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Opening Lecture

SITE INVESTIGATION AND FOUNDATION DECISIONS FOR OFFSHORE STRUCTURES

by
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SITE INVESTIGATION AND FOUNDATION DECISIONS FOR OFFSHORE STRUCTURES

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Synopsis.

Principal problems faced by geotechnical engineering for offshore structures are considered in the light of recent developments in site investigation techniques. Satisfactory designs have been highly dependent on empirical interpretations. High costs of investigation impose quests for generalizable procedures for establishing design parameters, for minimizing excessive site specific studies. One questions, however, if much of the scatter in current generalizations is not intrinsic to oversimplifications typically implicit in the correlations used.

1. Introduction.

So fantastically vast are the perspectives, investments, and responsibilities associated with submarine geotechnical engineering for offshore structures, and so rapid have been the advances, that the very thought of attempting to embrace the subject of site investigation and foundation decisions in this field within a single lecture seems unpardonably pretentious.

The excuse arises, however, from my belief that our engineering investigations and conclusions are always nominal, aimed at a specific purpose and hopefully achieving it: and therefore if there is any question or lesson to be drawn from or for a given geotechnical application, we cannot avoid considering the global problem. Thus the depth of attack of partial problems will be inevitably relegated to a subsequent stage.

It is as occurs in engineering practise, in going from early feasibility studies all the way to the final designs of construction details: we must funnel from general to particular, without losing the linkages with collateral fields. All aspects should deepen in sophistication and precision in a reasonably comparable manner, because it is indispensable in a complex engineering system or project to avoid some links of the chain being significantly weaker than others. At all stages of advancing design decisions the Factors of Safety FS are presumably maintained constant; and, therefore, the additional costs in delving deeper into any given sector should resist questioning regarding benefit/cost ratio, by promoting more economical solutions at the same FS.

2. Warning to geotechnical engineering.

The benefit/cost considerations are always particularly applicable to geotechnical engineering for many a reason, well recognized by ourselves, but sometimes forgotten by our clients who would eagerly bury and forget foundation problems. Foundations are one principal link that acts as an insurance premium on the entire superstructure investment. And, in considering design decisions, risks and factors of safety on foundation problems, one must consider not merely the above heavy responsibility, but also some factors that will be discussed with regard to conscious choice of FS values, that is: (a) the occasion (inopportune vs. easily prepared-for and supported) when the critical condition might occur; (b) rate of onset of critical factors; (c) rate of outcomes of critical responses; (d) what chances there will be for adjustive or corrective actions when the critical behavior might require them (Design Principles DP4 and DP5, de Mello, 27). It seems that in all these respects the geotechnician's problems might occur under comparatively most unfavourable circumstances.

Our specialist colleagues have supplied solutions meeting the challenges in the entire broad spectrum. It seems to me, however, that there are a couple of general observations worth my emphasizing from the start.

So rapid have been the increased demands, that despite the exponentially progressing theoretical capacities available in up-to-date geotechnical engineering, as Azzouz et al.02 state "Procedures commonly employed in the design of offshore structures generally have a high degree of empirical content, regarding both the methods of evaluating soil properties and the techniques of analysis". The use of empirical design procedures presupposes that near--critical conditions have been satisfactorily met, and that the conditioning behaviors will be suitably extrapolated all the way to the critical "once-never" condition for which the design is presumed prepared.

The second warning is that if we continue presumably fettered by too high a degree of dispersion and indeterminancy, there will be a tendency for collateral fields to create and develop solutions that are independent of our dispersions, and therefore independent of refinements of geotechnical engineering. We will be set aside by UMBRELLA SOLUTIONS that satisfy minimum common denominator conditions, and there will not be many to mourn our demise as useless, since those responsible for the superstructures and their visible and controllable operations have always considered it an irking nuisance to have to bury in the foundations the earliest heavy first costs.

The fact is that whereas Nature and natural factors (including geotechnical) tend to survive on the brink of natural selection at close to FS ≥ 1.00, it is in the nature of creativity (artistic or technological, inventive) that successful solutions are only embraced if they have innate capacities much higher than immediately required (i.e. FS >> 1). Thus what we observe in the modern rapid development of new fields is not surprising, but quite explainable. The areas which literally had to start from scratch, and were unfettered by accepted practices and the confidence of knowledge, gave a leap forward.

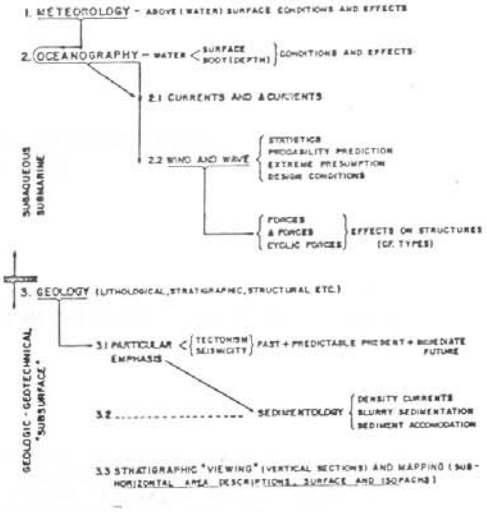
One cannot surmount the psychological reality that when one studiedly com--prehends (grasps together) something, one is supposed to UNDER-STAND it, and unfortunately not, as could well be the case, OVERSTAND it.

With these introductory remarks, I shall begin directly the comments on the attempted overview of the overall problem, hopefully sequential, but definitely concentrated on geotechnical solutions, analyses, and questions.

3. Fundamental intervening fields and data.

Chart I summarizes the well-known

CHART I CIVIL ENG'G. IN OFFSHORE STRUCTURES



general areas of intervening data, with the exclusion of obvious additional effects to be considered in certain areas, such as ice, impact from floating bodies, etc...

The intent is to maintain perspective, but to concentrate on the areas GEOLOGIC-GEOTECHNICAL, termed "subsurface", under item 3. GEOLOGY. Brief mention could be made of problems of seismicity (3.1) and sedimentology (3.2) within the text on other items, but our interest really begins with stratigraphic "viewing" and mapping.

Conventional geophysics itself has made tremendous progress in the past decade, as witnessed, for example, by many papers presented at the International Symposium on Soil and Rock Investigations by in Situ Testing (e.g. Bjelm, L. et al. 05) and, simultaneously, the use of HIGH RESOLUTION GEOPHYSICS has proved itself a principal tool for first phase detection of the slightest discontinuities in seabed stratigraphic description. Figs. 1 and 2 serve as mere examples, the latter of high resolution geophysical results.

There is a wide range of exploration and engineering applications of a complete analysis of high-resolution geophysical data. Generally acoustic, but judiciously employing more than one acoustic device of varying frequency responses, to obtain more complete "pictures" from the water surface to several hundreds of meters below the seafloor. The high-frequency (12-80KHz) depth sounder (FATHOMETER) and dual channel side scan (38-250KHz) SONAR systems are of low energy, with primary purpose of "resolution" rather than "penetration", thereby displaying the seafloor profile because of the significant acoustic interface. Meanwhile

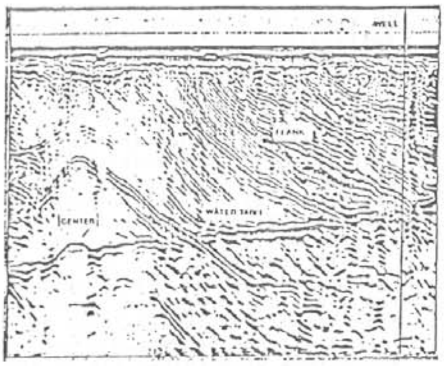


Fig.1 -Detailed geophysical diagraphies (on land).

various lower-frequency systems transmit energy that partly penetrates the subbottom and detects and depicts several interfaces. An interface depends on the contrast in acoustic impedance between the two consecutive materials: interfaces are displayed graphically based on time of to-and-from travel of transmitted sound energy from the source to each interface, actual depths being dependent on the density (varying with depth) and thus subject to interpretative adjustment by experience and by correlation with borings. One might list, among others, such acoustic subbottom profilers as:-

TUNED TRANSDUCERS, 3.5-7KHz; Subbottom penetration to 30m: bubble detection.
ELECTROMECHANICAL (ACOUSTIPULSE), 0.8--5KHz; penetration to 135m, best resolution of shallow active and non-active gas-charged zones.
OPTICALLY STACKED SPARKER, 0.04-0.15KHz; penetration to 1000m, better horizontal resolution.
FAST FIRING SPARKER, 0.04-0.15KHz; penetrations to 350 or 1000m. Excellent horizontal and vertical resolutions of the sediment and geologic environment; recorded on magnetic tape. These sparker units employ a principle of firing rate of pulses versus energy output.

