. (1985) and, Kulhawy and Mayne (1993) where they have presented more upgraded databases to relate q/N as a work of cruel grain measure (D_{50}) compared to the proposed relationship by Robertson et.al (1983). These relationships give a really valuable rule to change over the CPT tip resistance to the proportionate SPT N-value for soils with D_{50} changed between 0.001 mm to 10 mm. From these relationships, it is observed that the q/N ratio increments with expanding cruel grain size. They have illustrate that the scatter in q/N ratio shows up to extend with expanding cruel grain size. Unfortunately, correction components for q or N-value have not been connected to most of their unique information. It should be illustrate that energy estimations on SPT information demonstrate that the normal energy ratio of approximately 55 to 60 % may represent the normal energy level related to the q_c/N relationship (Robertson et.al (1983)). Chen et al. (2016) recommended an agent esteem of q/N60 proportion of 4.5-5.0 and 4.0 for medium and silty soils, individually.

In other studies, Chin et al. (1988) have displayed relationship between q/N as a function of rate fines (littler than 0.074 mm). In their think measured N-values were adjusted compared to 55 % of the greatest energy. It appeared that, for sands, qc/N decreases significantly with expanding fine substance. In expansion to this information, Duan et al. (2018) summarized a few investigate comes about (Duan et al. (2021), Jefferies et al. (1993), Lai et al. (1985)) which show a comparative slant between qc/N and fines substance as proposed by Chin et al. (1988). RamazanBorjerdi et al. (2019) have compared compelling points of inside contact (∅) evaluated from the CPT with those decided from SPT N-values and laboratory triaxial tests. Their study revealed that effective angle of internal friction (∅) obtained from triaxial tests correlated well with those obtained from the CPT and SPT below the water table, but above the ground water obtained ∅ values from the CPT and SPT were significantly high compared to laboratory measurement. Investigate by RamazanBorjerdi et al. (2018) proposed that, within the nonattendance of location particular CPT-SPT relationships, it is appropriate to utilize the common relationship proposed by Robertson, et al (1983). Within the nonattendance of grain measure information, they have too proposed a modern relationship utilizing the q/ (N1)60 ratio of 0.45. However, a few of the existing relationships give the best-evaluated relationship between qc and N-value, but the locale particular study result presented by Jefferies et al. (1993) represents the very inapplicability of existing relationships. In this manner, coordinated application of the normal relationship presented in a few relationships may lead to critical deviation from the correct result.

3. Soil Conditions and Field Testing

Field investigations include results from standard penetration tests (SPT), and cone penetration tests (CPT) as shown in Fig. 2. A add up to of four sets of CPT and SPT were performed in several areas inside the considered region and each match of CPT and SPT was carried out as near as conceivable, the greatest flat separate was not more prominent than 10 m. In this study, field investigations were carried out along Chitgar Lake in Tehran, Iran. The geologic formations of the research location are basically comprised of alluvial sand and silt stores. The alluvial sands are light to brown-grey colored, coarse to fine silty sand of subrounded in shape. The sand contains generally quartz, feldspar, mica, and the critical sum of heavy minerals. The alluvial silts have the same color as the sands, but are fine sandy to clayey silt and are ineffectively stratified (RamazanBorjerdi et al. (2021), Lai et al. (1985), Elbanna et al. (2011), Mola-Abasi et al. (2019), and Prakash et al. (1992), Shuttle et al. (2007)).

a

b

Fig. 2 - (a) Test Soil bearing capacity Multifunction Standard Penetration Test and Dynamic Cone Penetrometer and; (b) CPT Apparatus

3.1 Cone Penetration Test (CPT)

CPT soundings were progressed employing a Multifunction Standard Penetration Test and Dynamic Cone Penetrometer with a cross-sectional region of 10 cm2 and which can measure the pore water weight (u_2), as well as the cone tip resistance (q) and sleeve friction (f_s). During the progression, measurements of dynamic pore water pressure, tip resistance, and sleeve grinding were recorded persistently at 10 mm profundity increases. The ordinary penetration depth for this study was approximately 25 m underneath from the ground surface.

