A NON-MECHANISTIC PERSPECTIVE OF GEOTECHNICAL ENGINEERING

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ABSTRACT

Engineers deal mostly with materials that have unique and specific properties. On the other hand, entities are somewhat like living beings that do not possess unique properties but exhibit behavioral responses to stimuli (actions). Clays and sands can exist in different states ranging from liquid to solid and loosest to densest respectively. Responses of soils in general and ground in particular are examined and analyzed under different test and design conditions. Similarly, the state in which ground exists can be quantified though the overconsolidation ratio. The most commonly used parameter, the undrained shear strength, is sensitive to the manner in which it is determined. The paper emphasizes the need to visualize soil in the specific sense and ground in a broader perspective as entities rather than as strictly engineering materials. It is suggested that geotechnical engineering should be viewed comprehensively and beyond a simple mechanistic perspective. A unique comparison of ground with a human being elucidates the concepts enunciated.

Keywords: Ground, Entity, Non-mechanistic perspective, Human body

INTRODUCTION

Engineers typically deal with objects or materials with a view to build a component, a machine, or a process. A 'material' is defined as that consisting of matter but lacking spirituality or personality while an 'entity' or 'being' is an object that has life and thus reacts to stimuli. Civil engineers normally deal with several materials such as steel, cement, concrete, brick, wood, water, and soil. Following the tradition of applying the principles of mechanics to these materials for the purpose of analysis and design in Civil Engineering, soil also has been and is being treated as a material. Consequently, Soil Mechanics, as was known earlier, and Geotechnical Engineering, the current nomenclature, has evolved as an engineering discipline.

Engineers solve engineering problems based on their knowledge of solid and fluid mechanics, and analytical ability. The final judgment is based on the analysis carried out on the well-accepted premise that the 'thing' they are dealing with is a material. Traditionally, it is physicians and psychologists who deal with individual human beings while sociologists deal with groups of individuals and larger entities such as human societies. The response of an object made of a material is fairly well predictable, with the margin of error being less than a few percentages. However, no such luxury is available when dealing with human beings whose responses are not only indeterminable but are controlled by several factors.

Practitioners of geotechnical engineering can vouchsafe to the fact that ground responds, rather than exhibit precise engineering characteristics. Hence, there are several conferences on case histories in geotechnical engineering and predictive behavior symposia. It is the purpose of this paper to suggest a paradigm shift in conventional thinking from a 'material' to 'entity' centered approach while dealing with soil in general but specifically 'ground'. While the central kernel of the analysis may remain as conventional mechanistic view, the final judgment or decision should be based on a broader perspective of treating ground as an entity that has several features somewhat akin to a human being. Thus, both the approaches are complementary and not contradictory.

SOIL

Soil is a complex three-phase material formed over a long period of time from physical and chemical weathering of parent rock. Soil can neither be termed as a solid nor as a liquid, its behavior changing with either water content or dynamic input. For instance, the states of fine-grained soils are known to vary from liquid, plastic, semi-solid to solid states, with changes in water content. Loose saturated coarse-grained soils may lose all their strength and get liquefied during a seismic event of sufficient intensity. However, ground improvement techniques such as vibro-compaction and heavy tamping help densify such soils and mitigate liquefaction.

Upward flow of water through a granular medium, in particular, can lead to the phenomenon of ‘quick’ condition wherein the ground loses its strength. Furthermore, soils that were initially stiff and strong may lose their strength and stiffness upon disturbance. In fact, sensitivities of the order of 100 or even more are not uncommon. Thus, soil can be characterized as porous, saturated/unsaturated, non-homogeneous, anisotropic, inelastic (elasto-viscoplastic), dilatant, sensitive, with failure state varying from brittle to ductile, and a material with memory (preconsolidation stress, overconsolidation ratio).

NON-UNIQUENESS OF PREDICTABILITY

Geotechnical engineers perform basically two types of analysis, one for stability and the other for serviceability. Examples of stability analyses include estimation of bearing capacity of foundations, lateral earth pressures on retaining structures, and stability of natural or man-made slopes and embankments. A factor of safety, in the working stress method of design, usually accounts for most of the uncertainties of soil as a material, the type of analysis, and loads. The actual performance of the structure is unknown except for the fact that either it exists or has failed or collapsed, unless it has been instrumented and monitored.

Drilled Shaft

Figure 1 compares the measured capacity of an 18 in. (457.2 mm) diameter, 50 ft. (15.2 m) long drilled shaft with predictions made by several geotechnical consultants and practitioners in the academic and non-academic fields. The drilled shaft was constructed through 23 ft. (7 m) of poorly graded sand overlying 45 ft. (13.7 m) of soft to medium clay. Apart from the total capacity, the shaft resistance of the pile in the sand and clay layers, as well as the base resistance, are shown in Fig. 1. The measured ultimate capacity of the drilled shaft was 410 kips (1824.5 kN). Contrastingly, the predicted ultimate capacities varied from as low as 130.5 kips (580.8 kN) up to a large value of 518 kips (2305.2 kN). Thus, the predicted values ranged from 0.32 to 1.26 times the measured value, which is a substantial range. Out of twenty predictors, only two (predictors 1 and 2) were close enough while thirteen predictors grossly underestimated the pile capacity and one overestimated the capacity.

Deep Excavation

Figure 2 depicts the geometry of a 32 m deep excavation in Berlin sand using three rows of prestressed anchors connected to a diaphragm wall.

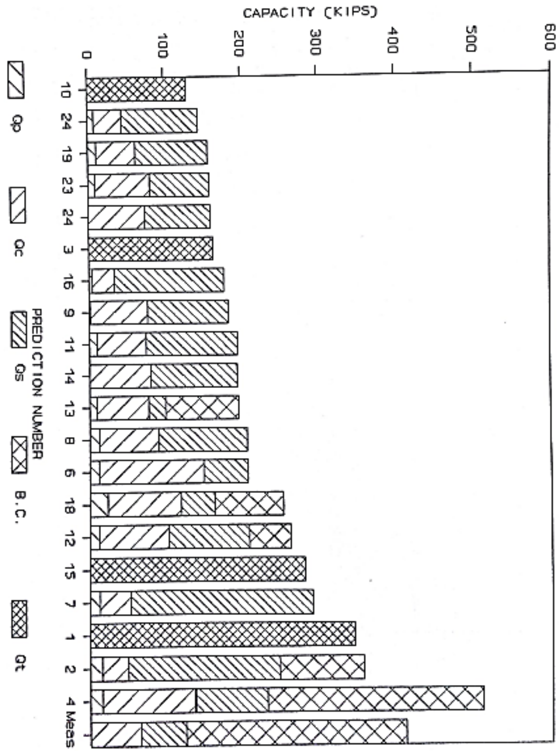

Fig. 1 Comparison of predicted and measured drilled shaft capacities [1].