It is obviously quite beyond the intent of this presentation to consider details of such sophistications of geophysical profiling and mapping, aided by computorized resolution and visual depiction. The main point that arises is if such taxonomic differentiations are not too good on the one extreme, or too poor at the other, for present geotechnical engineering capacity to use them effectively. Are some of those depicted details of foundation engineering significance? If so, what? Are there statistical correlations of such sug-

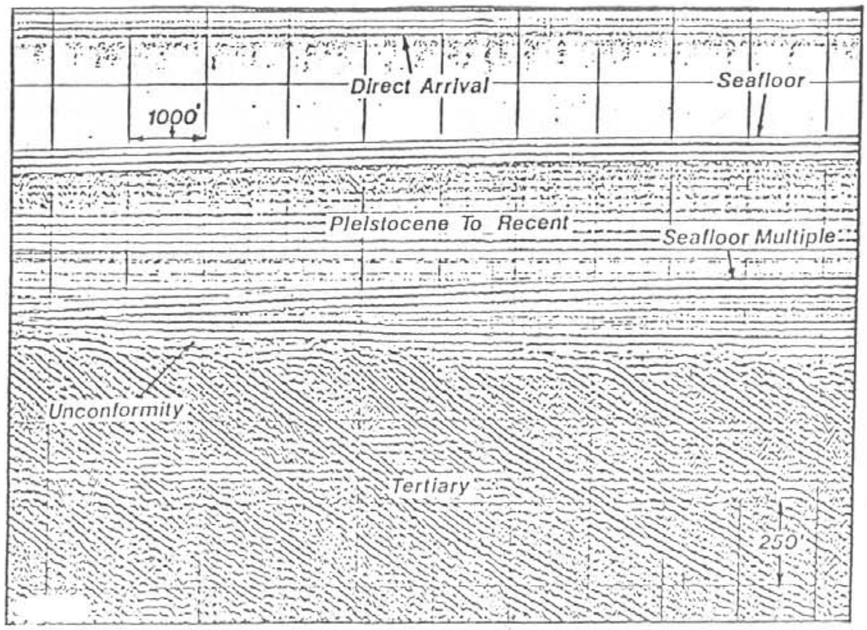


Fig.2 -Geophysical diagraphies below seafloor.

gestively differentiated profiles with the conventional classifications, index tests, and fundamental tests with which our foundation experience is associated?

In general one finds that this phase of investigation furnishes useful qualitative indications for further geotechnical investigation (e.g. Carpenter G.B. and McCarthy, J,C., 10), Fig. 3.

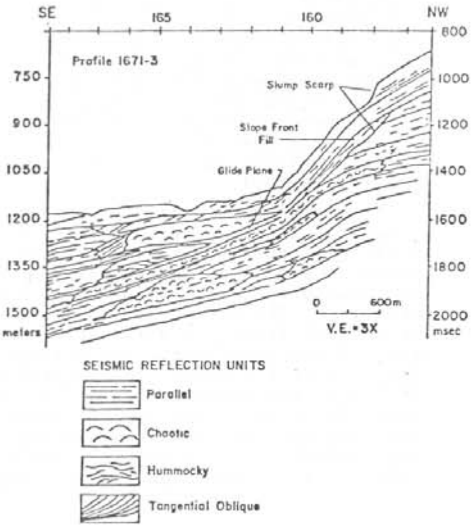


Fig.3 -Example of geophysical interpretation.

Once the above considered "super-mega" level of geomechanical behaviour conditioning, which is that of the geologic context, has been set aside in comforting reassurance, we should consider the suggested three levels (micro, macro, and mega) of geotechnical behavior investigation (RNESE, 35). These however, tend to be so expensive for offshore structures, that it is of interest to consider the applicability of the parameters sought, with regard to specific structures and problems, since engineering investigations must be vectorial, for a purpose.

. Types of offshore platform designs, and activities requiring geotechnical engineering.

I have repeatedly emphasized that engineering creativity first visualizes the physical models destined to serve the purpose envisaged (Engineering ≡ ≡ Physics + Common Sense), and forthwith endeavours to submit the body created to quantifiable analyses and subsequent adjustments. Offshore engineering proves to be no exception.

In order to focus on geotechnical investigation problems and solutions, I have collected in Fig. 4 some of the principal types of offshore platforms successfully devised and used (Hoeg, 20 Kure and Teymourian, 24). There is no presumption of being thorough or fair with regard to the multitudes of papers on the subject, and omissions are regretted: the intent is merely to summarize some of the principal sources of problems faced.

Quite comprehensibly in shallower waters and special conditions, a solution sometimes preferred involves the artificial sand island (Fig. 5a, Tilmans,44 and Boone, 08); and in some situations of marginal fields and somewhat less permanent platforms (inclusive for drilling and test-exploration) the type of monotower multipurpose platform (e.g. Condeep) has found application (cf. Fig. 5b, Kure and Teymourian, 24).

Each of the above structures creates somewhat different demands on geotechnical investigation and design. It is worthwhile, however, to begin by emphasizing a broader spectrum of requirement. Fig. 6 reproduces an often repeated (e.g. Selnes 41) summary of principal construction activities and structural units requiring geotechnical engineering. We emphasize the great conceptual difference between the needs of investigation for a localized structure of highly concentrated load and investment per square meter, and for the pipeline problems, of transportation of the high investment value but without any possibility of investigation of the shallow profiling with adequate benefit/cost ratio for the minimum details to which the geotechnician is accustomed. Furthermore, for the case of sand-islands, the dominant problem areas lie in the field of ocean hydrodynamics, the geotechnique being somewhat limited to surface protection and special conditions of stability despite susceptibility to cyclic liquefaction.

5. Generalities on site investigation and multiple profiling.

Soil investigations for offshore installations are becoming increasingly complex and costly. The total cost of an investigation is more dependent on the time aspect than any other single factor. Besides the shallow-water solutions (fill, island) or multipurpose drilling platforms, the recourse to drillships is most common. Two basically different types include the "conventionally anchored", and the much larger "dynamically positioned" drillship. Average efficiency is quoted as 0.6 hours per meter, including all variables; the dynamically positioned drillship appears better, especially for greater depths; the important point is

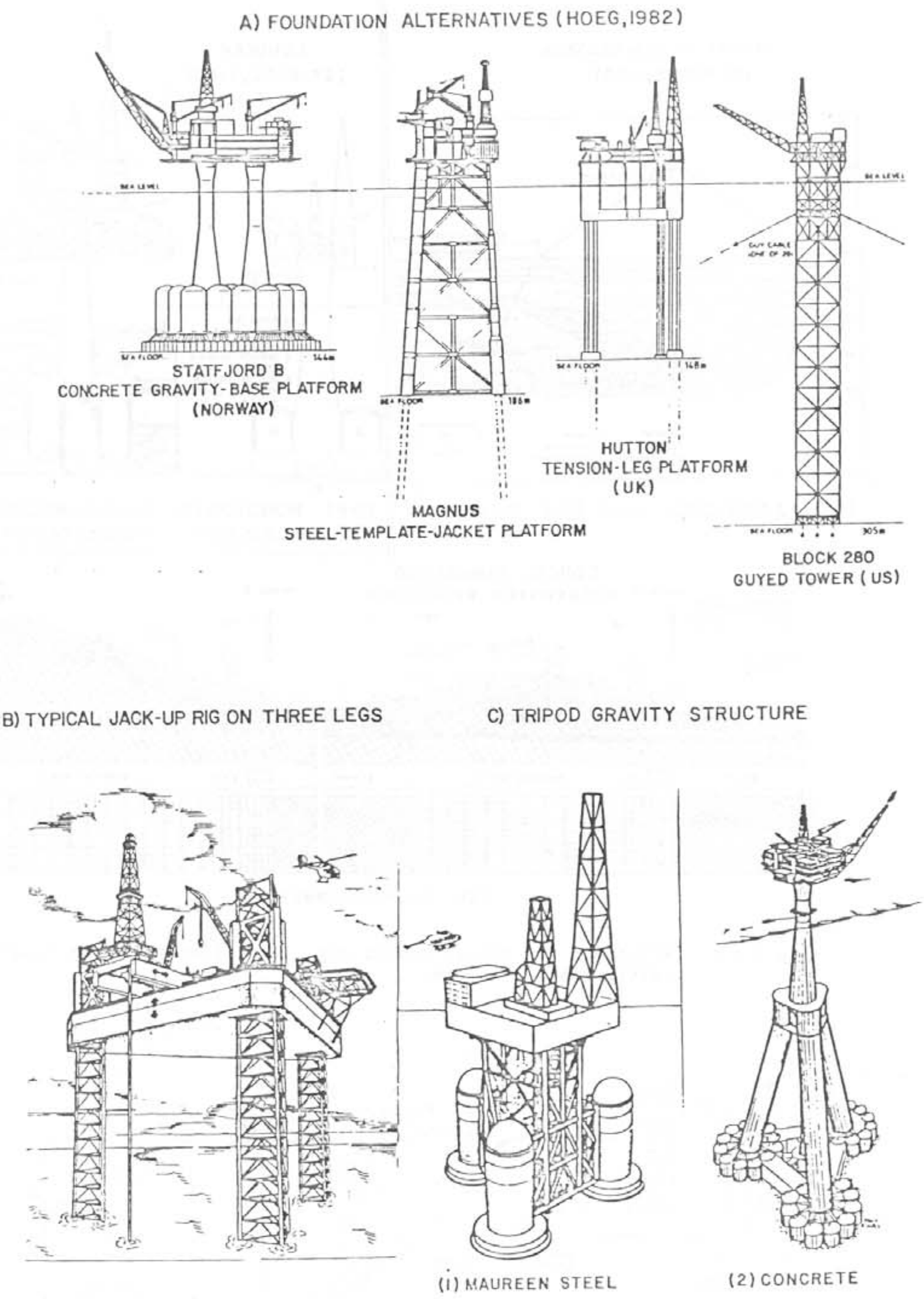


Fig.4 -Principal types of platforms.

