GEOTECHNICAL ENGINEERING

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Contents

Summary

This chapter is intended to introduce the major areas of concern in the geotechnical engineering. The methods commonly adopted in soil investigations in the field are first

introduced. These include velocity loggings and sounding techniques which are used to unearth soil profiles at a site in question. Design of foundations of structures is one of the major areas of concern. The principle and methodologies normally used in the design are described with emphasis on piles. Stability of open cuts is another problem of prime importance and the current state-of-the-art is introduced regarding the characteristics of ground movement induced by the open cuts. The ground improvement has constituted a major area in geotechnics. The main techniques being used to stabilize sandy and clayey soil deposits are introduced. Recent development in the technique of solidification of the ground is also described. This is a new subject area in association with underground work in difficult conditions. Lastly, the ground movement induced by shield tunneling through soft soil deposits is described which is an area of concern with respect to urban development. There are many other subject areas such as subgrade engineering and environmental geotechnology, but these items are not described in this report.

1. Introduction

The subject areas traditionally encompassed by geotechnical engineering are summarized in Table 1. Foundations of structures, excavations and tunnels, embankment dams, and subgrades of railways and highways are major areas of profession in which geotechnical engineers play a key role in fulfilling ultimate missions including rational grand planning, cost-effective design, and safe execution of construction work. To maintain integrity of the completed facilities and infrastructures, it is mandatory to have competent soil deposits or rock formation supporting these structures. Any kind of engineering knowledge and skills such as soil improvement and ground reinforcement associated with these operations also constitute the main area of the discipline in geotechnical engineering.

Problems related with protection and improvements of the environment have been recently recognized as a new subject area in which geotechnical engineers could play a key role. Design and construction of landfills and tailing dams are issues of concern in relation to the process of environmental control. Land contamination by hazardous liquids and measures for their containment constitute an important subject area to be tackled by geotechnical engineering.

Natural hazards such as earthquakes, floods and landslides have taken lives and destroyed properties of human beings over many years. The occurrence of these phenomena is not new, but because of proliferation of human activities, chances for people being exposed to these hazards have increased drastically in the recent times, particularly in urban areas. To cope and coexist with these natural hazards, the nature of the phenomena needs to be understood and interpreted properly from geotechnical perspectives to come up with some counter measures of practical significance in order to mitigate possible calamities.

The current state of affairs in these fields of expertise will be introduced briefly in the following. Recent developments in research and practice have been so extensive and formidable that it is beyond the ability of the writer to conduct all embracing overview of the current state-of-the-art. Thus, the following is merely a birds-eye view of several subject areas of major concern in the general framework of geotechnical engineering.

Construction of engineered facilities	Foundation engineering (buildings, bridges and harbors)
	Excavation and tunneling
	Embankments and dams
	Subgrade system (railways, highways)
	Improvement of soft soil ground
Improvements of environment	Disposal of waste (landfills, tailings)
	Control of hazardous materials
Mitigation of natural hazards	Landslides
	Earthquake-induced hazards
	Scour

Table 1: Scope of Geotechnical Engineering

2. Subsurface Investigation for Site Characterization

To execute any plan of construction, it is always necessary, first of all, to know the conditions of the ground consisting of soils and rocks. There are several stages of investigating subsurface conditions as to what kinds of earth materials there are and how competent these are in the light of safe performance of structures after they are installed. It is widely known that, for soils, the crucial factor governing their behavior is the magnitude of shear strain to which they are subjected in the working conditions in the field. Shown in Figure 1 is a list of various testing procedures classified in this vein and also in accordance with whether they are *in situ* tests or laboratory tests.