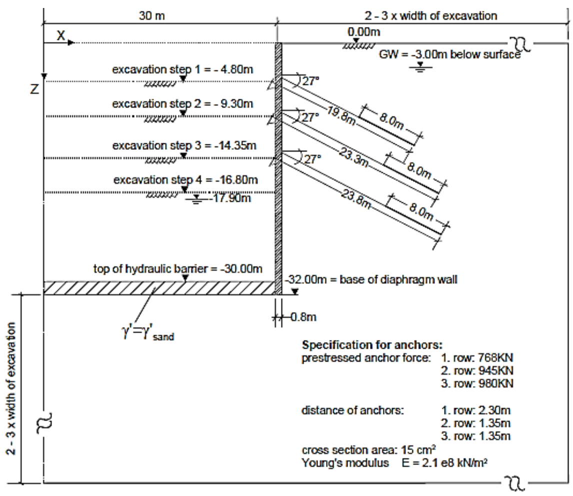

Fig. 2 Geometry of deep excavation in Berlin sand [2].

The excavation was conducted in four steps after lowering the groundwater table. The anchors were 20–24 m long, spaced at 1.3–2.3 m and inclined at 27° to the horizontal. The moist unit weight and angle of shearing resistance of sand were 19 kN/m^3 and 35° respectively. The problem was part of a benchmarking exercise specified by the German Society for Geotechnics and sent to 17 universities and companies all over the world who were known to perform numerical analysis.

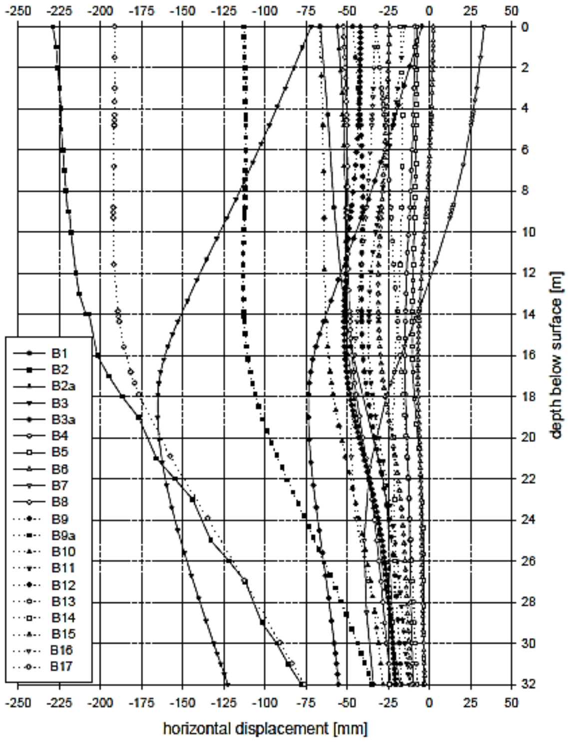

Fig. 3 Horizontal displacement of wall at final excavation stage [2].

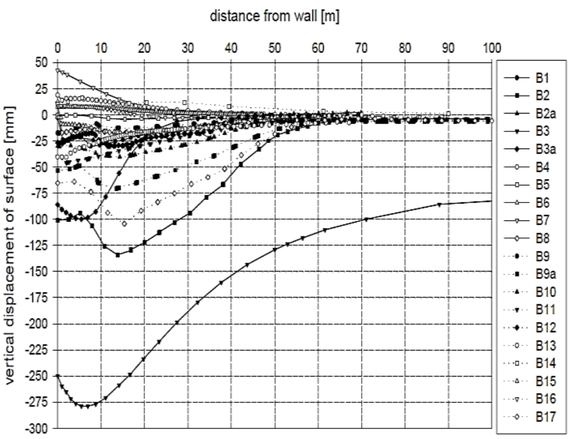

Fig. 4 Settlement profiles of ground surface at final excavation stage [2].

Figure 3 shows the predicted horizontal displacement profiles of the wall by the 17 groups. The predicted horizontal displacement at the top of the wall varied between -229 mm and +33 mm (negative sign for displacement towards the excavation). It can be observed that the differences in the horizontal displacements and deflected shapes of the wall, estimated by several predictors, are quite remarkable.

Figure 4 presents the predicted surface settlement profiles of the ground behind the wall. The settlement predictions varied from -275 mm to +40 mm (negative sign for heaving of ground). A hypoplastic model without consideration of inter-granular strains was used by predictor B3 to estimate the -275 mm settlement, whereas the +40 mm surface heave was estimated by predictor B7 using an elastic–perfectly plastic constitutive model with constant ground stiffness. The variation in the pullout forces predicted in the three rows of anchors is also enormous (Fig. 5). The predicted pullout forces ranged from 129 to 635 kN/m in the first anchor row, 431 to 937 kN/m in the second anchor row, and 514 to 1069 kN/m in the third anchor row, respectively. Significant differences in the results were reported even in cases where the same software was employed by different users [2].

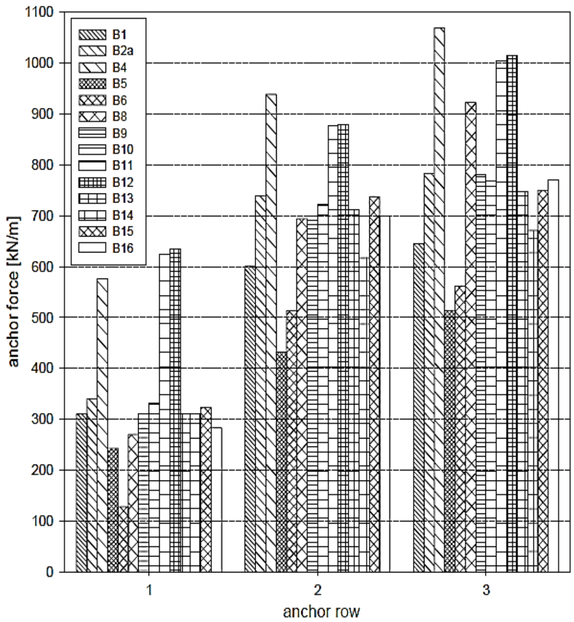

Fig. 5 Anchor forces at final excavation stage [2].

Figures 1 to 5 thus bring out an important result; either the ability to predict the response of ground to imposed loads using mechanistic approach is inadequate or ground does not fit into the conventional concept of a ‘material’ and hence its response needs to be predicted conjointly with non-mechanistic view as well. The response of a material is predictable as long as its properties are determined and the mechanics and kinematics understood. In contrast, the response of a non-material or an entity which is akin to a living organism, depends on several factors such as its origin, the past history, and the environment in which it exists and operates. One of the most important parameters is the stimuli which causes the response.