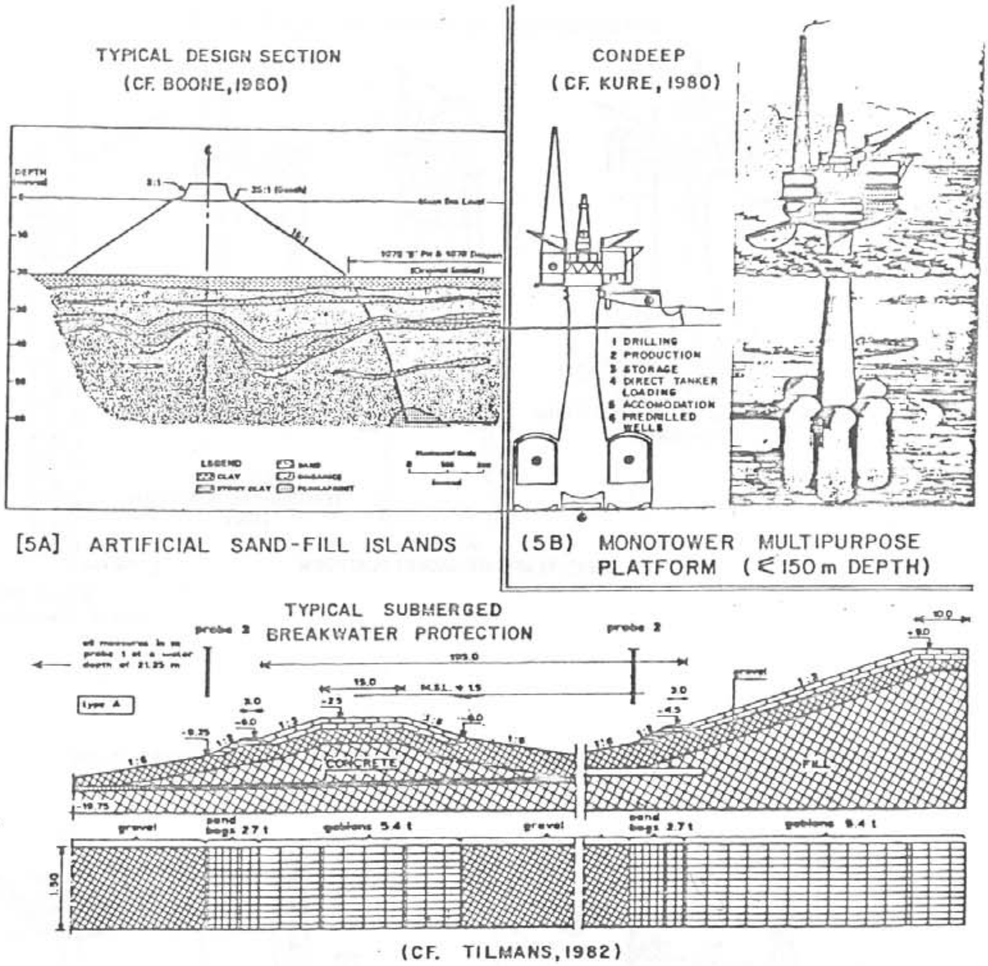


Fig.5 a,b - Solutions for modest depths etc. sand-islands and monotower multipurpose platform.

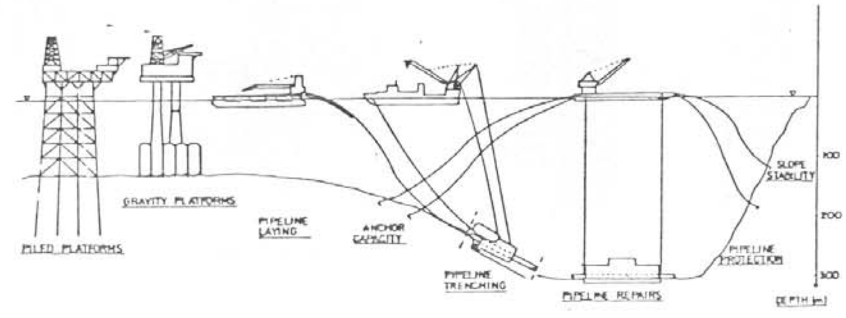


Fig.6 -Principal construction activities requiring geotechnical engineering.

that roughly 85% of the time is used in drilling, sampling and in-situ testing, alongside with collateral inevitables of pipe and equipment handling and stand-by time due to in-situ test equipment.

On the one hand, there have been developments of seafloor test equipments (e.g. the CPT, as in the case of Fugro's Seacalf, and McClelland's Stingray, and the SPT seafloor chamber, cf. Bogossian and Dias Machado 07). And, on the other hand there has been an aggressive increase in maximizing the use of each borehole by multiple profiling.

The great potentialities of simultaneous multiple profiling by penetrating probe-sensors coupled to computorized statistically correlated classifications has been repeatedly pointed out (e.g. de Mello, 29). The example of the successes in the field of health and sanitation, wherein mostly "non-destructive observations" are available, and hundreds of uncontrollable factors are at play, beckons us to optimism along this line. The main distinction is, however, that the parameters of behavior (health vs. anomaly) with which to correlate are nominally taken as better defined (by multitudinous usage and conventional acceptance); as will be pointed out forthwith, our greatest problem lies in the very degree of questionable credibility of the geotechnical parameters (conventional, rapidly varied with time and place) against which we would establish the multiple regressions.

One point, already mentioned, that persists, is the tremendous disparity between the use of vertical vs. subhorizontal (multiple) profiling. Since we accept that horizontal variations tend to be minimal within the limited widths of interest, and, in particular, since we tend to use automatically the linear interpolation for strata between borings, the future use of subhorizontal profiling seems highly recommended to detect the "statistical dispersion" that pertains strictly to the test and not to the stratum. Incidentally, for shallow borings on land it is often recommended that such research be based on repeating some borings at very close spacing.

Procedures employed for geotechnical computations in support of design choices for offshore structures have a high degree of empirical content, both regarding methods for evaluating soil parameters, and for techniques of analysis. However, the latter part seems to be comparatively free for rapid revisions and adjustments, for many obvious reasons: its basic limitation is tied to the "mega level" of investigation, which comprises back-analyses of case histories, wherein the limitation is, once again, the judgement on soil parameters. Thus, we must concentrate on the questions of geotechnical decision on design parameters, and recognize the need to minimize the site specific additional exploration and testing for each case because of costs and time, and because of the limited number of samples taken, and resulting acceptably undisturbed.

Within the concept of maximizing the data and generalizable interpretations that may be extracted from in-situ tests and boring-plus-sampling, there have been many efforts at improved analyses of behaviors at the micro-level (for instance, fabric of clays, influence of pore-water chemistry, etc..); these will not be broached except to point out when our macro-level testing has visibly disrespected some careful research conclusions from the micro--level. The other approach has been to maximize the use of studies of so-called "normalized behavior", for optimizing conditions of transfer of conclusions and correlations from one site to another. "Macro-level" investigations are related to laboratory test specimens: for so-called "normalized behavior" determination, the testing is oriented attempting adimensionalizing both the testing (stress-strain-time--strength) and, principally, the resulting expressions, as functions of the stipulated dominant variable (e.g. consolidation pressure).

Finally, normalized behavior studies are also concentrated on in situ tests, which represent either a macro-level test, as far as tested volumes are concerned, or a model-size mega-level test, when they are interpreted via model-to-prototype visualizations, since they involve similar questions regarding in-situ boundary conditions as are involved in back-analyses of case-histories.

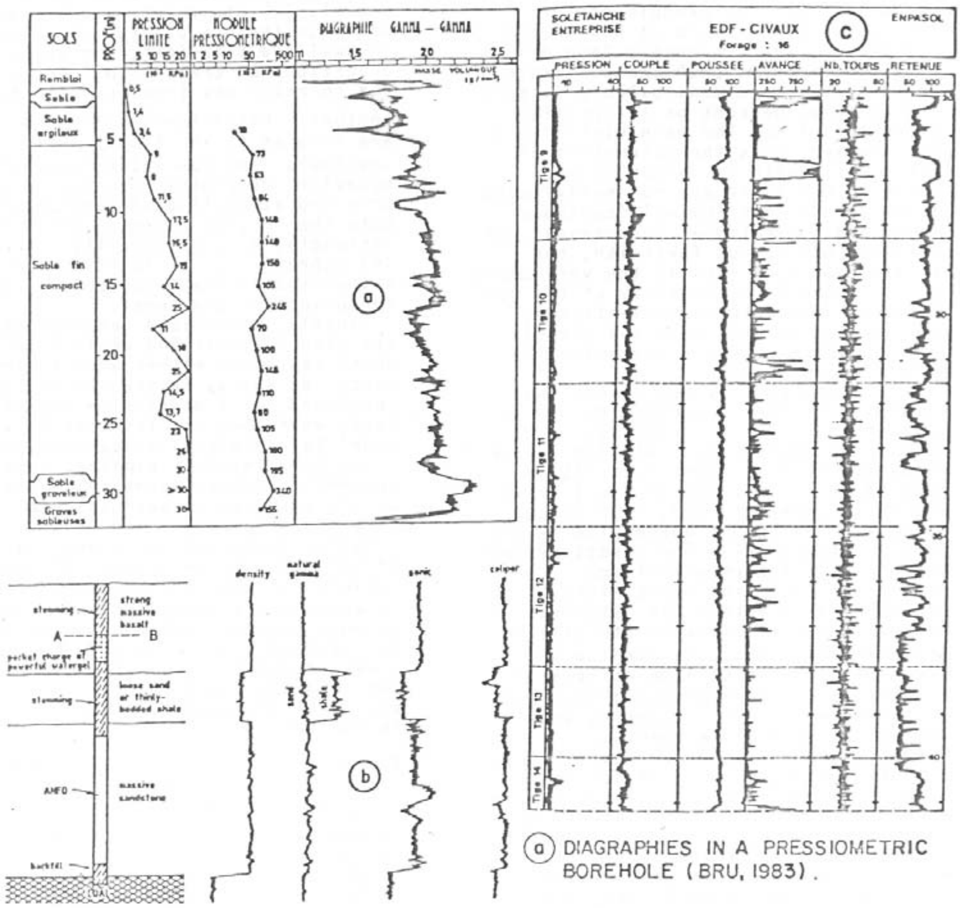
Having discussed the supra-mega level of geologic and geomechanical qualifications, at the macro and mega levels is where our investigations concentrate. We must concede that much is known but not applied with respect to the micro--level demonstrations of how crudely our laboratory and in-situ testing equipment and procedures established themselves.