3.2 Standard Penetration Test (SPT)

SPT was conducted concurring to ASTM D1586 (2002). Boreholes for the SPT were progressed by percussion strategy with Bentonite clay. The split spoon testing strategy was utilized to get soil tests from boreholes and exasperating agent tests were collected. Tests recouped from boreholes were stored in plastic packs that were utilized for laboratory testing. A programmed type SPT hammer-release was utilized for the SPT. A potential source of instability that will affect SPT N-values has been carefully taken into account. Borehole boring, soil testing, and SPT N-value recording strategies were observed by experienced geologists during the whole test program, and these individuals are given visual depictions of the collected tests. The SPT N-value and tests were collected for each 1.52 m interims. The rectified cone tip resistance (q) in MPa (top scale), $(N_1)_{60}$ values (bottom scale) are presented in Fig. 3.

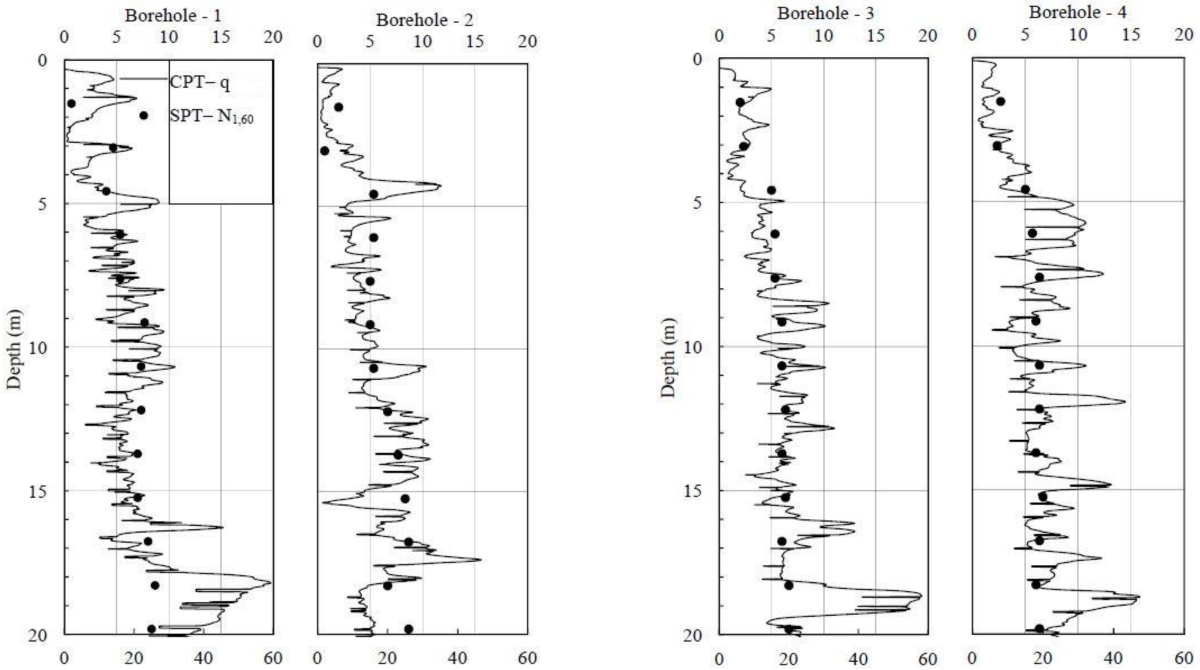

Fig. 3 - Penetration depth versus cone tip resistance (q) and normalized (N1)60 values for all boreholes

Based on the results of the subsurface investigations, the subsoil profile at the considered region can be separated into two strata. The soil inside the test region is essentially comprised of clay, silt, and fine and medium sand particles. As such, the combination of fines and sand is generally non-plastic and the combination of fines and clay is low-plastic. The top layer comprises of silt and clay blend with a total thickness around changed from 3 m to 7.3 m. Instantly below this layer, a combination of fine to medium sand and a few residues layer expanded to a profundity of 25 m. Soil layers are moist from the ground surface to the groundwater table and the ground water table is located 3 m below from ground surface. Note that, there was significant changeability within the measured SPT N-value in completely different boreholes at distinctive depths extending from 1 to 52 and maximum cone tip resistance (q) was near to 20MPa. The consistency of the soils at different depths varies from solid to loose.

4. Laboratory Investigations

Laboratory examinations included basically visual perceptions and mechanical sieve analysis. Soil tests recuperated from different depths were separately evaluated and classified based on sieve analysis. Soil tests gotten from 1.52 m and those containing significant amount clay were excluded from strainer analysis. From strainer examination, it was observed that soils containing calculable amount of fines (material finer than 0.075 mm) ranging from 3.5 to 39.2 percent, fineness modulus (F.M.) varied from 0.18 to 1.11, and mean grain size (D_{50}) within the range of 0.09 to 0.23 mm. According to the sieve analysis results, the soils were by and large classified into two bunches; either well-evaluated sands with small silt or poorly evaluated sands with silt. Concurring with the unified soil classification framework, the soils can be symbolized as SW and SP-SM respectively. As were fine content for each borehole is displayed in Table 1.