STATES OF EXISTENCE

A material exists in a given form or state unless extremely large changes in environmental conditions (temperature, humidity etc.) are induced. Thus, water which is in liquid form over a temperature range of 0 to 100º C changes to solid or vapour phases only if the temperature is less than zero or more than 100^0 C, respectively. Soil, contrastingly, changes state under normal engineering conditions with changes in water content, void ratio and effective stress. Parameters such as liquidity index $LI = (w_n - w_p)/PI$, where w_n is the natural water content, w_p is the plastic limit and PI is the plasticity index; relative density $D_R = (e_{max} - e)/(e_{max} - e_{min})$ where e_{max}, e and e_{min} are the void ratios at the loosest, natural and densest states, respectively; and overconsolidation ratio $OCR = \sigma'_{vp}/\sigma'_v$ where σ'_{vp} is the preconsolidation pressure and σ'_v is the current in-situ vertical effective stress, are defined as the state parameters. The response of soil or ground can be predicted provided the state in which they exist is known or quantified.

ATTERBERG LIMITS AND GRAIN SIZE DISTRIBUTION

The grain size distribution (GSD) and Atterberg limits are probably the most basic and fundamental properties of soil. Table 1 illustrates the sensitivity of these properties to the process of determining the same. The Atterberg limits and grain size distribution were determined for natural and washed soils in moist, air dried and oven dried conditions [3]. The response as measured can be very significant. The liquid limit w_L of natural soil reduced from 108% to 73% and 56.5% for air and oven-dried conditions, respectively. The plastic limit w_p also reduced in the same form but not as dramatically; however, the plasticity index PI got affected because of the sensitivity of the liquid limit.

A similar response can be observed for soil that has been washed prior to testing. The plasticity index reduced from 65.2% to 37.7% and 22.8% for natural soil and from 65.3% to 46.9% and 31.2% for washed soil under moist, air and oven-dried conditions, respectively. While the shrinkage limit w_s was least affected by these conditions and processes, the grain size distribution (clay, silt and sand contents) was affected to different degree.

Table 1 Sensitivity of soil properties to process of determination [3]

Description	Atterberg Limits				Grain Size Distribution		
	w_L (%)	w_p (%)	w_s (%)	PI (%)	Clay (%)	Silt (%)	Sand (%)
Natural Soil							
(a) Moist	108.0	42.8	20.3	65.2	42.0	39.0	19.0
(b) Air Dried	73.0	35.3	20.2	37.7	30.0	44.0	26.0
(c) Oven Dried	56.5	33.7	21.4	22.8	23.0	49.0	28.0
Washed Soil							
(a) Moist	109.0	43.7	21.3	65.3	43.0	41.0	16.0
(b) Air Dried	82.5	35.6	19.3	46.9	30.0	54.0	16.0
(c) Oven Dried	65.5	34.3	20.4	31.2	25.0	57.0	18.0

SHEAR TYPES AND TESTS

Analysis of stability is one of the most common tasks a geotechnical engineer carries out. Figure 6 depicts an embankment constructed on soft ground. A typical failure surface is usually assumed and the factor of safety is computed for this configuration. The question is what value of undrained strength should be assigned to the ground which is in saturated condition? The state of soil along the assumed failure surface varies from an 'active' state beneath the embankment to 'simple or pure shear' at the deepest point and to a 'passive state' at the farthest end. Is the undrained shear strength of ground a 'unique' property or does it depend on the manner in which it is determined? The undrained shear strength of a sample of soil from the ground can be determined in direct shear (DS), direct simple shear (DSS), plane strain compression (PSC), plane strain extension (PSE), triaxial compression (TC) and triaxial extension (TE). The direction of the principal stresses and the manner in which they are applied are different for each test (Fig. 7).

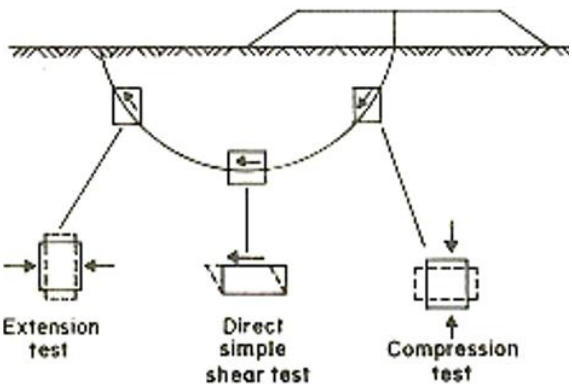

Fig. 6 Types of shear along slip surface of embankment on soft ground.

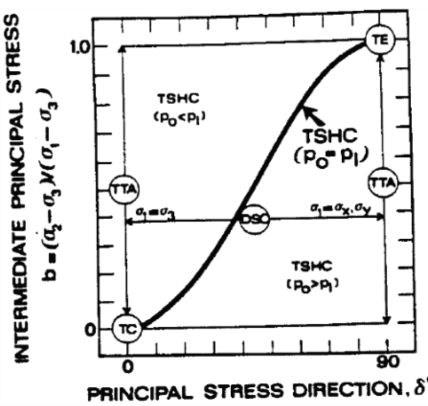

Fig. 7 Stress states for different shear tests [4].

The parameters of importance are the inclination δ of the major principal stress σ_1 with respect to the vertical axis, the relative magnitude of the intermediate principal stress $b = (\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3)$, and their variations during the test. The major principal stress is oriented in the vertical direction and $b = 0$ for TC while the major principal stress rotates by 90^0 and $b = 1$ for TE. The value of b is in-between 0 and 1 and close to about 0.4 for PSC and PSE. The orientation of the major principal stress is somewhat indeterminate and variable for DS and DSS tests.

Figure 8 shows the state of stress along the slip surfaces of various geotechnical structures (embankment, retaining wall, slope, and drilled shaft). It can be observed that no one type of test can address the actual field behavior of ground. Instead, a combination of tests is needed to characterize the state of stress of a soil element along various locations of the slip surface. If the wrong test is used to characterize a particular field problem, the corresponding factor of safety would be completely different to that of the actual value.