6. The cycle of experience, and the interference of the dynamics of progress in "normalization processes"

In any back-analysis of the mega-level case-histories, or of macro-level test results to be interpreted by normalized techniques, the rate at which progress has been required of geotechnical engineering has made us fall into some undisputable conceptual errors, which might account for some of the frustating dispersions. The same applies to some of the attempts at more careful interpretations of in-situ tests (model-mega-level tests). The author has mentioned this point so repeatedly that it would seem disrespectful to repeat it (de Mello, 30, 31): the principal points are concerned with:

(a) single-parameter correlations, and often crude parameters;
(b) data taken at face-value irrespective of techniques (in conscious evolution) used as per nominal practices (often prematurely, and sometimes insufficiently, standardized);
(c) adoption of computed nominal overburden stresses, irrespective of possible stress redistribution during differential compressions;
(d) assumed qualities of undisturbed sampling achieved, with poor reference to quantified quality indices.

Fig. 8 indicates schematically the source of concern. Of course, we recognize that in engineering at any given moment we must accept at face-value the test parameters as obtained: thereupon, the eternally nominal nature of engineering data, facts, knowledge. What cannot be understood, however, is that when much later syntheses and theoretical interpretations are attempted, the old publications should be re-employed without any adjustment, even when in the interim research results



(a) DIAGRAPHIES IN A PRESSIOMETRIC BOREHOLE (BRU, 1983).

(b) GEOPHYSICAL DOWNHOLE LOGS AND DETAILS OF BLASTING DATA (HAGAN, 1983).

(c) EXAMPLE OF DIAGRAPHIES OF CLASSICAL PERFORATION PARAMETERS (HAMELIN, 1983).

Fig.7 -Examples of multiple profile data analysis.

have proven the need to revise the earlier techniques and consequent results.

Let us consider some examples. We

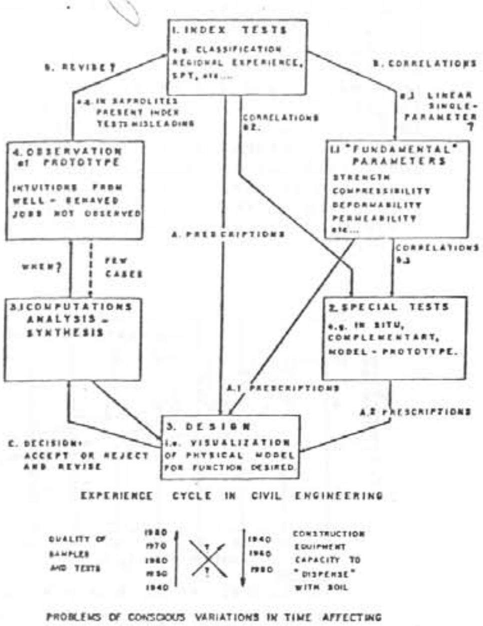


Fig.8 -Experience cycle and possible adulteration with time.

shall assume, however, that at least over the past 30 years all moderately prepared geotechnicians have definitely set aside as wrong, ab limine, the test techniques employing samples air-dried and powdered, an error sufficiently emphasized around 1930.

The blunt question that strikes us is: are we cornering ourselves into investigations of the very investigations, while relegating the practising professional to design prescriptions that our own theoretical knowledge cannot support in good faith? Let us consider the matter principally with regard to clays, with some considerations on sands.

6.1. Clay quality

The index parameter with which most, or practically all, fundamental behavior parameters of clays have been correlated is the plasticity index I_p. Even the most expensive and sophisticated correlations of mega and micro-level data in present day analyses are chronically established as single parameter functions of I_p (for instance, Bjerrum 06, correction factor μ for vane test strength to embankment foundation failure). Now, if we accept the knowledge of the importance of Sensitivity S_t strongly differentiating undisturbed vs. remolded behavior, the first shock should be at the news that undisturbed behavior can even be thought associatable to an index test on totally remolded material. If the artistic quality of a persian rug can be correlated with the quality of the ashes upon burning it, we must have hit upon a miraculous coincidence indeed! In other words, would the S_t of all clays of a given I_p be essentially similar? Or would S_t be directly related to I_p? If any undergraduate were requested to gather all the published pairs of data on S_t vs. I_p, doubtless for a given I_p the S_t variations will turn out to range within $\pm$ 500% around some mean: in any given clay, however, the simplified hypothesis may be tenable.

The second point is almost as shocking. If we do accept some relevance in the Casagrande plasticity chart, we must insist that one needs the pair of coordinates, I_p and ω_L, to define the (remolded) clay behavior quality. What difficulty is there in working out simultaneous regressions on the 2 index parameters? We can understand that in discussing a series of case histories in a given deposit (or similar deposits) one of the 2 parameters becomes unnecessary because the pair tend to be linearly correlated in the plasticity chart: but for scientific self-respect one should insist on the presence of the 2 "independent" parameters, even if in each deposit the coefficient associated with one of them turn out negligible. The fact is that in order of magnitude, for a given I_p the ω_L values can vary about $\pm$ 50% of the value, from high above the A-line (tough clays) to far below it (very silty material), and it would seem a denial of the plasticity chart classification, that such widely differing pairs of parameters should reflect in no differentiation in geotechnical behavior. At least the point should have merited meticulous investigation.

The third point has been emphasized since the early 1950's and is illustrated by the data plotted on Fig. 9 (drawn from Rosenqvist, 37). The interference of salt concentrations in the pore water solution has been repeatedly emphasized, and has been associated with the catastrophic sensitivities and slope failures of the Scandinavian clays. Rosenqvist's study meticulously states (p. 74) "the salt content of the pore water was determined accurately just at the point of the liquid limit and the plasticity limit".

On the other hand for the routine ω_L determinations, was the liquid added a solution of similar salt concentration, or was it merely tap or distilled water? If, as is usual, the test procedure in no way involved maintaining constant the porewater salt concentration, our I_p values are based on tests at inevitably different concentrations: a correction of first degree approximation can be easily made; for instance, in the Rosenqvist data if the clay at about 55% water content were gradually dried to close to the plastic limit, the salt concentration (assumed initially at 15 g/l) would essentially double and the I_p should have dropped from about 30% to 25%. It is not immaterial, especially at low concentrations, and

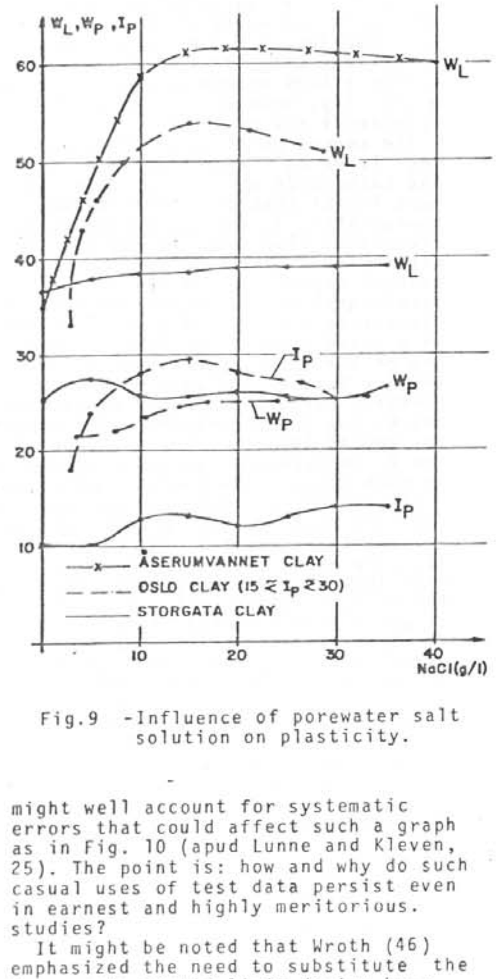


Fig.9 -Influence of porewater salt solution on plasticity.

might well account for systematic errors that could affect such a graph as in Fig. 10 (apud Lunne and Kleven, 25). The point is: how and why do such casual uses of test data persist even in earnest and highly meritorious studies?

It might be noted that Wroth (46) emphasized the need to substitute the Casagrande liquid limit device by a cone penetration device, to avoid conceptual and practical errors implicit in the former (shear strength indication of slope failure much influenced by self-weight, and also impact).

6.2. Phase relations

A modest part of the scatter in analyses of ocean sediments is doubtless due to as simple a problem as the lack of adjustment of phase relation calculations (e.g. Noorany, 33) to take into account differentiated mineralogical grains (e.g. calcareous sands, etc...), different chemical solutions of the pore water, and different gases (organic, etc..) generated and trapped. In common cases the scatter due to these parameters may be modest (say about 10%) but in certain cases in which two or three erroneous tendencies accumulate in the same direction, the error band may well widen 3 or 4 times as much.

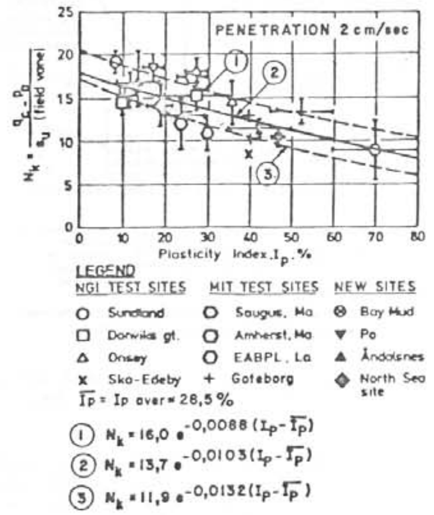


Fig.10 -Empirical cone factor N_k (cf. Lunne & Kleven, 25).