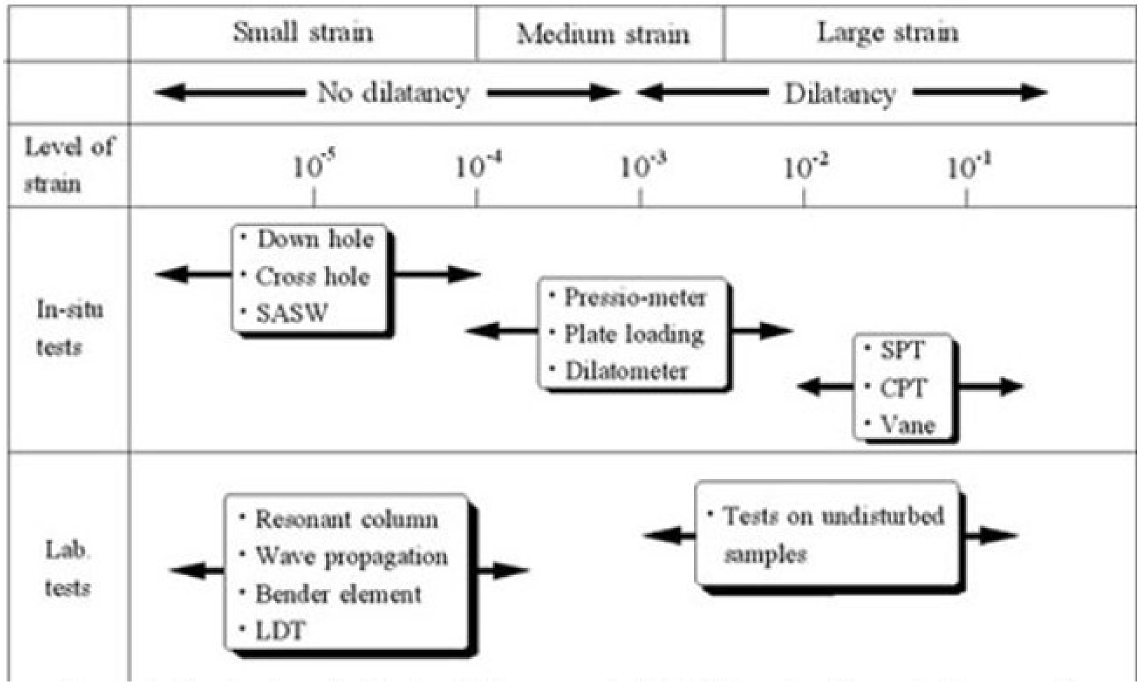

Figure 1: Classification of Methods of Measurement of Soil Deformation Characteristics according to the Level of Strain Involved

In the range of small strains, several test methods based on wave propagation such as the downhole test, crosshole test and Spectral Analysis of Surface Waves (SASW) are employed to investigate the shear modulus of *in situ* soil deposits. In the laboratory, the techniques based on wave propagation such as resonant column test and bender element test are used to estimate the shear modulus of undisturbed samples recovered from field deposits. The precise measurement of the modulus at small strains can be made under static loading conditions as well by means of what is called Linear Differential Transformer (LDT) for undisturbed samples placed in the triaxial chamber.

The above methods in the laboratory are considered to yield fairly accurate values of shear modulus, if the samples are of high quality without being significantly disturbed. Thus, it can be mentioned that good correlation can be established directly between soil property data obtained in the laboratory and in the field, if due care is taken in handling high-quality samples. This fact is indicated in a two dimensional illustration shown in Figure 2.

