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VICTOR F. B. DE MELLO

Present Criteria for Design and Construction of High-Capacity Piles

Criterios Actuales para el Diseño y Construcción de Pilas de Alta Capacidad

VICTOR F. B. DE MELLO

Presidente de la Sociedad Internacional de Mecánica de Suelos e Ingeniería de Fundaciones; Catedrático de la Universidad de São Paulo; Ingeniero Consultor

1. INTRODUCTION

It may well be said that, with regard to pile foundations, the Paris 1961 International Conference (Kerisel and the prototype-scale tests at IRABA Chevreuse station) created a revolution, principally in invalidating rigid-plastic bearing capacity formulae for depth, and thereby making it questionable to use the simple concept of Factor of Safety on failure as the single or principal conditioning factor for larger diameter and deeper piles. The questions of deformations came to the fore (cf. de Mello, Lisbon 1956). An equivalent death-blow to the then prevalent design concepts on large bored piles, piers, caissons, was dealt at the time of the London Large Bored Pile Conference, 1966; not only were deformations paramount, but the deformations of shaft and base behaved in a significantly differentiated manner.

Meanwhile the needs of deep foundations for high load capacities and, especially, small deformations, increased considerably since the 1970's, both because of some very special structures (principally industrial) on land, and because of offshore structures. The worth of the investment of the superstructure per square meter of area of supporting ground has grown so phenomenally that it has fostered tremendous developments in deep foundation construction equipment.

As usually happens because of the much greater benefit/cost ratio of intuitive invention compared with analytic progress, the first big moves forward were with regard to equipment capabilitites. Moreover, industrial production and multiples permit adequate benefit/cost investment in equipment far more easily than in geotechnical problems which are strictly individual at each site. Thus one can understand the modern dominance firstly of advances in equipment and construction procedures, essentially intended to be applied irrespective of subsoil profile particularities.

In this presentation we shall consider only the problem of static vertical loading. Many other problems have received closer attention in view of needs ever growing: among them we might mention transverse loading, cyclic loading, impact loading, uplift or tension resistance, negative friction, corrosion and fatigue, etc.

In fact, as the range of new topics of importance widens, it often seems that the traditional first basic problem has been sufficiently well defined and has lost all glamour. We shall contend that although the basic principles are well established, the dispersions are still so wide that for garantee of success, the profession has to use PRESCRIPTIONS that incorporate a big factor of safety and therefore affect incremental costs perceptibly.

2. DISPLACEMENT PILES AND LIMITATIONS. HEAVE. SETTLEMENTS OF PILE CLUSTER.

Obviously with regard to developing high load-capacity deep foundations, the starting trend was to rely on the biggest possible displacement piles, preferably with widened pedestal. It is reasoned directly that by pushing ground outwards during the pile penetration, there should be compaction, favorable to higher resistances (excluding the cases, some of them quite disastrous, of sensitive clays in which no immediate "compaction" can ever take place, and the losses due to remolding far outweighed hypothetical benefits).

Also, there would be an intuitive expectation that denser and more resistant soil profiles would permit improved conditions for support of heavier total loads. One good example of the high capacity piling of the 1950's and early 1960's is the Franki pile.

The limitation to such thinking revealed itself promptly in the form of the heave of ground (and already driven piles, often damaging them) as summarized in Fig. 1. Many foundations have repeatedly suffered from this problem (and unfortunately continue to do so) especially when the designers thought they wanted higher Factors of Safety FS on ultimate load of the single pile. In Brazil, for instance, where the routine FS required is 1.5 and experience would generally be adequate with Franki pile center-to-center spacings of 2.5 d, as soon as a foreign design office intervened imposing an FS of 2.0 (and no immediate adjustment on local experience was