Table 1 - Sieve analysis results

Depth (m)	Borehole-01	Borehole-02	Borehole-03	Borehole-04
3.05	26.8	Not Determined	17.96	Not Determined
4.57	32.5	32.44	16.89	Not Determined
6.10	32.5	26.39	19.8	Not Determined
7.62	38.4	36.35	20.7	5.85
9.14	11.3	39.2	16.95	6.25
10.67	12.4	38.1	17.8	5.5
12.19	8	38	7.44	5.5
13.72	17.3	22	5.5	5.39
15.24	12.5	24.4	4.15	5.4
1.76	7.68	5.85	5.05	5.15
18.28	32.03	7.5	4.95	6.75
19.81	22.7	6.85	3.5	6

4.1 Preparation of Reconstituted Laboratory Sample

To obtain an adequate amount of soils for direct shear and triaxial tests, tests with comparable grain size distribution curve, mean grain measure (D_{50}), and fines content (f_c) were chosen to be blended together to plan reconstituted tests for direct shear and triaxial tests. Blending was carried out by carefully stirring little parcels of the chosen tests together until all soil was blended homogeneously. After blending the test was put away in an air-tight plastic box and dealt with carefully to maintain a strategic distance from any significant misfortune of test.

4.2 Direct Shear Test

A total of 36 direct shears (DS) tests, with three tests at each stress level, were carried out on reconstituted examples at diverse relative densities changing from 42% to 70% and diverse compelling stresses changing from 25 kPa to 200 kPa. The chosen test stresses and relative densities were considered comparable to field conditions at depths of 6 m, 12 m, and 18 m. To begin with, the number of dry tests required to get ready the examples at the wanted thickness were calculated. After taking the required sum of dry test in a holder, a measured sum of de-aired water was included in the soil to bring the soil moisture substance to around 10 percent. Examples were arranged in a shear form with an interior diameter of 63.5 mm and stature of 25.4 mm by hand tamping strategy. All tests were performed on soaked tests. Amid the test, vertical relocation gage, shear load gage, and even uprooting gage perusing were recorded until the level shear stack peaks and after that falls. The point of internal contact was calculated in a plot of the most extreme shear stretch versus comparing normal stress.

4.3 Triaxial Test

A total of 36 consolidated undrained triaxial tests (TX), with three tests at each stress level, were carried out on reconstituted examples. The effective stresses and relative densities for triaxial tests were the same as the direct shear tests. Cylindrical soil examples of 142 mm height and 71 mm diameter were utilized and arranged utilizing the wet tamping method. The examples were, to begin with, saturated with back-pressure saturation until the pore weight parameter comes to a value rise to 0.90. Following saturation, the examples were at that point isotropically consolidated at the desired successful stress. To cause shear disappointment within the example, the deviator stretch was connected to the example until the most extreme pivotal deformation comes to value of 15%. The deviator stress was connected at a rate of 0.08 percent/min. During the test, deviator stress, shear stress, typical stress and pore water weight were recorded at every 10 sec interim. The angle of internal contact was calculated by plotting effective-stress Mohr's circles for different tests (three tests at each depth) and drawing a common digression to these Mohr's circles passing through the root. The effective point of internal friction gotten from direct shear and triaxial tests are given in Table 2. The cyclic triaxial tests were conducted in accordance with ASTM D5311 (2002) using the automatic triaxial test, as shown in Fig. 4.

a b

Fig. 4 - (a) cyclic triaxial apparatus used in the experimental program; (b) prepared specimen

Table 2 - Effective angle of internal friction calculated from direct shear and triaxial tests

Depth	**Direct Shear Tests**				**Triaxial Tests**			
	Borehole-01	**Borehole-02**	**Borehole-03**	**Borehole-04**	**Borehole-01**	**Borehole-02**	**Borehole-03**	**Borehole-04**
m	∅	∅	∅	∅	∅	∅	∅	∅
6	31.8	28.81	28.81	31.38	32.5	31.8	31.5	33.5
12	33.82	32.21	34.22	33.02	37	35	35.5	35
18	40.03	34.21	36.87	35	38.5	37	36.5	36