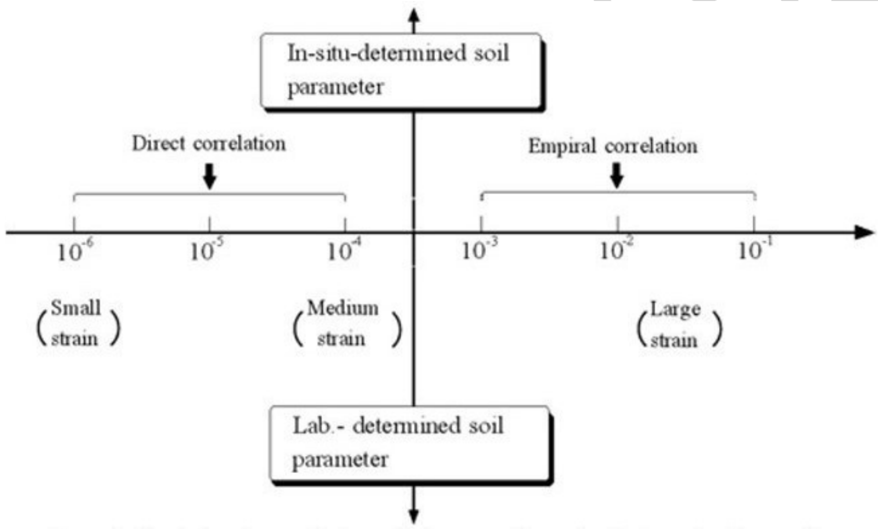

Figure 2: Correlations between *In situ* and Laboratory - Determined Deformation Characteristics of Soils as Governed by the Level of Strains

In the range of large strains involving failure, what might be called the element tests such as triaxial and torsional tests can be performed in the laboratory on undisturbed samples to determine deformation and strength characteristics of *in situ* soils to a desired level of accuracy. On the other hand, it is generally difficult to conduct an element-like test in the field where uniform stress or strain is to be imposed on the soils *in situ*. Therefore, the *in situ* tests such as pressiometer test, plate-loading test and dilatometer test are performed to identify properties of soils as they exist naturally in the field. However, non-uniformity of stress or strain distribution makes it somewhat difficult to interpret the test results from the viewpoint of identifying basic soil parameters. In addition, these *in situ* tests are generally expensive. Thus, various kinds of sounding techniques such as the standard penetration test (SPT), cone penetration test (CPT) and vane test have been commonly used instead in routine practice. Since the soils are deformed largely during any penetration tests, these tests appear to reflect implicitly the deformation properties of soils undergoing large strains including failure.

In an important project to be implemented in the area of soft soil deposits, more detailed information is required. Recovery of intact soil samples can be sometimes executed by means of some sophisticated techniques. Undisturbed samples are tested in the laboratory to identify physical and mechanical characteristics of these soils, but a more detailed description of these techniques is beyond the scope of this chapter. In what follows, an overview will be given regarding some of the most frequently used *in situ* sounding tests such as SPT and CPT and also on the techniques of *in situ* velocity logging.

2.1. Standard Penetration Test (SPT)

One of the most frequently used procedures for monitoring the resistance of soil deposits to penetration is the standard penetration test. The arrangements of this test in the field are shown in Figure 3. The test equipment is composed of a vertical rod equipped with a sampling tube at the end. When performing the test, the rod is lowered to the bottom of a bored hole and a hammer weighing 0.622KN $\cong$ 63.5 kgf is dropped 75 cm along the rod until it hits a stopper attached to the rod. The energy due to the free-falling hammer is transmitted to the sampling tube which is forced to penetrate into the soil deposit beneath the bottom of the hole. The hammer dropping is repeated and the number of droppings required for the sampling tube to penetrate 30 cm is recorded. This blow count number is termed N-value in SPT. The N-value thus obtained has been widely used to assess engineering properties of *in situ* soils.

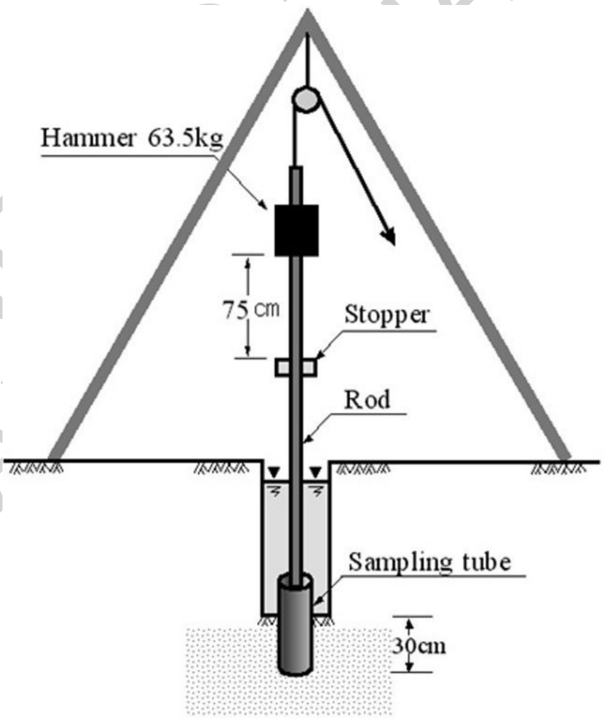

Figure 3: Standard Penetration Test (SPT)