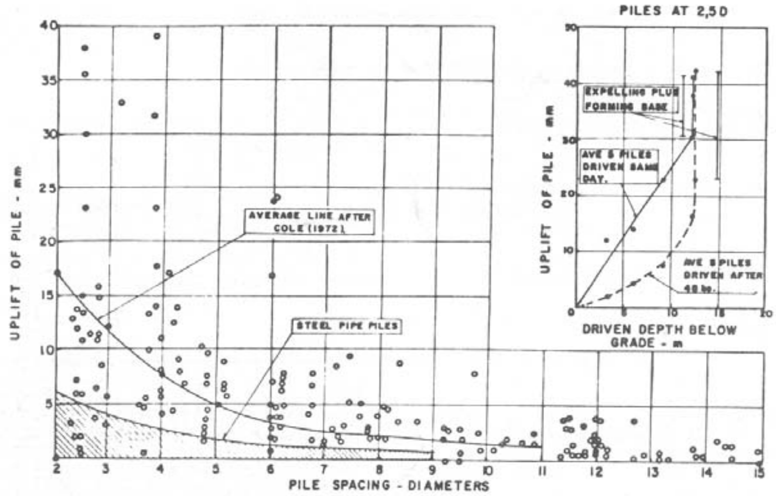


FIG I - UPLIFT MEASUREMENTS IN FRANKI PILES (REF 2, 3, 7)

made) there were serious cases of heave damages in such dense multitudinous pile clusters as required for silos, blast furnaces etc. In cases of buildings there was a palliative solution for cases of 3 to 4 piles per column by slightly battering the piles outwards. Another frequent palliative solution is to pre-bore to a certain depth. But visibly there is an absolute limitation that can be estimated by proportions of areas: since the nominal allowable σc on the Franki pile concrete (reinforcing implicit) is taken as 60 kg/cm^2, if 2.5 d spacing and FS = 1.5 are used, the superstructure loading is limited to about 5 to 6.5 kg/cm^2; if FS = 2, it drops to 3.8 to 4.8 kg/cm^2; if a 3.5 d spacing is required with FS = 1.5, the structure loading drops to but 2.5 to 3.3 kg/cm^2.

Obviously the use of steel sections would decrease displacement volumes and benefit from the much higher allowable σc. Thus one of the obvious tendencies was to move towards steel piles, both H and Tubular.

As regards FS values on single pile ultimate loads, it must be clearly recognized that in a case a rigid mat supported on a big pile cluster one should not be running the same level of risk as in column supports by one or two piles, and therefore it is absurd to continue to apply the "routine" values frequently codified (most often on the basis of buildings on which the typical economic selection leads to groups of three piles per average column). At any rate, in group behavior the problem becomes one of settlements.

The prime reason for the exponential increase in the use of high capacity piles, some of them of very special design and construction details, has been because of stringent requirements on allowable settlements for heavy industrial loadings. It is of interest therefore to collect as much data as possible for estimating settlements of groups of piles, since it is next to impossible to visualize load tests on groups of high-capacity piles. All estimates for design will have to be based on behaviours of individual piles and extrapolation to groups. Fig. 2 furnishes the simplest generalized suggestions (Skempton 1952, Vesic 1977) and some plotted data from case histories. Obviously the very installation of piles, changes the deformation and compressibility characteristics of the soil mass governing the behavior of single piles under load, and with respect to such changes much depends on differences between clays and sands, displacement vs. bored piles (corresponding compaction vs. stress release effects), and so on. It seems rather strange, therefore, that such a graph should not by now have been subdivided into several graphs, each related to a recognizedly distinct geomechanical statistical universe, for systematic gathering of statistical data for more confident use by the profession. The