6.3. Quality of undisturbed sampling.

The purpose of all our investigations is to predict the behavior of intact soil elements in situ: therefore one constant preoccupation has been the retrieval of samples as "undisturbed" or "perfect" as possible. Recognizing however, that by principle itself no intact sample can ever be retrieved, the need should be to attempt to quantify somewhat varied degrees of disturbance vs. "perfection", so that, by extrapolations, one might assess the behavior of the fully perfect, "intact", sample. Mention is made, for instance, of "block-quality" undisturbed samples, but in the Mexico 1969 ICSMFE Conference State-of-the-art Report on Foundations the Author demonstrated that in situations of deep excavation stress release and delayed sampling, such as occurred in the Ashford Common Shaft, undisturbed blocks can be much worse than good tube samples.

Besides the technique proposed by Schmertmann, 40 for interpreting oedometer compression curves of different thicknesses to predict the oedometer compressibility of an intact specimen, there have been limited and scattered references to the question. Seldom, if ever, however have publications analysing megalevel investigations of strength-stability, or of deformability-settlements, taken due notice of any necessary adjustment on this count. Fig. 11 reproduces a nominal, practical, method, hypothesized by the Author (Refs. 30,31) based on the shapes of stress-strain curves of routine CU triaxial tests, that had been used for about 25 years in professional practice for establishing a presumed numerical adjustment index for partially disturbed sample quality.

Sophisticated methods based on electron-microscopy, etc. have no quantified indices. Such recent (e.g. Azzouz et al, 02) applications of special radiographic techniques in offshore investigations for distinguishing between "good" and "unsatisfactory" samples, as reproduced in Fig. 12, clearly indicate the samples to be disregarded. The principle is based on detection of lenses (cracks) or bubbles of liberated gas. Unfortunately no dichotomic separation into "good" and "inadequate" establishes an avenue of promise for quantifiable quality indices. At any rate, "visible" cracking by gas liberation is too singular a manner of sample adulteration among the many possibilities.

For improved analyses, and normalized behavior analyses, it is indispensable that one should reduce the really major dispersions that can be due to different qualities of sampling, within an acceptable range.

6.4. In situ vertical effective stress and normalization with respect to the normal preconsolidation pressure.

One of the earliest achievements of soil engineering was the discovery of the importance of the preconsolidation pressure σ'_p as a real "maximum past precompression stress", or nominal stresses representing an analogous discontinuity. The adequate determination and interpretation of the modest discontinuity in behavior of soils, below vs. above the σ'_p value, has become extremely important for "normalized behavior" investigation and definition: the noticeable hysteresis of compression-decompression in clays hinted at easy rational explanations.

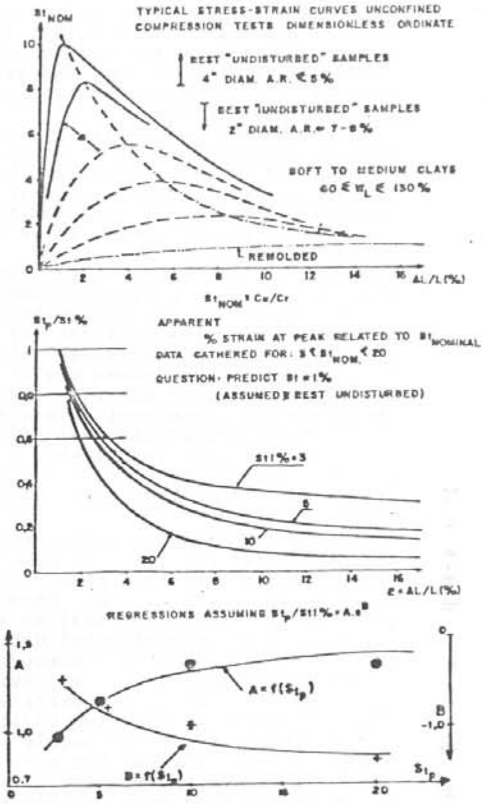


Fig.11 -Quality of "undisturbed sampling" vs partial S_t sensitivity indices.

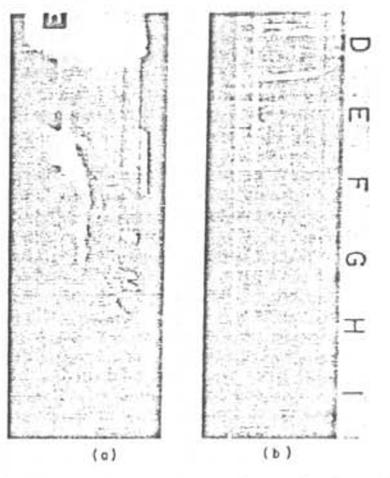


Fig.12 -Clay sample quality insinuated by radiographs (cf.Azzouz et al.02).

As a start the adoption of the semilog plot of oedometric strains vs. log p became an artifice for making the discontinuity more apparent, and detectable by "intersections of two approximately straight lines".

It has become extremely important to know adequately the effective stresses in situ and the σ'_p value, and to compare the two, to determine if the sediment is "normally consolidated", "overconsolidated" (OCR > 1) or "under consolidated".

Several problems have been gradually discovered and investigated, and as a result the originally simple background of assumed stress conditions which served as a basis for undisputed calculations may well have become one of the principal causes of the wide scatter of test data and consequent interpretations. Thus nowadays the area of least cognizance in soil engineering is quite possibly that of in situ stresses, initially assumed as most automatically defined.

Firstly it has always been accepted as automatic that the vertical total and effective stresses have been rigorously known to be γz and $\gamma z - u_\omega$ (hydrostatic). It is inescapable that on an average a soft subhorizontal deposit must respect the absolute homogeneity condition implied in $\sigma_v = \gamma z$: but we do know that if there is any differential compressibility between two adjacent columns there is an inescapable stress redistribution, whereby the more rigid column is loaded by negative skin friction while the more compressible adjacent soil column is ipso facto alleviated. Thus two contiguous soil masses of the macro-level (samples) may have total vertical stresses $\gamma z + \Delta p$ and $\gamma z - \Delta p$. How do compressibility differentiations arise at a given plane? As shown schematically in Fig. 13, many a realistic natural condition will cause them, both deterministically and under statistically random dispersions. We must recall that lateral skin friction (e.g. on piles) reaches peak values with differential deformations of but a few millimeters.

Taylor around 1946-48 generally used the procedure of reconsolidating normally consolidated clays to pressures at least 1.3 σ'_p and higher (sometimes in multiple-stage tests to improve defining $ds/d\sigma_c$), in order to determine the presumed "intact" virgin compression behavior straight lines, which he estimated by extrapolating backwards to the presumed in situ consolidation stress of the soil element. Such were the simple beginnings of the so-called normalized behavior principle and technique (e.g. SHANSEP, which I would propose should be called ASHANSEP, Assumed Stress History and Normalized Soil Engineering Properties). Incidentally, if abrupt application of incremental consolidating stresses is accepted as likely (inevitable) to cause "destructuration", the principle should be to apply the incremental stresses gently, slowly, as gently as possible: we cannot successfully reproduce Nature without some degree of her infinite patience, which seemed a little less difficult in 1936 (Langer), in 1947 (Taylor, Kjellman-Terzaghi apud Chang, 11), than in 1984.

The simplicity of early concepts was gradually destroyed, but for the most advanced researchers in a given area overall geotechnical engineering has become too big a bite, and the net result is that while advancing most worthily one avenue, they accept the collateral avenues as facts, static, often at a level of over 30 years ago.

Bishop (03, 04) explored in detail the gradual invalidation of classical Terzaghian idealizations of pore fluid incompressibility etc., and surely many such reflections impinge strongly on behaviors of marine sediments, with gas contents, with very deep water columns, and so forth.

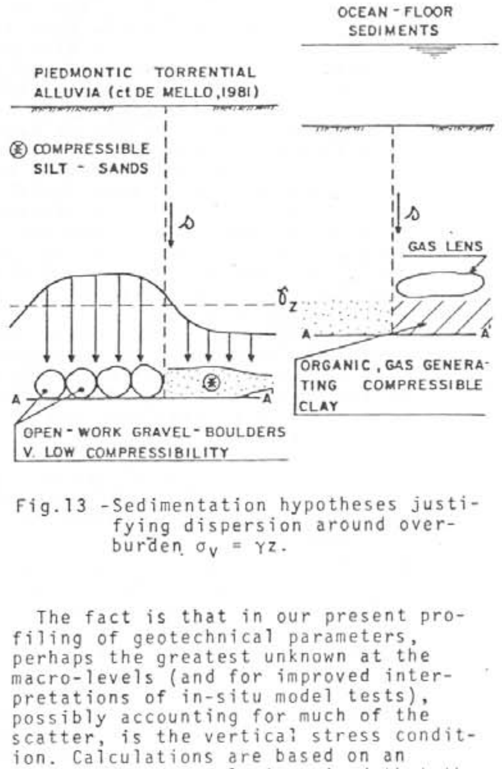


Fig.13 -Sedimentation hypotheses justifying dispersion around overburden $\sigma_v = \gamma z$.

The fact is that in our present profiling of geotechnical parameters, perhaps the greatest unknown at the macro-levels (and for improved interpretations of in-situ model tests), possibly accounting for much of the scatter, is the vertical stress condition. Calculations are based on an assumption so deeply ingrained that the question of attempting to measure this stress in situ has not even been raised. It forms part of the incomparably poor horizontal profiling.

With regard to the horizontal stresses and K_0 values (N.B. the Author has repeatedly expressed his preference for recognizing it as a K'_0 parameter, of effective stresses), some definite strides have been made. The Self-Boring Pressuremeter (SBP), the Marchetti Flat Dilatometer (MFD), the spade-like Total Stress Cells (TSC), and the Hydraulic Fracturing Test in borings (HFT), have been gradually proving themselves, and suffering the adjustments considered applicable, but in truth there are no proven in situ techniques which allow the direct determinations either of the initial stress state variables, or, much less so, of the maximum past pressures. This is a serious defficiency since most of the normalized behavior technique is highly dependent on K'_0, the σ'_{oct}, the maximum past influential preconsolidation pressure σ'_{vc} accounting for K'_0, and the overconsolidation ratio OCR considering such σ'_{vc} and not merely the vertical nominal σ'_p.