In the SPT the sampling tube consists of a thin-wall steel tube that can be split into two half-cylinders which are fastened together when driven into the soil deposit by the hammer dropping. The intact soil is forced to intrude into the split sampling tube from its open end during the penetration. After lifting the sampling tube on the ground, the split tube is opened and intact soil samples can be recovered. The samples thus obtained are used for visual inspection, physical property testing in the laboratory and for soil classification based on grain composition, plasticity index and so forth. Because the sample is subjected to some degree of disturbance, it is not considered suited to determine the deformation characteristics such as stiffness and strength, but it provides several useful pieces of information on the kind and nature of *in situ* soil deposits. An example of a soil profile unearthed by the SPT is shown in Figure 4.

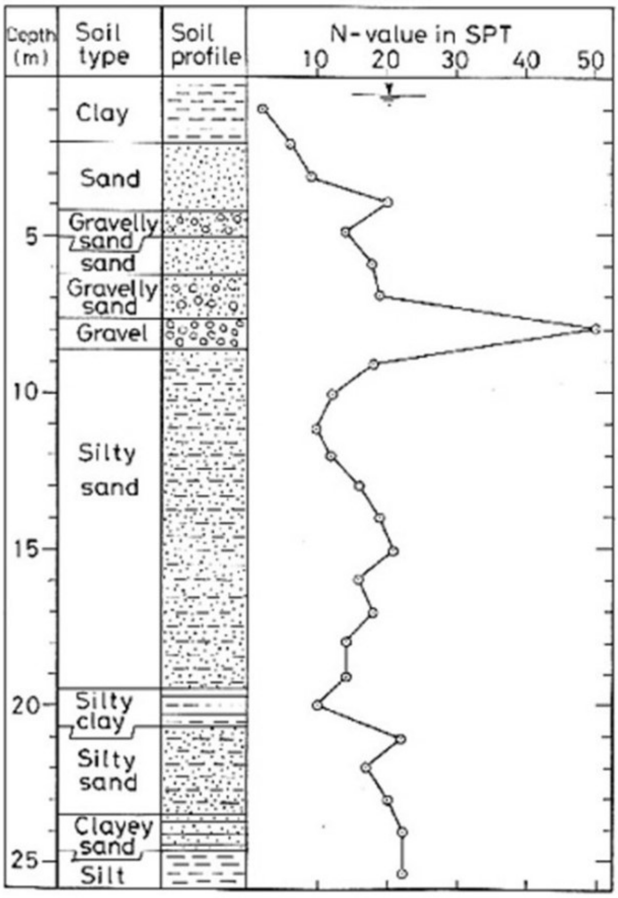

Figure 4: An example of Soil Profile

The advantage of using the SPT would be, first of all, the availability of actual soil samples for visual observation and testing in the laboratory, and secondly the blow count N-value which is indicative of the stiffness and strength characteristics of *in situ* soils. Thirdly, the SPT is versatile and robust enough to perform and accommodate a wide range of soil types from soft clay to gravelly soils.

The SPT is a kind of dynamic penetration test consisting of several steps in operation and hence tends inherently to create some inconsistency in the test results due to differences in machines and skill of operators. For example, the amount of energy transmitted to the sampling tube is known to vary depending upon the mechanism as to how the hammer is released for dropping, the size of the stopper and so forth. Therefore many efforts have been made to identify influencing factors and to make corrections for obtaining consistent data which are comparable among numerous SPT N-values obtained at different parts of the world. For example, the SPT practice in U.S.A. has been identified to impart energy of about 60% of that in free fall of the weight and therefore the blow count in the U.S.A. has been denoted by N_{60}. In Japan the energy ratio transmitted is generally known to range between 70 and 80%.