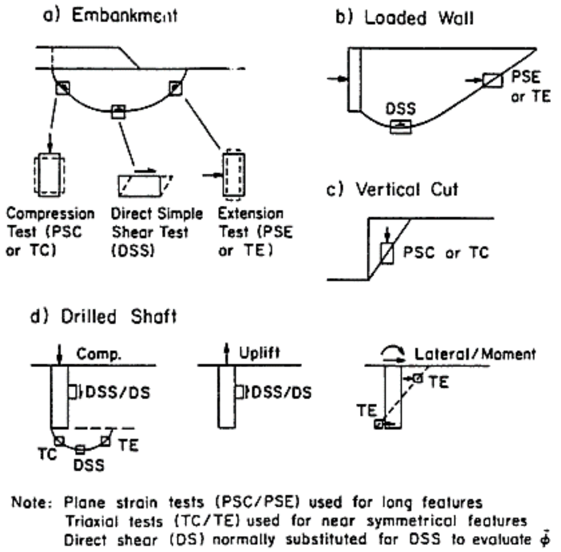

Fig. 8 Relevance of laboratory strength tests to field conditions [5]

UNDRAINED SHEAR STRENGTH

The variation of the ratio $(s_u/\sigma'_{v0})/(s_u/\sigma'_{v0})_{\text{CIUC}}$ of normalized undrained strength (normalized with respect to the in-situ vertical effective stress σ'_{v0} at the depth where s_u is evaluated) for a given shear test, over that for undrained strength from isotropically-consolidated undrained triaxial test, versus the angle of shearing resistance from triaxial compression test $\bar{\phi}_{tc}$ is presented in Fig. 9. The ratio $(s_u/\sigma'_{v0})/(s_u/\sigma'_{v0})_{\text{CIUC}}$ for K_0-consolidated undrained triaxial compression CK_0UC decreases from 1.19 to about 0.7 for $\bar{\phi}_{tc}$ increasing from 0^o to 50^o. The corresponding decreases for PSC, DSS, PSE and CK_0UE tests are 1.35 to 0.7, 0.8 to 0.46, 0.69 to 0.46 and 0.58 to 0.34, respectively. Thus, soil exhibits different strengths from different shear tests and does not have a unique undrained strength. Even the angle of shearing resistance ϕ is not a unique parameter, the value for plane strain conditions being 1.1 times the value for triaxial conditions.

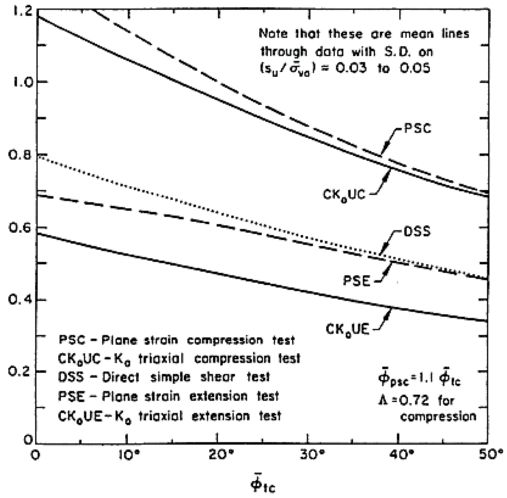

Fig. 9 Mean normalized undrained strength ratios for various angles of shearing resistance from different laboratory tests [5]

Table 2 quantifies the different undrained strength ratios $(s_u/\sigma'_{v0})/(s_u/\sigma'_{v0})_{\text{CIUC}}$ (the normalized undrained strength for the specific field conditions with respect to the normalized undrained strength for undrained compression on isotropically-consolidated samples) for the corresponding field condition. The ratios are given for angles of shearing resistance of 20^o, 30^o and 40^o. Different combinations of shears operate for different field loading conditions and hence the strength appropriate to a specific condition is very different compared to those for other conditions. For $\bar{\phi}_{tc}$ of 30^o, the strength ratio varies from a high value of 0.85 for short vertical cut to a low value of 0.42 for shaft lateral load condition. A designer or geotechnical specialist has to identify the particular value based on the relevant filed condition as the value can be obtained from a simple codal provision as is possible for engineering 'materials'.

Table 2 Undrained strength ratios for different field loading conditions [5]

Field Loading Condition	$(s_u/\bar{\sigma}_{vo})/(s_u/\bar{\sigma}_{vo})_{CIUC}$		
	$\bar{\phi}_{tc} = 20°$	30°	40°
Long Embankment (PSC + DSS + PSE)	0.75	0.67	0.60
Long Wall (DSS + PSE)	0.62	0.57	0.51
Short Vertical Cut (CK_oUC)	0.94	0.85	0.75
Shaft Bearing Capacity (CK_oUC + DSS + CK_oUE)	0.68	0.62	0.55
Shaft Side Resistance (DSS)	0.64	0.58	0.51
Shaft Lateral Load (CK_oUE)	0.47	0.42	0.38

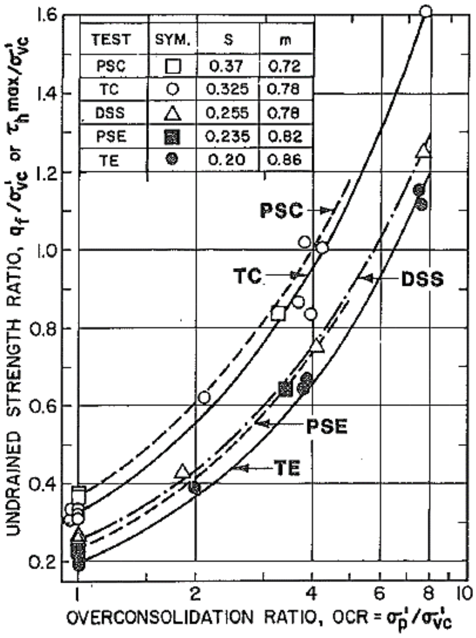

Fig. 10 Normalized undrained strength ratio versus OCR from CK_0U tests [7].

Figure 10 illustrates the variation of the undrained strength ratio s_u/σ'_{vc}, with the OCR of New Jersey marine clay for K_0-consolidated undrained (CK_0U) TC, TE, PSC, PSE and DSS tests. The CK_0U tests were performed on specimens reconsolidated beyond the in-situ preconsolidation pressure σ'_{vp} to a maximum vertical stress σ'_{vc} equal to $(2 \pm 0.5)\sigma'_{vp}$. The corresponding vertical strains induced were of the order of 14 ± 3%. The specimens were then sheared either in a normally consolidated state (OCR = 1) or rebounded to obtain an OCR value higher than 1. *S* and *m* are the parameters of the Stress History and Normalized Soil Engineering Properties (SHANSEP) technique [6]. The undrained strength ratio for TC increases from 0.32 to 1.6 for OCR increasing from 1 (normally consolidated) to 7.5 (highly over-consolidated). The corresponding increases for DSS and TE tests are 0.27 to 1.25 and 0.2 to 1.13, respectively. The undrained strength ratio for PSC and PSE increases from 0.36 to 0.84 and 0.22 to 0.64, respectively, for OCR increasing from 1 to 3. Thus, soil at a given OCR exhibits different strengths from different shear tests and does not have a unique undrained shear strength.