In honest recognition, there have been no systematic correlative studies of determinable σ'_p values from oedometer curves (assumed duly corrected for sample disturbance) with reference to what should be the physically interfering factors: the discontinuity of behaviors with OCR > 1 and OCR ≈ 1 should depend on the hysteresis loop of consolidation compression vs. swelling decompression. Thus, besides having to depend on clay-type indices (e.g. possibly ω_L and I_p) much should depend on the factors that increase irrecoverable compressions.

Among such factors, the one widely recognized (the Leonards-Bjerrum quasi-consolidation) is due to time of secondary compression. Moreover, obviously at the same σ'_v, different ratios of $(\sigma'_1/\sigma'_3)_c$ during consolidation are known to influence significantly but the necessary systematic triaxial consolidation testing is still pending. Effects of cyclic stress and creep compressions should be significant in marine sediments. And, finally, one need hardly mention the oft-repeated effects of desiccation preconsolidations, and of micro-strain brittleness due to cementations (chemical, colloidal-thixotropic, etc.), and brittleness destructurations that are not connected with consolidation stress history. The fact is that at the micro-levels both the destructuration processes and such effects as micro-volume upward seepage gradients etc.. have thus far led to rather vaguely hypothesized equilibria during secondary compressions (e.g. Chang, 11).

In summary, it seems that the normalized behavior technique points in the obvious right direction (since the early 1940's), but hitherto has been unable to contribute as would be desired and hoped, because the A (Assumed) in our proposed ASHANSEP carries too wide a range of variable assumptions in any practical case of deep profiling. The net results for use by practising professionals in marine geotechnology cannot go any better than such a scatter as depicted in Fig. 14 (apud Azzouz et al, 02) after all the adjustments.

Moreover, it would seem somewhat of a defeat to conclude with major parts of a clay profile simply defined, for in situ undrained strengths, by one or two very simplified expressions: there is no reason why a deep deposit should have had similar genetics and historic stresses from bottom to top of the postulated stratum. In particular, one would anticipate something more than an oversimplified relationship $c_u/\sigma'_{vc} = S(OCR)^m$ where S is the strength ratio for (laboratory) normally consolidated clay (OCR = 1) (see, for instance Azzouz et al,02). Presumably from many other studies S (undrained strength, normally consolidated, ϕ_{ap}) would increase with I_p, whereas quite in the opposite trend the irrefutable fact is that drained strengths ϕ' (for OCR = 1) decrease with increased I_p. (de Mello, 30). What persistent coincidences occur when natural complexities are straight--jacketed into simple single-parameter correlations!

7. Example of current interpretations of profiles of subsoil shear strength or resistance.

Of the very many geotechnical parameters needed, only one is selected herein to exemplify the current practices and problems. It is the profiling of subsoil shear strength, necessary for bearing capacity foundation design, pile dimension design for supporting capacity and "elastic deformation behavior", skirt penetration design, anchor designs for tension legs, and so on. It may be claimed to be the parameter of greatest use and greatest effort of investigation, both on land and in offshore geotechnology.

7.1. Cone Penetration Test, CPT.

The most widely used test hitherto is the CPT, although there are rapidly increasing tendencies to complement its direct indications by uses of CPTU and Friction Ratio profiling. It seems to the Author that one may fairly state that most offshore platform foundation designs have used direct semi-empirical correlations with the CPT point resistance values. Thus,a summary of the principal recommendations has been prepared for (a) clays (presumed saturated), (b) pure sands (zero c or c'), (c) general (c, ϕ) or (c', ϕ') soils.

(a) Undrained strengths of clays.

The principal correlations and recommendations concerning undrained in situ shear strength of clays are summarized in Table 1. Although the equations used are but modest "variations on a theme", the dispersion is not small.

It is interesting to note that, as is typical of civil engineering design decisions, the same set of data are used at one probable limit of the dispersion band for one situation, and at the other limit for another situation. Thus, for instance, quoting Lunne and Kleven (25) "The value which yields the most conservative strength for the type of problem considered should be selected...; where no local correlations exist, to use a cone factor of 19 for computing an average undrained shear strength for stability or bearing capacity problems... For estimating penetration resistance of skirts beneath a gravity platform, where it is

TABLE - 1

CORRELATION BETWEEN CONE RESISTANCE AND UNDRAINED STRENGTH.

Source	Type of soil	average Nk	field or laboratory tests	OCR
Kjekstad et al. NGI Publ.124 p.1	Nonfissured stiff overconsolidated clays (North Sea).	15 - 20(1) (best agreement-17)	Nk average: CIU 17,9; UU 15,7; UC 17,5	5 - 25
Amar et al. ASCE Conf. 1975,VI, p.22	Soft clay ~ Silt	30(1) (5 - 75)	triaxial tests	
(40)	Clays (young, nonfissured).	≈ 10(1) (electrical penetrometer tip). ≈ 16(1) (Begemann mechanical tip).	compression tests/ vane tests.	< 2
Lunne et al. Can. Geot. Jour. 1976, Nº 4,p.438	Soft to medium stiff clays. Norway and Sweden.	13 - 24(1) (five marine clays) 8 - 12(1) (Ska-Edeby Clay)	vane tests	
Koutsoftas, Fisher, ASCE GT 9, p. 989	Holocene clays New Jersey Coast	16 ± 3(2)	vane tests	3.5 - 9.0
Komornik, ESOPT, 1974, V.I, p.185	Medium-stiff to stiff clays.	15(2) (8 - 20)	vane tests	
Eide, ESOPT 1, 1974, 2.1, p. 128	North Sea clay $Su = 25 - 30\ t/m^2$ Soft and medium stiff clay	17(1) 8 - 12 12 - 20	vane test	-
(2)	Orinoco clay	site E1 = 12 ± 2 F1 = 15 ± 3 (depth < 110ft) 13 ± 1 (depth >110ft)	SHANSEP type tests	

Obs.: (1) $Nk = \frac{qc - po}{su}$ where qc: cone resistance
su: undrained strength
Nk: cone factor
po: total overburden pressure

(2) $Nk = \frac{qc}{su}$

conservative to consider a high value of undrained shear strength, we recommend the use of a cone factor of 11". A range of about ± 30% around the mean factor of 15.

Design decisions are appropriately taken at the conservative level: it would be hoped that such definitions might be taken to correspond to some statistical percent confidence levels, since statistics and probabilities have been increasingly applied in all collateral fields and design decisions in offshore engineering. Narrowing the band is not merely of interest for credibility and economics: sometimes the dispersion impinges on conflicting design decisions in the same foundation element. For instance, in planning drivability and penetrability of piles in some points one should use high strengths, whereas for the prediction of the same pile's supporting capacity one should use low strengths.

Quite obviously some of the width of the presumed statistical dispersion is due to the fact that the correlation is established at a very crude level, merely between q_c and $(s_u, \gamma z)$, and if additional first-order intervening parameters might be introduced (by use of profiling of Friction Ratio, S%, CPTU, and so on) it should be expected that the costly differences between upper and lower values adopted for design parameters might be reduced.

It is important to recognize that theoretical expressions on interpretations of CPT have been based on the assumptions (often fulfilled) that penetrations are under practically undrained conditions in (homogeneous) cohesive strata, and achieve essentially drained conditions in relatively clean sands. The use of CPT data for establishing deformability design parameters will not be discussed, to avoid over-extending the paper.

(b) Drained shear strength of sands.

The use of the CPT for profiling estimated ϕ' values in sands has followed traditional bearing capacity formulations (e.g. Janbu and Senneset, 21) using bearing capacity numbers Nq. For instance

$$q'_c + a = Nq\ (p' + a)$$

where $q_c' = q_c - p'$ = net cone resistance.

a = attraction (novel suggestion).

p' = effective vertical overburden pressure.

Obviously the results depend on shape factors (for instance, an angle β of the boundary of the plastified zone), and must be significantly adjusted for K'_0 and OCR.

Once again, the net results are "variations on a theme", with considerable variability, and need to employ empirically established adjustment factors based on tests in large calibration chambers. Lunne and Kleven's (25) recommendations are incorporated herein in Fig. 17 , where the SPT nominal interpretation chart is discussed.

(c) General soils with (c, ϕ) or (c', ϕ') contributions.

The vast majority of real, general soils continue to be the orphans of geotechnical investigations theoretically oriented. There are too many variables interfering, and it is impossible to solve many unknowns with but one or two simultaneous equations.

"Friction" $(ds/d\sigma)$ values occur not merely due to grainsize classification differences, but due to many other causes, S%, OCR etc.. reflecting in u and u dissipations). Obviously Friction Ratio FR and CPTU profiles have opened the doors to interesting indications, as shown in Fig. 15 (apud Robertson and Campanella, 36, and Senneset and Janbu 42).