For sandy deposits the SPT N-value is generally used to estimate the relative density D_r which is known to increase with increasing resistance to liquefaction during earthquakes. Not only with the density, but the N-value is known to increase also with the effective overburden pressure σ_v' at the depth of deposits in question. Thus, the correction is generally made to assess the relatively density by eliminating the effect of the effective overburden pressure. An empirical relation as follows may be used to estimate the relative density of sand deposits from measured N-values.

$$\frac{N}{\sqrt{\frac{\sigma_v'}{98}}} = C_D \cdot D_r^2, \tag{1}$$

where σ_v' is in kPa, and C_D is a constant taking values between 20 and 30 for sands. By putting $\sigma_v' = 98\text{kPa}$ in Eq. (1), N_1-value corresponding to $\sigma_v' = 1\text{kgf/cm}^2 \cong 98\text{kPa}$ is obtained as

$$N_1 = C_D \cdot D_r^2. \tag{2}$$

It is to be noted that the N_1-value normalized as above is indicative of the relative density alone irrespective of the overburden pressure. The value of $C_D = N_1 / D_r^2$ defined by Eq. (2) is known to vary depending upon the grain composition of the granular materials. The outcome of compilation of many data on C_D is demonstrated in Figure 5 in terms of plots against the mean grain size D_{50}.

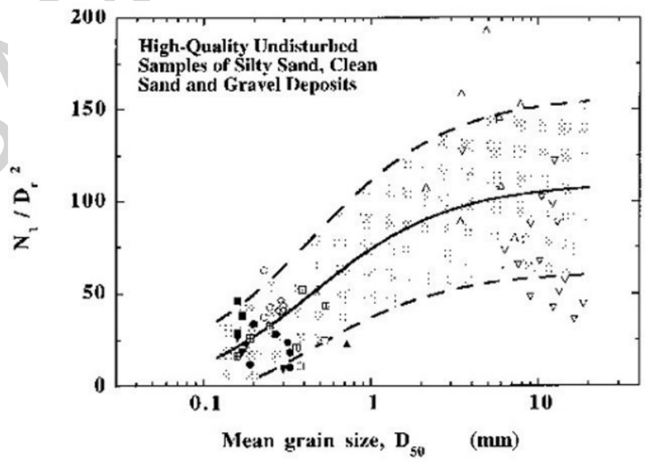

Figure 5: N_1 / D_r^2-Values Versus Mean Grain Size of Cohesionless Soils

2.2. Cone Penetration Test (CPT)

The cone penetration test has received wide publicity in recent years as a useful means to identify soil properties under intact conditions in the field. An apparatus developed by Begemann (1965) in its early period of development is shown in Figure 6, where the cone device consists of the end cone and adhesion jacket. Using this device, the local skin friction could be measured in addition to the end cone resistance. The cone device as above is pushed down by the force that is applied and measured on the ground surface and thus called “Mechanical cone” as against the electrical cone as described below. Various types of the mechanical cone penetrometers are now in use widely because of their low-cost, simplicity and robustness.

Electrical cone penetrometers have also received wide publicity in recent years, because of easiness in handling and consistency in obtaining reliable test data irrespective of possible variation in skill of operators. One of the typical models is shown in Figure 7 where measurements of the cone resistance and sleeve friction can be made by independently operated load cells. The penetration of the electric cone is also performed at a constant speed automatically. Sometimes, pore water pressure could be measured during the penetration, enabling more exact identification of soil types to be made for layered structures of *in situ* deposits of soils. The device equipped with the probe for pore pressure monitoring is called “Piezocone”.

In the operation of the CPT, the cone tip at the end of the rod is pushed into the soil deposit at a constant rate and measurements are made continuously or intermittently of the resistance against penetration of the cone. Measurements of frictional resistance of the surface of a sleeve are also made individually or in combination with the cone resistance. The total force acting on the cone divided by the cross sectional area is called “*Cone resistance*” and denoted by q_c and it is commonly used as a parameter to quantify the cone resistance. The total force acting on the friction sleeve divided by the surface area of the sleeve is called “*Sleeve friction*” or “*Skin friction*”, and denoted by f_s.