Figure 11 depicts the undrained shear strength s_u profiles of Bothkennar clay obtained from cone pressuremeter test (CPMT), field vane shear test, unconsolidated undrained triaxial (UUT) test and self-boring pressuremeter (SBPM) test. It can be observed that s_u is not unique but depends on the type of test. Further, s_u clearly increases with depth, thus indicating non-homogeneity of soft ground. CPMT gives lower values of s_u while SBPM gives higher values of s_u when compared to the other tests.

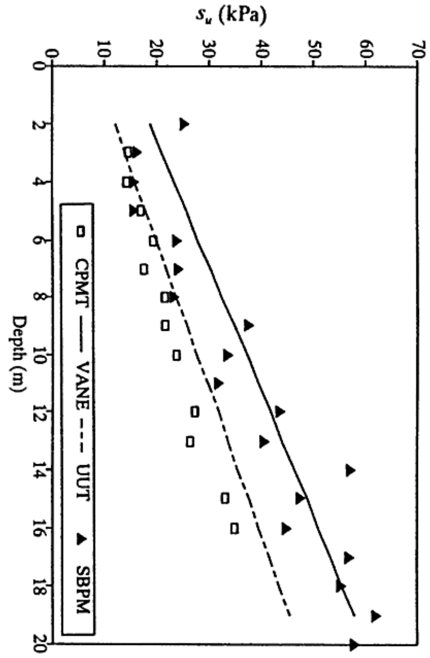

Fig. 11 Undrained shear strength profiles of Bothkennar clay from CPMT, field vane, UUT and SBPM tests [8].

SHEAR MODULUS

Figure 12 shows the variation of the shear modulus *G* of Bothkennar clay with depth, obtained from CPMT, SBPM and seismic cone penetration test (SCPT). Similar to the undrained shear strength, the shear modulus also increases with depth and depends on the manner in which it is determined.

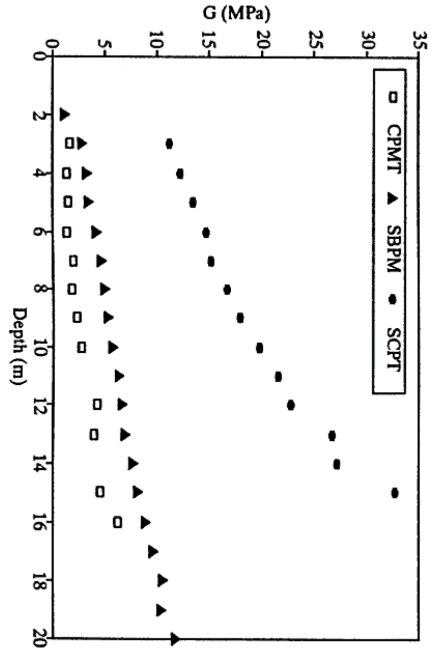

Fig. 12 Shear modulus profiles of Bothkennar clay from CPMT, SBPM and SCPT [8].

The large difference in the shear modulus (almost by an order of magnitude) obtained from SCPT versus that from CPMT and SBPM is possibly due to different strain amplitudes imposed on the ground by each test and the type of deformation induced [8]. In the case of SCPT, the loading is vertical and the soil near the tip of the cone is pushed down leading to both vertical and lateral deformations. Pressuremeter testing involves deforming the soil around the probe in the lateral/radial direction. The response is thus from an orthogonal direction compared to that of SCPT. Also, the cone tip is of extremely small area while the pressuremeter test involves soil around a finite diameter cavity. Thus, a unique shear modulus for ground cannot be defined. In fact, the differences between the results from SCPT and SBPT can be attributed to anisotropy of ground.

Figure 13 presents the variation of the shear modulus of Bothkennar clay obtained from CPMT and SBPM, normalized with that obtained from SCPT, versus depth below the ground surface. The degree of differences in the results from different types of testing can be attributed to the contrastive response of ground towards each type of test. Thus, unlike most engineering materials that have unique engineering properties such as stiffness and strength, ground and soils exhibit behavioral responses that depend on the kind of probing carried out, whether in the vertical or lateral or radial direction, small or large magnitude, etc.

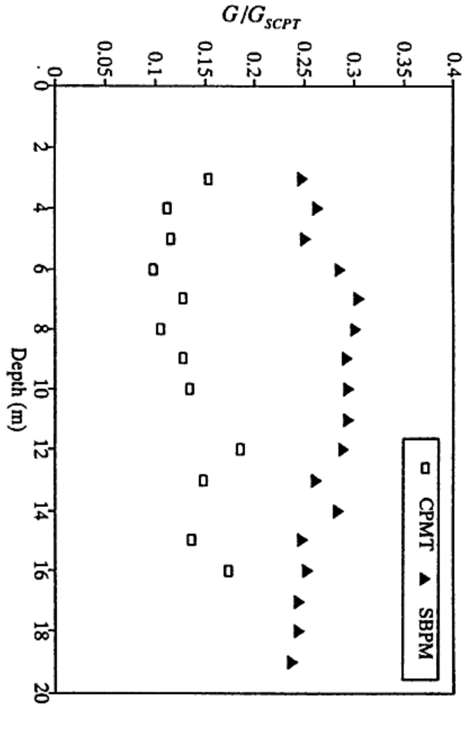

Fig. 13 CPMT and SBPM shear moduli of Bothkennar clay normalized by SCPT moduli [8].

Figure 14 illustrates the possible soil models associated within their applicable strain range. At shear strains less than about 10^{-5}, soil response is essentially linear elastic. From about 10^{-5} to 10^{-4}, the response can be approximated to be non-linear elastic. The stress-strain response within this region remains stiff but highly non-linear and stress path dependent. Creep and rate phenomena generally play a minor role in this region and unload-reload cycles exhibit some hysteresis but plastic strains are generally small [9]. However, beyond shear strains of about 10^{-4}, plastic strains come into the picture as the stress state approaches the yield surface in stress space. As soon as the stress state reaches the yield surface, soil experiences plastic deformations. The selection of the appropriate soil constitutive model is a function of the design problem and the project requirements. It may not be always feasible to apply complex and sophisticated elasto-plastic soil models with features such as kinematic hardening to small geotechnical problems where budgets are limited. Hence, there involves some judgement on the appropriate constitutive model to use depending on the type of soil and the anticipated shear strains induced in the soil by the structure.