Prudently such indications are yet considered principally as support for classifications. Design parameters are

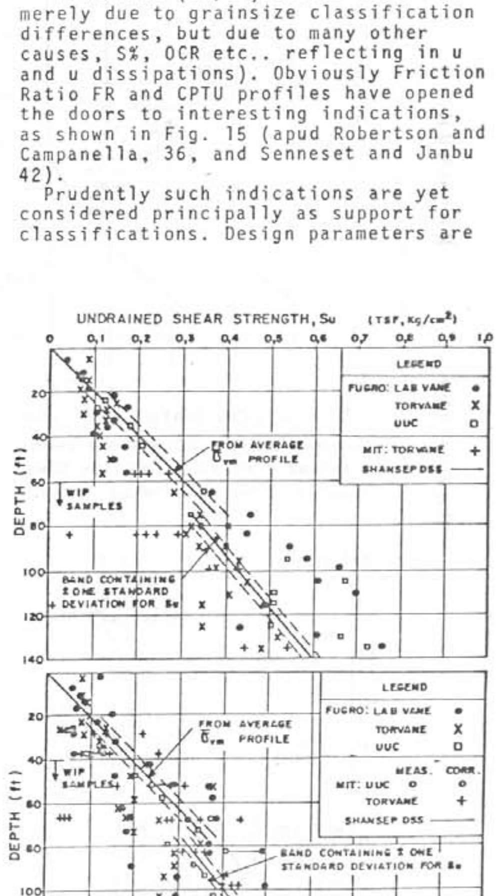
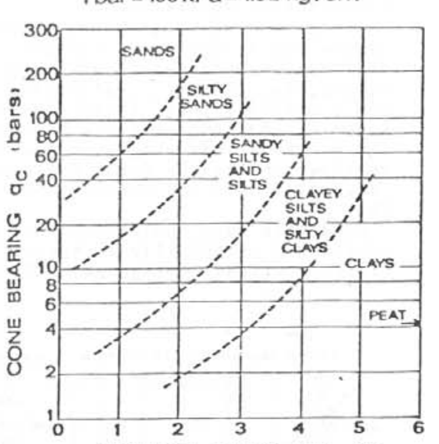


Fig.14 -Frustating scatter in clay undrained strength profiling.

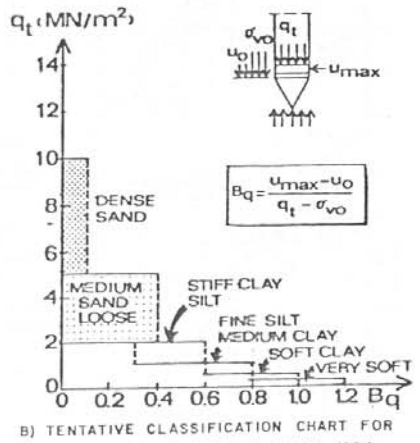
established by "judgement". Such data as summarized in Fig. 16 would frighten most engineers responsible for extracting judicious decisions.

7.2. Use of the dynamic Standard Penetration Test, SPT.

After the Author's state-of-the-art postulation (1971, 26) on the Standard Penetration Test, there have been some significant research efforts to improve the manner of interpreting SPT for



A) PROPOSED CLASSIFICATION CHART USING STANDARD ELECTRICAL FRICTION CONE (APUD ROBERTSON + CAMPANELLA, 1984)



$$B_q = \frac{u_{max} - u_o}{q_t - \sigma_{vo}}$$

B) TENTATIVE CLASSIFICATION CHART FOR SAME, FROM SENNESET + JANBU, 1984

Fig.15 - Charts for profiling (c, ϕ) soils via FR and CPTU.

cohesionless soils (ϕ' materials) on land.

It is comprehensible that no effort should have been expended in saturated clays, sensitive to some degree, since the thick-walled samples dynamically driven constitute a theoretical provocation. In fact, however, as will be mentioned in connection with drivability of piles, one might wonder if the immense research investments into more theoretical approaches (CPT and improvements) could not have yielded equivalent short-term benefit/cost ratios if shifted to strictly nominal empirical adjustment factors correlating SPT even in clays.

At any rate, we shall further recognize that another one of the Author's important suggestions was not pursued: it regards the simultaneous (intermittent) use of two or three driven samplers of designed differentiated dimensions and shapes, for investigating the minimum of 3 independent significant shear strength parameters in general soils (c', ϕ', σ').

We are thus reduced to re-investigations of pure sands, in somewhat regrettable variations on a theme, based on prior pseudo-theorizations. Dominant preoccupations and wishful thinking led to focussing attention on three principal issues, the first two affecting all investigations of sands, on land and in the ocean, and the third principally interfering with the latter:-

(a) search for generalizable correlations of SPT with Relative Density, D_R: ab limine impossible, except in each individual sand, since there is no general relation for ϕ', σ', OCR as a function of D_R except in very broad bands.

(b) attempts at predicting deformabilities of sands by means of SPT: quite predictably and inevitably an elusive possiblity for a crude test oriented principally as affected by stress conditions and strengths, not suited for detection of minutely differentiated deformabilities of sands, normally consolidated vs. overconsolidated.

(c) assessment of correction factors for rod lengths: the problem has been left as a moot question. In order of magnitudes it may well be so, for borings on land, involving moderate depths of interest. Interferences should, however, be anticipated to grow considerably as water depths increase. The practical problem has been shown easy to solve by down-the-hole sampler penetration driving equipment.

The fact is, however, inescapable, that the prevailing impression that the SPT test will be far too crude in comparison with other in situ tests and sophisticated adjustments of sampling plus laboratory testing, has practically excluded its systematic use, and adjustment.

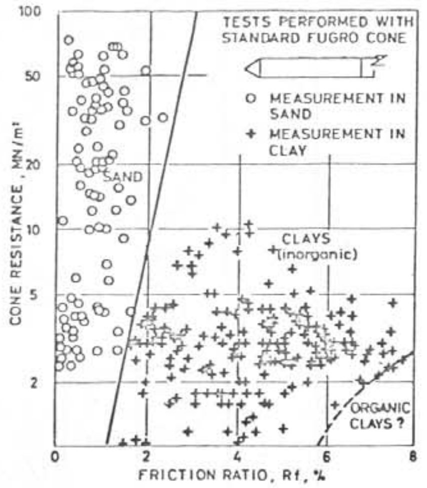


Fig.16 -Frustating scatter with FR data.

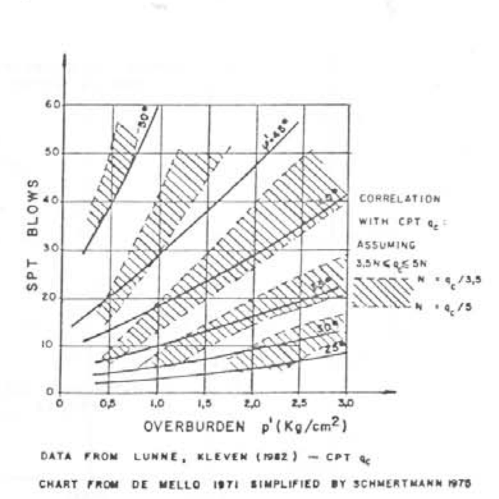


Fig.17 -Chart for estimating ϕ' in sands from SPT.

Why has the SPT suffered such discrimination? Doubtless mostly because it had humble origins of absolute empiricism. However, on the one hand one might concede that some degree of analysis can be associated with it. Fig. 17 shows that the estimation of ϕ' of sands via SPT, as postulated by a first order analysis (de Mello, 26) is quite compatible with the relationships that are in current use for obtaining ϕ' via CPT.

Furthermore, the Author has frequently emphasized that direct relationship between two complex lumped parameters may often prove as fruitful in practice as the academic attempts at composing theorizable analysis-synthesis relations, with insufficient feel for the variabilities hidden under idealized hypotheses. Despite the very limited data available, a hint of such a case is shown below in connection with SPT profiles compared with wave-equation pile penetration profiles.

8. Offshore pile foundation vertical load capacities.

Among the many major problems of geotechnical engineering, attention will be restricted to the one that should be simplest and most documented.

Firstly, one notes that in high capacity piles (de Mello, 32) for building foundations on land there is a rapidly increasing dispersion with incrêase of pile capacity, as plotted in a graph relating driving energy vs. ultimate load. Secondly, since the said data are plotted as a minute patch near the origin of coordinates in Fig. 18, one notes that (a) the increases of pile capacities for ocean platforms have been really disproportionate to the range of available data on land; (b) the trend for a greatly widening dispersion continues.

Fig. 19 summarizes a fair proportion of the data located, attempting to correlate "design" failure loads with load test failure loads. Note that the so-called "design" load is not merely a predicted value by static strength profile equations, but each adjusted value by sophisticated wave-equation computer-programmed continuously readjusted values (as would, in principle, satisfy Bayesian improvement from prior to posterior probable value predictions). Again, the variation is major

Fig.18 -Data of pile driving energy vs. ultimate loads.

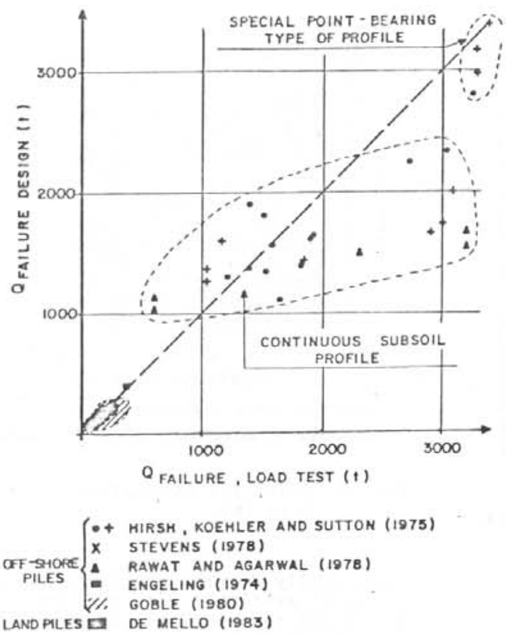


Fig.19 -Ultimate pile loads: wave-equation vs. load test.