One of the disadvantages of the CPT is that it is not possible to recover actual soil samples from *in situ* soil deposits. Therefore, some indirect methods should be utilized instead to identify soil types in each layer of the deposits. It has been known that for sandy soils, the major component of resistance to penetration is the cone resistance q_c, whereas the skin friction f_s constitutes the major part of resistance in the case of cohesive soils. So, the percentage of the cone resistance tends to increase with increasing proportion of cohesionless materials included in actual soils. Shown in Figure 8 is the qc-value for various materials with different proportion of fines content of cohesionless soils plotted versus the skin friction f_s. It is obvious that the value of q_c trends to increase with increase in the skin friction f_s, but the rate of increase in q_c is known to rise from clayey to sandy soils, with increasing percentage of cohesionless materials contained in the soils. Thus, if a chart were provided plotting q_c versus what is called friction ratio, F_r, defined by $F_r = q_c / f_s$, it would be possible to distinguish various zones which are associated with different soil types ranging from sandy to clayed soils. One of such charts proposed by Douglas and Olsen (1981) is demonstrated in Figure 9. If a set of measured q_c and F_r values is plotted in the upper left part of the chart, the soil is classified as sandy. If a data point falls in the lower right portion, the soil is identified as clayey. More detailed classification chart was proposed by Robertson (1990).

The advantage of using the CPT is the fact, first of all, that it is speedy and easy to execute and robust. Secondly, it permits coherent data to be obtained being less influenced by the skill of the operator, and thirdly, data processing can be automated without difficulty, if the electrical cone is used. The shortcoming of the CPT is that it becomes impossible to penetrate to a target depth if the CPT encounters stones or gravels during its penetration.