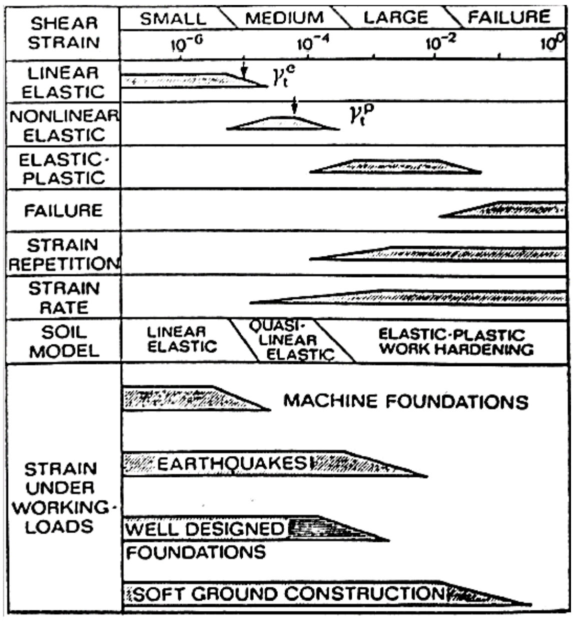

Fig. 14 Possible soil models and associated strain ranges [9].

Figure 15 depicts the variation of normalized stiffness G_{ur0}/G_0, the ratio of the stress-normalized stiffness G_{ur0} to the small strain stiffness G_0, versus the shear strain of Thanet sand. From Fig. 15, it is clearly evident that the shear stiffness of soil is not a constant or unique value but depends on the magnitude of shear strain. In addition, the shear stiffness of soil is a function of the load cycles, which leads to shear modulus degradation.

Uniqueness of modulus, expected of most engineering materials such as steel and concrete, does not hold good for soils. The response of soil is a function of the degree of straining and the number of times it is loaded and unloaded.

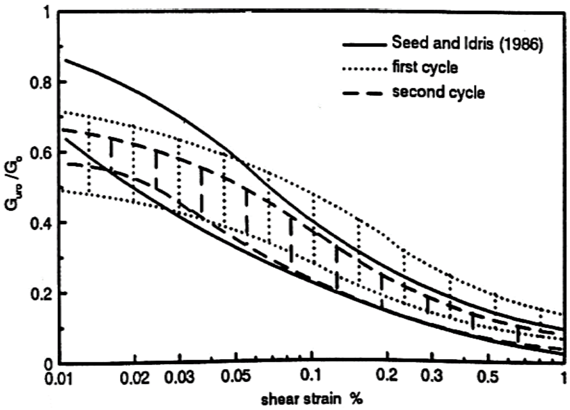

Fig. 15 Variation of normalized shear stiffness with shear strain for Thanet sand [10].

SHEAR WAVE VELOCITY

Another interesting aspect of ground behavior is illustrated in Fig. 16 wherein measurements or response of ground from cross-hole testing, in which the wave travels laterally or horizontally, are compared with that from down-hole testing in which case the wave travels vertically downward.

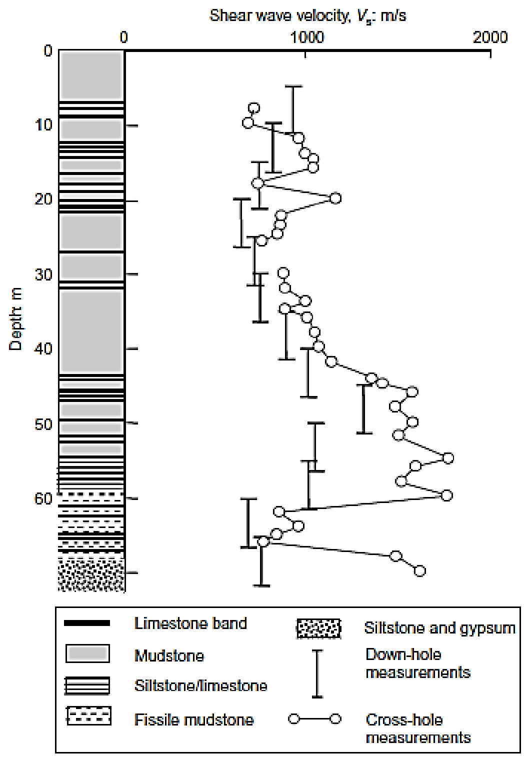

Fig. 16 Down-hole and cross-hole seismic shear wave velocities in layered ground [11].

The former measures the shear wave velocity in the horizontal direction over a finite distance while the latter provides a similar but distinctly different response in the vertical direction. Congruence should not be expected between the results from cross-hole and down-hole testing unless isotropy of ground is expected or presumed. Ground is inherently anisotropic and the same is reflected in the shear wave velocities measured in two orthogonal directions.

COEFFICIENT OF LATERAL EARTH PRESSURE AT-REST

Figure 17 shows the variation of total horizontal stress with depth in Thanet sand. The coefficient of lateral earth pressure at-rest K_0 is a state parameter that illustrates the condition of the ground, whether it is normally or over-consolidated. A K_0 value of 0.5 implies close to being normally consolidated while values of 1.0 or 1.5 clearly correspond to an overconsolidated state. With the kind of response observed in Fig. 17, it becomes difficult to identify clearly whether the ground is in normally or overconsolidated state. Therefore, one has to use judgment to predict its behavior based on one's experience.

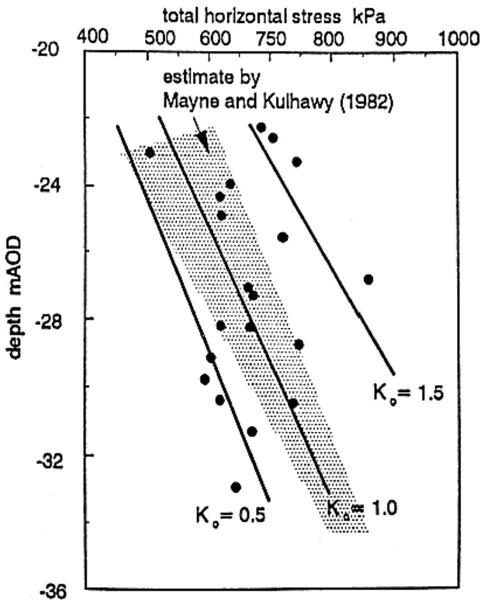

Fig. 17 Variation of total horizontal stresses with depth in Thanet sand [10].

PRACTICES OF MEDICINE AND GEOTECHNICAL ENGINEERING

Comparison of Human Body and Ground

Several similarities can be drawn or observed between the practices of medicine that deals with the human body and geotechnical engineering that deals with the ground. Firstly, both human body and ground are not manufactured to specifications, although cloning (with identical genetic footprint) is possible in the case of living organisms. Secondly, both the human body and the ground have evolved over long periods of time, by natural evolution in the case of the former, and by geological processes in the case of the latter. Table 3 compares and contrasts a human being with ground. A human being has the usual set of organs, limbs, bones, and muscles. While these features are the same for most human beings, however, each is very different from the other because of genetics, pedigree, upbringing, parental care, and social environment. The personality of the individual is the most distinguishing characteristic of a human being apart from of course some physical features. Thus, there are people who are extroverts or introverts, with traits such as sad or happy, angry or jovial, friendly or misanthropic, helpful or neutral or unhelpful, with positive or negative attitudes.