5. Results and Discussions

Based on the most commonly utilized CPT, SPT, and soil parameter relationships, a few calculated and connected soil parameters were determined as a portion of the information interpretation and to check the appropriateness of existing relationships to the silty sand soils. A total of 51 information focuses on the sand stores from the 4 boreholes presented in Table 1 were chosen for this study to perform a comparative investigation with mean grain size (D_{50}) and percent better (f_c) based on CPT-SPT relationships. Calibrations were performed on recorded cone tip resistance information to kill pore weight impact on tip resistance by a calibration factor of 0.32, as given by the cone manufacturer. Moreover, the normalized cone tip resistance (q) was calculated for compelling overburden stress levels as proposed by Juang et al. (2012) and Seed et al. (1985)The SPT vitality corrections and overburden weight corrections were connected to the recorded field N-values to calculate $(N_1)_{60}$. The accepted vitality level was 60 % for vitality correction and soil unit weight was calculated from the SPTN-value versus soil unit weight relationship proposed by Shuttle et al. (2007) to normalize N-value. Both the CPT and SPT information presented in this consider are normalized for overburden stress. In case overburden corrections on cone tip resistance (q) and N-values would not have been made, the ratio of q/N is more veered off from the normal line presented in relationships.

It ought to be noted that, in this study, the same CPT gear and SPT fix were utilized in all tests to minimize inherent test variability. Mean grain measure (D_{50}) based relationships illustrated by Robertson et al. (1983) and Kulhawy and Mayne (1990) are presented in Fig. 5. The information sets chosen for this consideration are then plotted on the same figure to assess the pertinence of these relationships to the silty sand soils. A significant diffuse in the information can be seen around the normal relationship curves of these two relationships. It can to be observed that a little alter in D_{50} can cause a critical change in $q/(N_1)_{60}$ proportion and in most cases $q/(N_1)_{60}$ ratio expires with expanding D_{50}, which is to some degree conflicts with the existing relationships. Besides, no common trend can be observed in the plotted information with the alter in D_{50} recommending destitute appropriateness of these relationships to the silty sand soils.

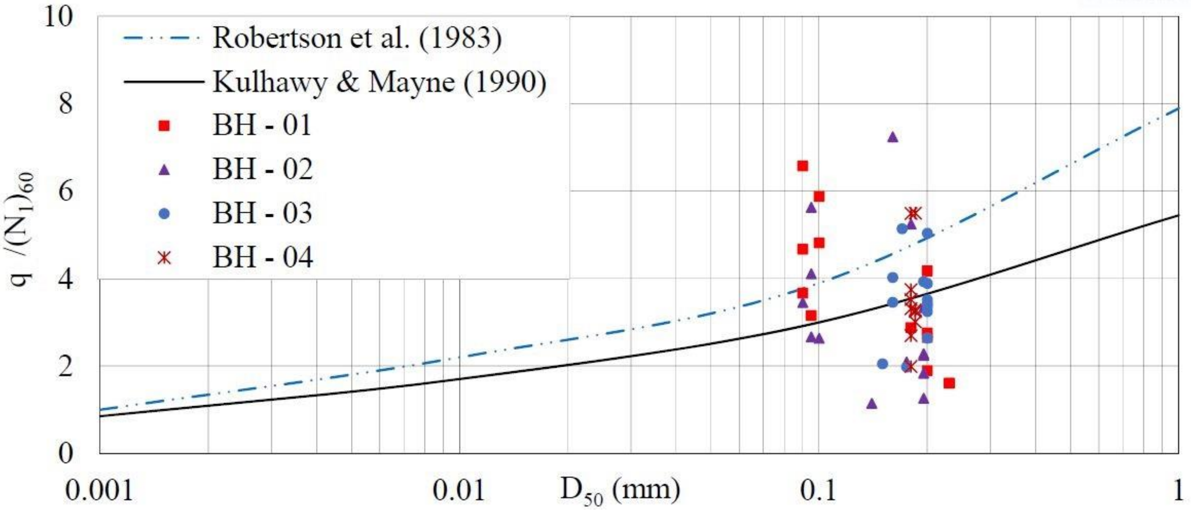

Fig. 5 - Variation of the ratio q/ (N1)60 with mean grain size (D50) comparison

Fig. 6. Along collected information sets of this study. The collected information sets appeared a destitute fit to the fines substance-based relationships and cannot substantiate a common slant between fines substance and $q/ (N_1)_{60}$ ratio. From the plotted information it can moreover be noted that the $q/ (N_1)_{60}$ ratio expanded with expanding fines substance, which is very contradictory to the existing correlations. In a comparable design, fines substance based relationships moreover demonstrate destitute pertinence to the study soils. The information scramble in comparison results may be due to the inherent inconstancy of the two entrance tests and inaccuracy of overburden calculation during N-value correction. Other than soil structure, location topography, or changes in subsurface conditions a few of the irregularity may be due to the some degree expansive remove between the SPT and CPT locations.