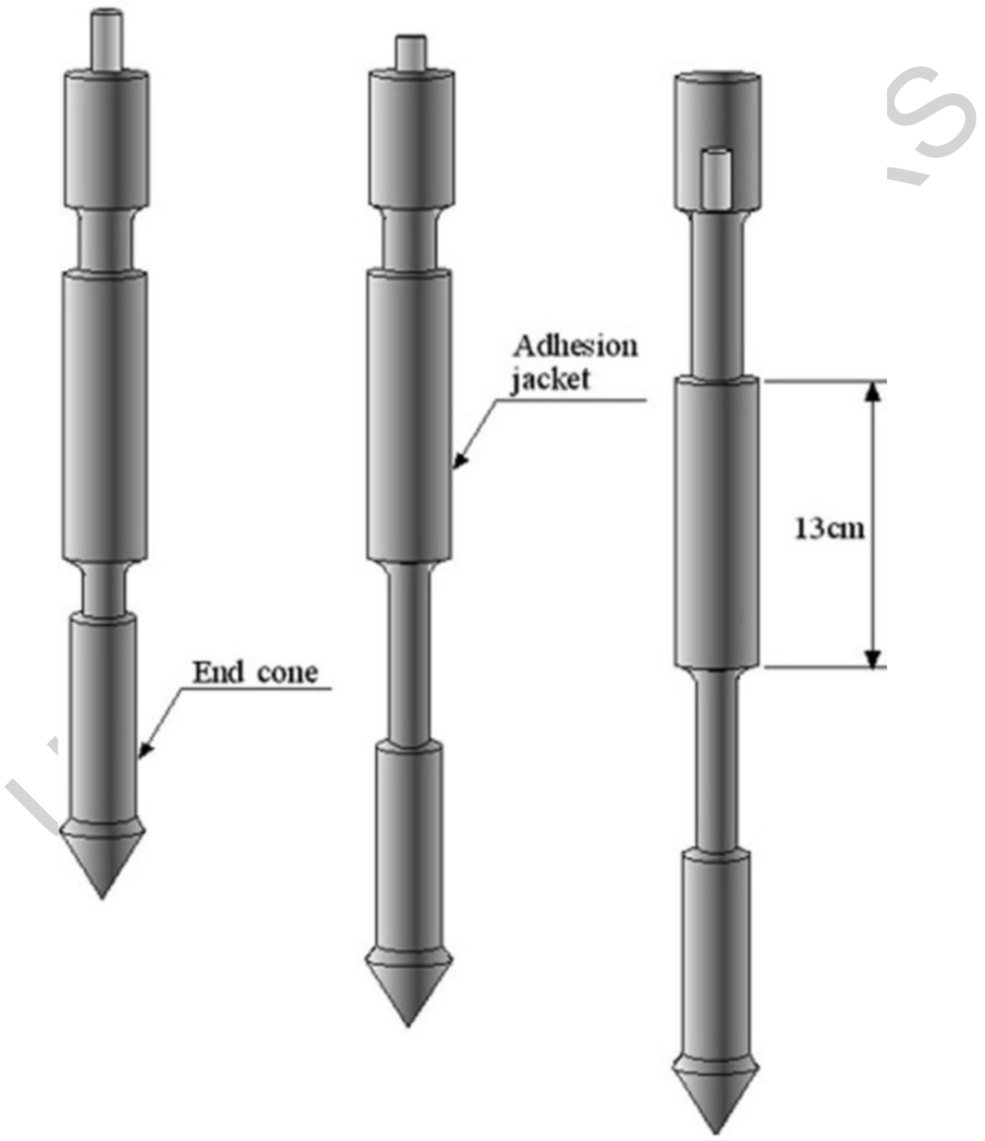

Figure 6: Begemann Type Cone with Friction Sleeve

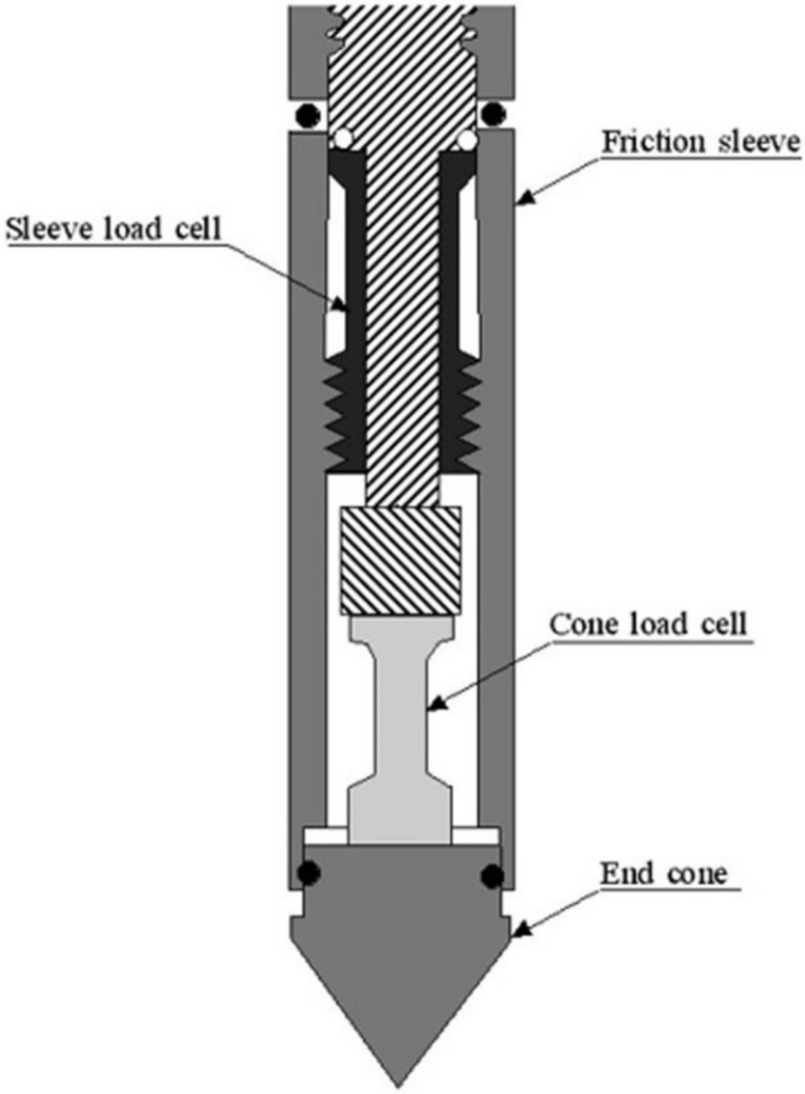

Figure 7: Electrical Cone

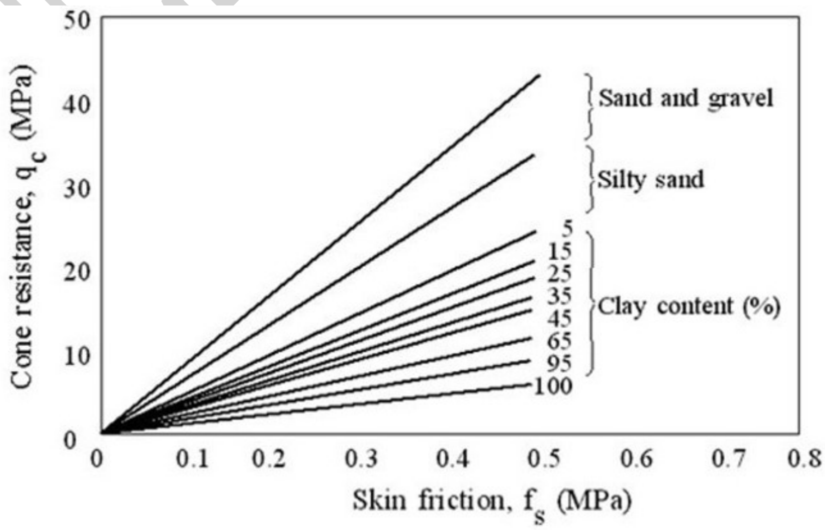

Figure 8: Soil Classification from Cone Resistance and Sleeve Friction Readings (Begemann, 1965)

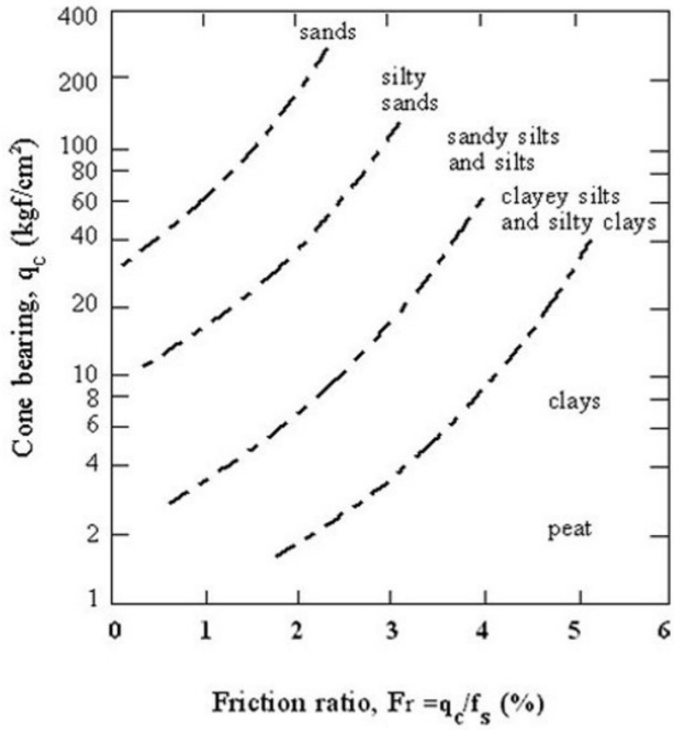

Figure 9: Simplified Soil Classification Chart for Standard Electric Friction Cone (Douglas and Olsen, 1981)

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Biographical Sketch

Kenji Ishihara had served as a professor of soil mechanics and geotechnical engineering at the University of Tokyo from 1977 to 1995 and then became a professor of civil engineering at the Science University of Tokyo from 1995 on to 2001. He now teaches soil mechanics and foundation engineering at Chuo University in Tokyo, Japan. He served as President of the International Society for Soil Mechanics and Geotechnical Engineering for the term 1997-2001.