Table 3 Comparison of human body and ground

Human Body	Ground
Eyes, nose, ears, organs, bones, muscles	Different strata, soils – properties/characteristics
+	+
Genetics/DNA	Formation, geology
Environment	In-situ conditions
Personal history	Past history of site
Mood changes	Water table fluctuations
Evolution with age	Thixotropy
Stimuli	Stress/strain path
=	=
Behavioral response	Behavioral response

Diagnosis and Treatment

Humans consult a doctor for various purposes: (1) a general checkup, (2) to get treated when ill or sick, and (3) to get vaccinated as a preventive measure against viruses. The practice of medicine involves: (1) diagnosis and (2) treatment. The latter consists of both prophylactic (preventive) and therapeutic (curative) measures. In addition, general fitness, social and preventive medicine, and sports medicine, are practiced in order to help athletes recover from injuries and enhance their performance. Table 4 presents diagnostic parallels between the fields of medicine and geotechnical engineering.

In medicine, the diagnosis begins with a qualitative and simple examination of the physical features, such as eyes, tongue, skin, and chest of the patient. The doctor may enquire about the patient's family background, environment, history of previous illnesses, and then performs some index type tests on the patient such as height, weight, temperature, blood pressure, sugar, and pulse. Conventional pathological or radiological (X-ray) tests may be suggested if warranted. Modern day medical practice is relying more on advanced investigations such as ultrasound, computerized axial tomography (CAT) scan, magnetic resonance imaging (MRI), which are non-invasive but provide a very detailed and reliable picture of a patient's inner vitals.

Table 4 Diagnostic parallels

Item	Medicine	Geotechnical Engineering
Background	Patient's history, family background, environment	Site history, geology, adjacent structures
Qualitative examination	Visual, eyes, tongue, skin, chest	Reconnaissance, surface features, water table
Quantitative tests	Height, weight	Atterberg limits, GSD, clay content, mineral type
State parameters	Temperature, pulse, blood pressure	Relative density, liquidity index, OCR
Routine tests	Pathological, X-ray	Permeability, consolidation, shear tests; in-situ tests such as SPT, CPT, vane shear
Specialized tests	ultrasound, CAT scan, MRI	Piezocone, pressuremeter, dilatometer, SASW

On the other hand, a geotechnical engineer given a job first undertakes a reconnaissance survey of the site and tries to gather information related to the history of the site and adjacent structures. The geotechnical engineer then collects few soil samples either by hand auguring or by making a trial pit, and runs index tests such as grain size distribution and Atterberg limits for identification and classification of soil type. As part of the detailed investigations, the so-called 'undisturbed' samples are collected, taken to the laboratory and tested for strength, compressibility, hydraulic conductivity and stress–strain response under different loading paths. Since extracting truly undisturbed samples is near impossible, in-situ tests such as standard penetration test (SPT), cone penetration test (CPT) and vane shear test are conducted to evaluate the in-situ characteristics of the ground. The penetration resistance of each soil layer obtained from the SPT or CPT can be correlated with soil strength parameters for use in design. With modern day advances, the piezocone, pressuremeter, dilatometer and spectral analysis of surface waves (SASW) tests may be carried out to obtain more reliable characteristics of the ground. Geotechnical engineers often have to comprehend difficult ground conditions and deal with the consequences of unforeseen damages that arise either during construction or during the performance of the structure. Here in comes the ability to diagnose the causes that lead to these situations. The Leaning Tower of Pisa is a classic example of one such instance [12]. After several investigations by a large number of experts has it been possible to remedy the situation centuries later.

Problems

Several similarities exist between the problems faced by doctors and geotechnical engineers (Table 5). Genetically, some people have a tendency to be obese while some others develop anorexia, a problem similar to expansive soils and soil shrinkage. Giddiness is somewhat similar to instability, epilepsy to liquefaction, fatigue to strain softening under cyclic loading, high blood pressure to high pore water pressure, prostrate and urinary problems to drainage, cancer to contaminated ground and groundwater.

Table 5 Problems in medical and geotechnical practices

Medical Problem	Geotechnical Problem
Obesity/Anorexia	Swelling/Shrinkage
High blood pressure	High pore pressure
Fatigue	Degradation under cyclic loading
Giddiness	Instability
Epilepsy	Liquefaction
Fracture	Brittle failure of stiff soils/rocks
Prostrate/Urinary	Drainage
Cancer/AIDS	Contaminated ground

Solutions and Comparative Practices

It is therefore not difficult to draw parallels in dealing with many of the ailments of diseases and the solutions practiced by geotechnical engineers (Table 6).

Table 6 Similarities in practices

Medical Practice	Geotechnical Practice
Bypass surgery	Vertical drains
Vaccination	Preloading
Physiotherapy	Heavy tamping
Transplants	Inclusions, e.g. granular piles/stone columns
Dialysis	Electro-osmosis
Transfusion	Grouting
Orthopedics	Nailing
Chemotherapy	Remediation of contaminated ground
Surgical removal	Excavation/soil extraction

Bypass surgery or insertion of stents into the arteries of the heart allows increased blood flow from the heart to the other parts of the body. Similarly, vertical drains are provided to accelerate consolidation and increase the flow of water through fine-grained soils. Physiotherapy is a physical medicine and rehabilitation specialty that remediates impairments and promotes mobility through fitness and weight training programs. It is somewhat akin to heavy tamping which involves dropping a heavy weight from a large height on top of loose granular soils to improve their relative density. Surgical removal is analogous to soil extraction, a technique used to stabilize the Leaning Tower of Pisa [12].

Organ transplantation involves moving of an organ from one body to another to replace the recipient's damaged or absent organ. Granular piles/stone columns perform a similar function by replacing soft/weak ground with granular material having higher shear resistance. Orthopedics deals with the strengthening of deformities or functional impairments of the musculoskeletal system, which is somewhat akin to soil reinforcement by nailing or geosynthetics. Vaccination uses a mild dose of antigenic to increase body resistance against viruses while soft ground is preloaded to withstand regular structural load after the removal of surcharge. Chemotherapy, which is used to treat cancer, is comparable to remediation of contaminated ground.

Observational Method

In medicine, the doctor, after diagnosing the precise medical problem of the patient, prescribes medicines along with a medication timeline for the patient to follow. Further, the doctor requests the patient to visit him/her after a certain number of days in order to assess the response of the patient towards the prescribed medication. If the medicines work well and the patient has fully recovered, then the doctor terminates the treatment; if not, then the doctor either increases the dosage or prescribes a different medicine for the patient. Similarly, in geotechnical engineering, pre- and post-treatment responses of ground from the prescribed ground improvement technique are compared.