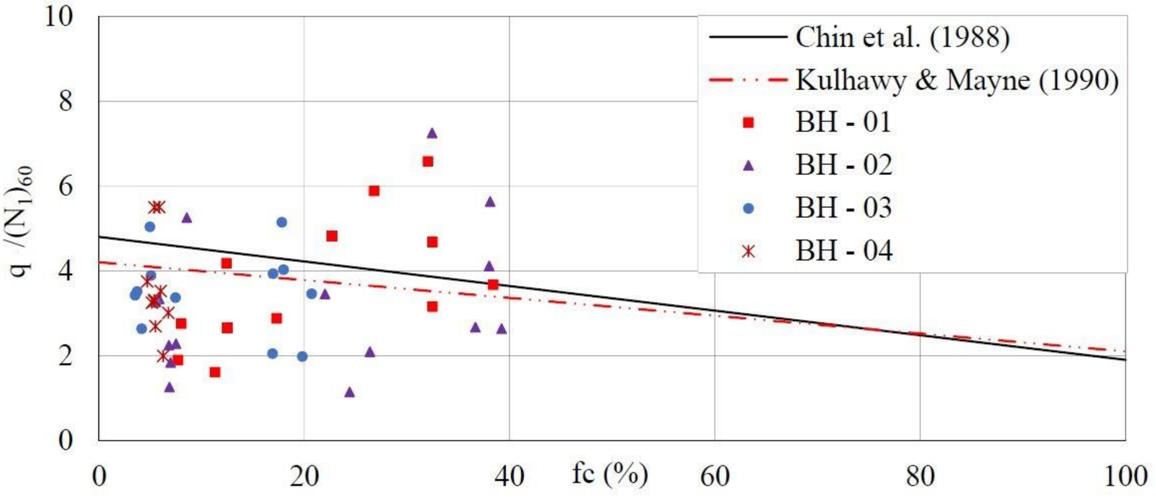

Fig. 6 - Variation of the ratio q/ (N1)60with fines content (fc) comparison

Figure 7 compares the measured information from the lake with the single value of $q/ (N_1)_{60}$ ratio of 0.45 recommended by Elbanna et al. (2011) for sands. The information set from this consider appears great assertion and clusters together around the normal relationship. However, the plotted information appears marginally higher esteem than the normal esteem of 0.45, but it is shocking that this relationship appears a common drift and less scramble plot relative to past comparisons. It is accepted that the relationship between normalized cone tip resistance (q) and normalized SPT $(N_1)_{60}$ can serve as a better relationship for these sandy soils. The most advantage of this relationship is that it can be utilized within the nonappearance degree results. Therefore, it could have to be compelled to collect extra tall quality CPT and SPT information to create a better relationship.