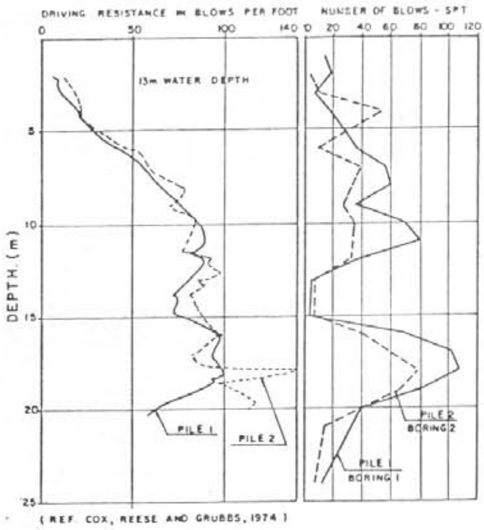


Fig.20 -Case of SPT and pile driving profiles from vessel.

since for a similar wave-equation prediction the load test failure load varied from about 700 to about 3500 tons.

Fig. 20 summarizes, for comparison, two profiles of SPT borings and corresponding pile driving resistances in shallow water, 13m deep. The dispersions are not alarming: but the Author would not insist on the insinuation, because the data are too few.

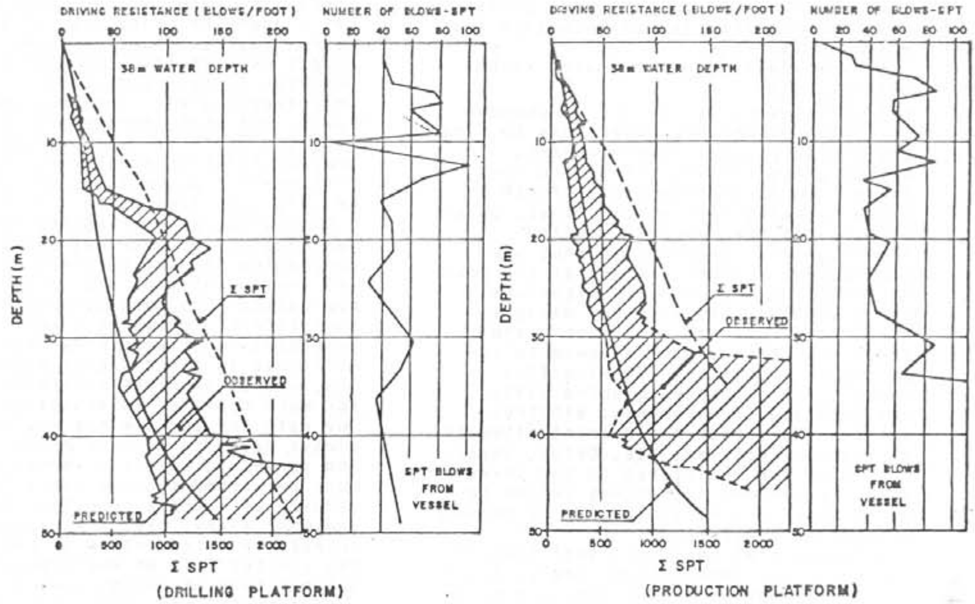
The additional data (too scant) that permit relating SPT with driving resistance, are given in Fig. 21. Attention is drawn to the curves of wave-equation driving resistances in comparison with the simple summation of SPT values (per meter) from top to bottom of the boring profiles, establishing a nominal integration of penetration resistance. Many predictions of driven pile lengths are based on such Σ SPT along the boring profile (de Mello, 28). The similarity between the two "calculated" graphs appears notable; it serves to show that any integrations tend to be dominated by our theorizations, failing to reflect the local variabilities (profiled data of the real pile driving resistances, Fig. 21).

In order to expose such a possible correlation more clearly, Fig. 22 has been prepared, using as simple a nominal correlation factor as merely the ratio m between Σ SPT to any depth and the wave equation prediction of resistance for the pile to that depth; two lumped parameters, and a nominal correlation. The dispersions in Fig. 22 are not as wide as in most of the previous drawings.

It must be strongly emphasized that there is absolutely no intention to promote much more crude empirical correlations in substitution for the laudable theoretical studies and proposals. There is a place and time for each. Merely as a humbling reminder of the highly relative value of such "dynamic penetration studies" we submit Fig. 23 that summarizes data (also with wide dispersions) of the effect of time (of healing) in changing the ultimate pile capacity. As is well known there are interferences of remolding and reconsolidation: the concentrated efforts in investigating undisturbed, intact, in situ strengths, have not been accompanied by compatible degrees of research into such effects, essentially inexorable.

9. General considerations.

Offshore engineering, including all foundation problems of its principal structures, have very rapidly met challenges of absolutely unprecedented scales, and constitute a success story in many respects. There are, however, some important reflections that must be brought to the geotechnical engineering contributions within the field, and that may serve as advice, both for the immense progress that still lies ahead



(APUD VIJAYVERGIYA AND CHENG, 1978)

Fig.21 -2 cases of offshore SPT vs. pile driving resistances by wave-equation prediction and observed.

in this field itself, and for analogous situations in the future.

In many a presentation the Author has yielded to the seduction of emphasizing the significant differences between complementary activities, of creative--inventive physical solutions, as compared with the systematic theoretical developments that permit justifying and exploring such engineered mega--level solutions in a quantifiable manner. The latter painstaking, indispensable efforts have been requested of our theoretically inclined colleagues: and, incidentally, there is no lack of intense creative-inventive production at the macro- and micro--levels in their doings. The proportions of apparent production vastly favour the latter and their publications, while the former often become as "obvious" as Columbus setting the egg upright at the dining table, and so are seldom published. In fairness we recognize that, generally, practice, creative practice, advances first, and analysis-synthesis follows in the wake.

By way of a single, most important, most repeated problem, dwarfed in the vast range of offshore geotechnical problems, the Author attempts to focus on some impressions affecting the present, and the desired, design concepts

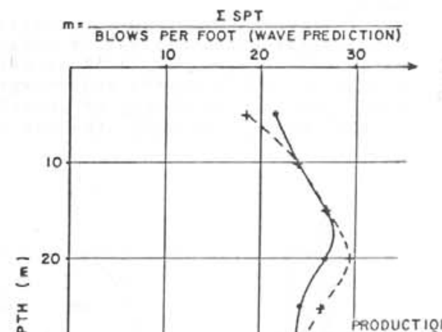


Fig.22 -Nominal comparison of ΣSPT vs. pile driving resistance.

and practices. Brevity appears preferable despite the risk of injustice and misunderstandings.

We are really concerned with rather distinct steps.

(a) Soil investigation and assessment of design parameters. One seems to find too little playful inventiveness, except along totally unfettered new lines: the developments come forth along existing theoretical lines, often blissful of the severely limited idealizations; progress is thus slow and steady. However, we are at the point of investigating our investigations.

(b) Inventive solutions and designs. These have been the great boon, and freer bridle should be allowed to investments along such "construction ideas"; for instance, grouted piles, jet grouting improvements, electrosmotic and/or other improvement attempts for pile capacities, etc. Often, imperceptibly, the resistance to the inventive improvement techniques arises because there are no (intuitive or proven) methods of analysis.

(c) Theoretical analyses, soil-foundation-structure interaction. Truly, numerical solutions, computorized, are more than 50 years ahead of our ability to feed appropriate parameters and behavior (constitutive) equations. Since we have fallen shy of associating purely nominal, empirical, parameter determination techniques, we find that theoretical analyses often find themselves imperceptibly tied down by our own earlier theoretical assumptions.

(d) Failures influenced by statistics of extremes versus allowable behaviors based on averages. Implicit in essentially all our analyses and integrations, are the hypotheses of behaviors conditioned by averages: the use of confidence bands around averages does not change the principle. In situ stress states assume averages. New soil models used in advanced finite element analyses incorporate yield stresses, anisotropic yield surfaces, and so forth. But what improvement is there in defining yield stresses related to preconsolidation pressures, as compared with nominal assumptions of some factor of safety FS limited to 1.5 or 1.3 or 2.0? How can we extrapolate from "law-abiding" (i.e. average) behavior, to decreasing FS, and to risks of failure?

(e) Instrumentation and performance evaluation. One must strongly support such efforts, choosing a well-balanced compensation between many mega-cases monitored in first-degree approximation, and a few cases carefully planned for more detailed investigations. At any rate any performance evaluation is always based on a prior mental model, and serves to adjust theorization. One must be warned against the illusions on warning of extreme condition failures, and must always have a strategy for corrective intervention on time.

(f) Limited life of exploration and optimization of design versus risk. In some respects oil production platforms suggest some similarity with mining engineering: they should be as safe as possible while in use, but could hopefully reduce to residual economic value zero at the end of the productive life. Geotechnical (and much of civil) engineering lies rather far from any possibility of aiming at such efficient designs.

Indeed, much of our work continues to develop on the basis of umbrella solutions. Tremendous potentialities lie ahead and beckon to our increased engineering and analytical-synthetic efforts.

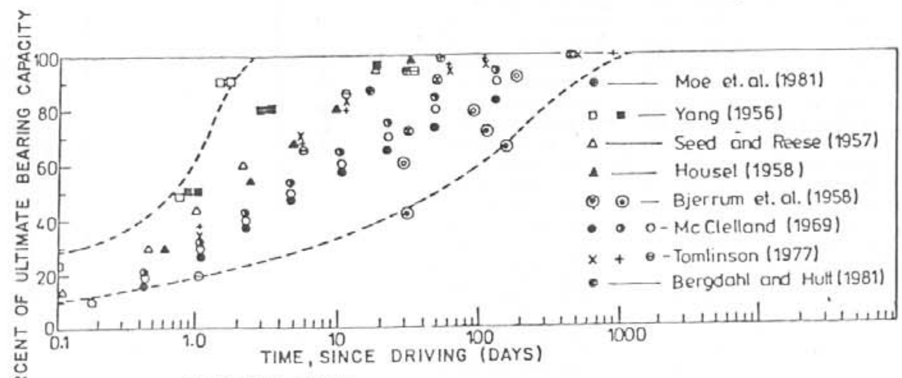


Fig.23 -Time effect on pile ultimate loads.

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