Figures 18 to 20 compare the SPT *N*-values of 15 to 20 m thick reclaimed sandy soil at Port Island in Kobe City, Japan, before and after treatment by vibro compaction, sand drains, and preloading with sand drains, respectively. The seabed at the site was comprised of a 10 to 20 m thick soft alluvial clay layer. The dotted lines and solid lines show the *N*-values before and after treatment, respectively. The *N*-values of unimproved ground were of the order of 10 or less, but after ground improvement, the *N*-values increased upto as high as 30. SPT *N*-values greater than 30 should be treated with caution due to presence of large cobbles at those depths [13].

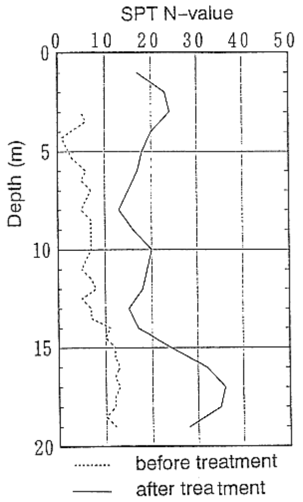

Fig. 18 SPT *N*-values of ground before and after treatment by rod (vibro) compaction method [13].

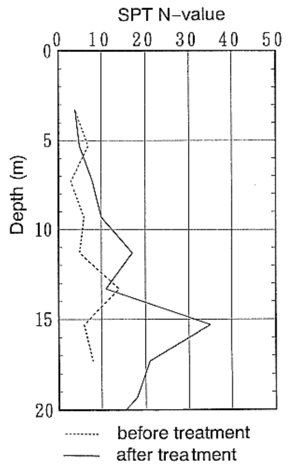

Fig. 19 SPT *N*-values of ground before and after treatment by sand drains [13].

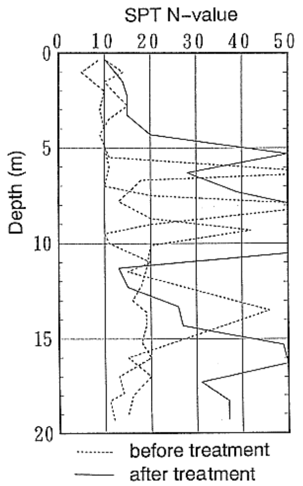

Fig. 20 SPT *N*-values of ground before and after treatment by preloading with sand drains [13].

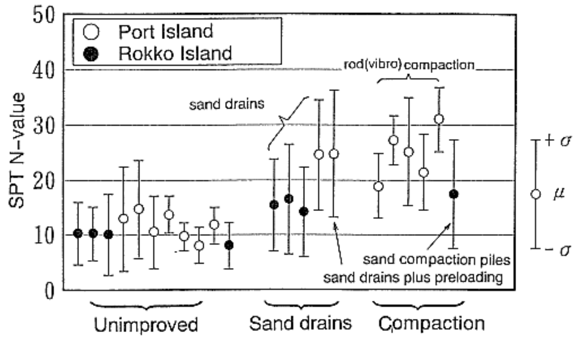

Fig. 21 Comparison of SPT *N*-values of ground before and after treatment at Port Island and Rokko Island, Japan [13].

Figure 21 compares the SPT *N*-values of ground at Port and Rokko Islands, Japan, before and after treatment by sand drains, preloading with sand drains, sand compaction piles and rod (vibro) compaction. The average SPT *N*-value of untreated ground is about 10 while the *N*-values of ground treated by sand compaction piles, sand drains plus preloading and rod (vibro) compaction are 18, 25 and 31, respectively. The apparent increase of *N*-value of the reclaimed sandy soil layer due to installation of sand drains was attributed to extra vibration needed to advance the casing down to the alluvial clay layer [13]. However, the SPT *N*-value improves significantly (by almost three-folds) for vibro compacted ground due to densification of sand by vibrations.

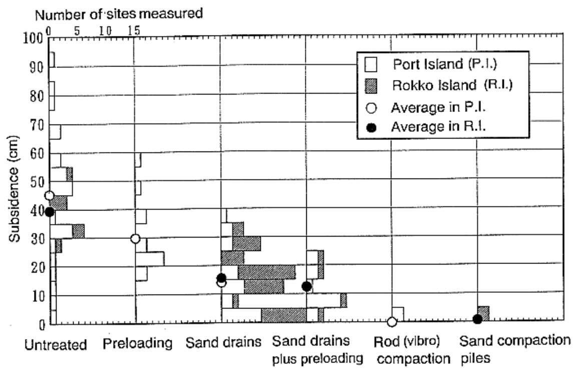

Fig. 22 Comparison of ground subsidence in zones treated with different methods [13].

Figure 22 compares the ground subsidence caused by the 7.2 magnitude Hyogoken-Nambu earthquake in 1995 at Port Island and Rokko Island before and after ground treatment by different methods. The average ground subsidence in the untreated zone was 40 to 45 cm (due to liquefaction) while that in the zones treated by preloading, sand drains, and sand drains plus preloading was 30 cm, 15 cm, and 12 cm, respectively. No ground subsidence and no damage to structures were reported in the zones densified with sand compaction piles and rod (vibro) compaction despite strong ground shaking with maximum surface acceleration of more than 400 cm/s^2.

Major Differences

While there are several parallels between the practices of medicine and geotechnical engineering, there are, however, some major differences:

1. In medicine, the patient goes to a doctor, whereas in geotechnical engineering, the engineer has to go to the site to diagnose the problem.
2. The patient talks to the doctor, whereas a geotechnical engineer listens to the ground.
3. The failures of doctors are often buried or cremated in the ground, whereas the successes of geotechnical engineers get buried while the failures show up glaringly.
4. Doctors are paid better than geotechnical engineers.

In sum, is a geotechnical expert, a doctor, an engineer, a psychologist, a clairvoyant, or all of the above? But most of all, an artist like Terzaghi may be. On that note, it is suggested that geotechnical engineering is a science but its practice is an art.

CONCLUDING REMARKS

The purely mechanistic view that postulates that materials have unique and determinable properties does not adequately describe the response of soils in general and ground in particular. The gross unpredictability of the ultimate capacity of a drilled shaft and the horizontal displacement, surface settlement and anchor forces of a deep excavation attest to the fact that ground is not a ‘material’ with unique determinable ‘properties’. Instead, it exhibits behavioral response somewhat akin to entities that respond to stimuli. The paper presents examples to illustrate the above premise that a simple mechanistic view is inadequate and a paradigm shift is needed in geotechnical engineering to understand and predict the response of ground for different engineering activities carried out for enhancing the quality of life. A parallel has been drawn between the fields of medicine and geotechnical engineering. It is illustrated that soils in general and ground in particular can be examined, evaluated and understood from a framework similar to that used for examining human beings.

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