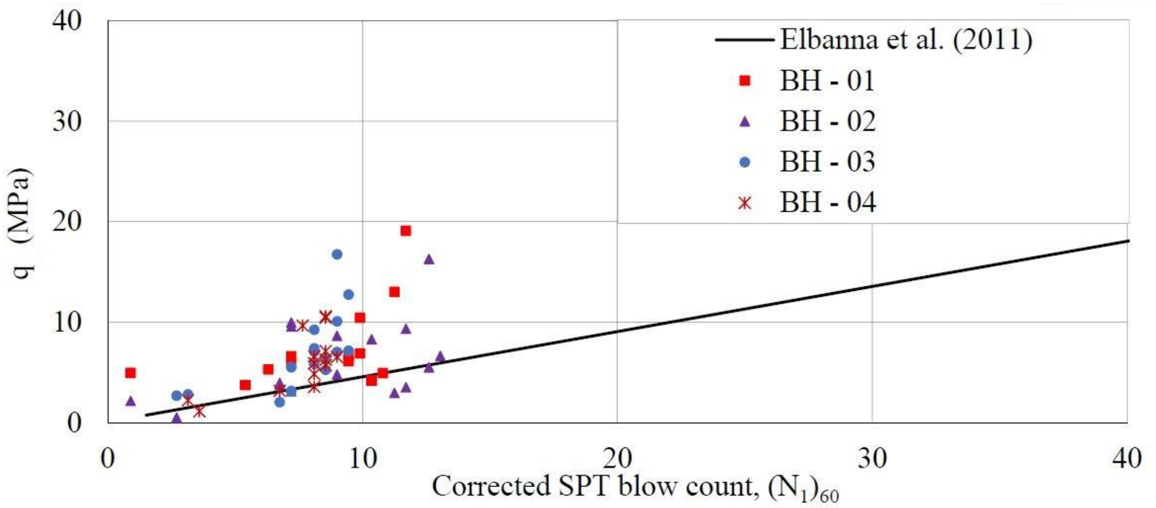

Fig. 7 - Variation of normalized cone tip resistance (q) with normalized SPT blow count (N1) 60 comparison

The basic shear strength parameter of sands is the effective point of inner friction. Hence, an exertion has been made to perform a comparative investigation between the point of inner friction assessed from CPT, and SPT, and those measured in direct shear (DS) and triaxial (TX) tests. An in-situ compelling point of inside grinding was calculated from set-up relationships given Liao et al. (1988) for CPT and SPT tests. Research facility contact points were assessed from solidified undrained triaxial and direct shear tests on reconstituted soil examples at the assessed in-situ relative densities. The effective angle of inner friction values assessed from CPT and SPT tests and those gotten from laboratory triaxial and direct shear tests are presented in Fig. 8.

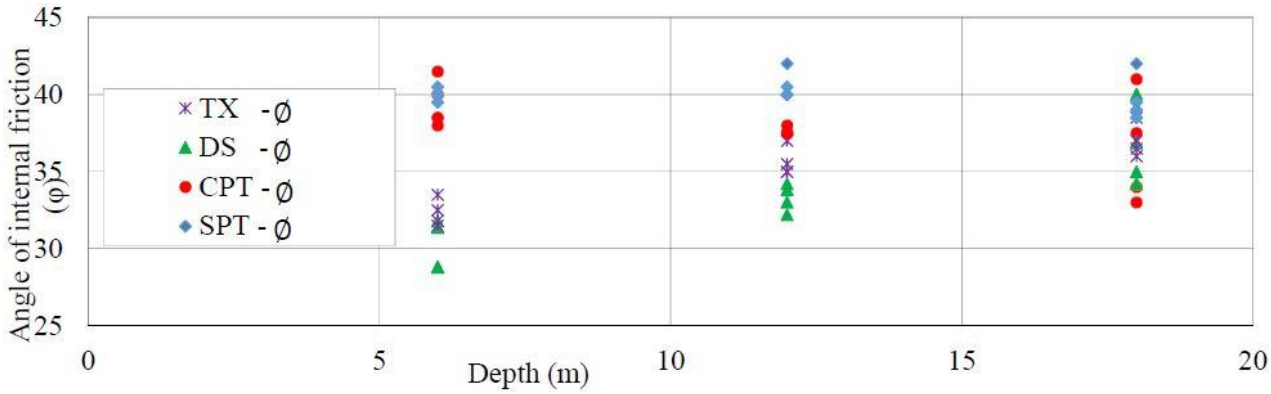

Fig. 8 - Comparative analysis of effective friction angle calculated from SPT, CPT, DS and TX tests

From Fig. 8 it can be seen that Ø values gotten from laboratory triaxial and direct shear tests were put within a contract rage and compared well with each other and expanded with expanding depth. The Ø value calculated from CPT and SPT information appears no common drift and speaks to higher esteem at littler depth and lower esteem at more prominent depth than those gotten from direct shear and triaxial tests.

6. Conclusions

This paper presents the pertinence of different existing relationships between CPT and SPT for nearby soils. It was watched that the CPT and SPT information utilized for this consider superior suit with $(q/N_1)_{60}$ proportion of 0.45. But that, the results of this study impressively change from existing laboratory subordinate CPT-SPT relationships and show that most of these existing relationships are ineffectively pertinent to the neighborhood soils. Hence, it would be significant value to set up dependable relationships based on locally available soils. In addition, accessible setup relationships give a great system, to begin with, but the coordinated application of the normal curve in designing practice may lead to significant deviation. Unfortunately, it is genuine that, as it were chosen information sets were utilized which put generally contract extend to create most of the conspicuous relationships. In this way, it has to be compelled to assist examination by amassing expansive amount of high quality information from different soil types, extending from clay to gravelly sand.

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