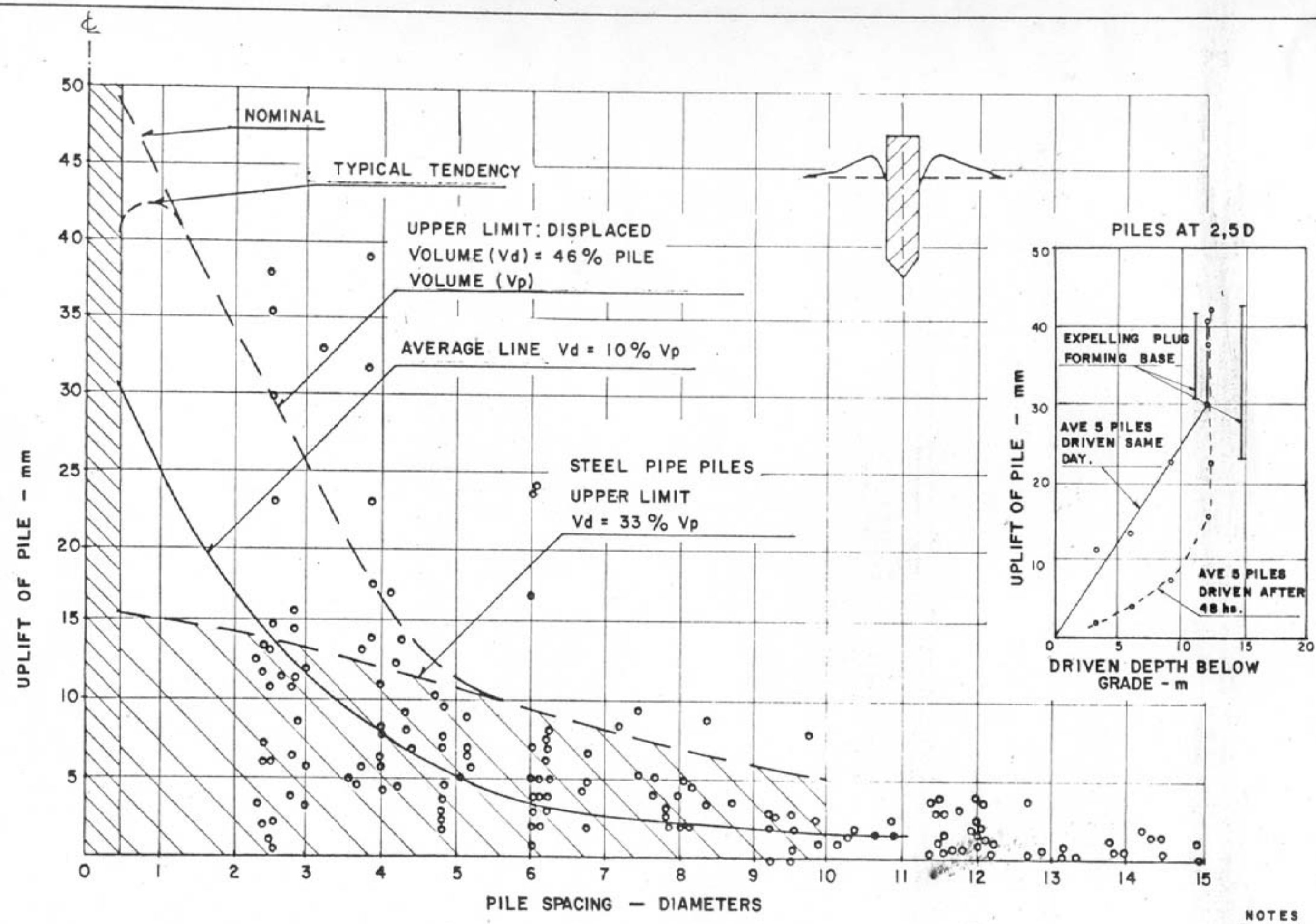


FIG.01 - UPLIFT MEASUREMENTS IN FRANKI PILES (REF. 2,3,7)

NOTES

1) - LENGTH OF PILES, ADOPTED $\ell \sim 20\,\phi$

2) - PIPE PILES ADMITTED WITH CLOSED END.

principal reason might be connected with the historic preocupation with group vs. single bearing capacities (failure loads), on which also the early simple reasonings of gradually reducing "efficiency" coefficients (e.g. Chellis, Wiley 1950) lost credibility, because of direct reasonings on different installation effects and different pile resistance contributions.

3. PROBLEMS OF SCATTER IN PILE BEHAVIOR VS DESIGN PREDICTIONS METHODS.

The basic factors of difference between shallow foundations (chronologically considered first), and deep foundations, are postulated to be:-

a) Conditions for investigation and prediction of shallow foundation behavior are much more amenable to reasonable prediction, both of bearing capacity and of settlements. Often, in dispersions of predictions for shallow foundations, much of the dispersion derives from the tests and is not associated with prototype behavior. As shown in Fig. 2 (b) the tendency is for observed maximum settlements in pile groups to be but slightly smaller than in shallow foundations when compared with the corresponding estimated settlements.

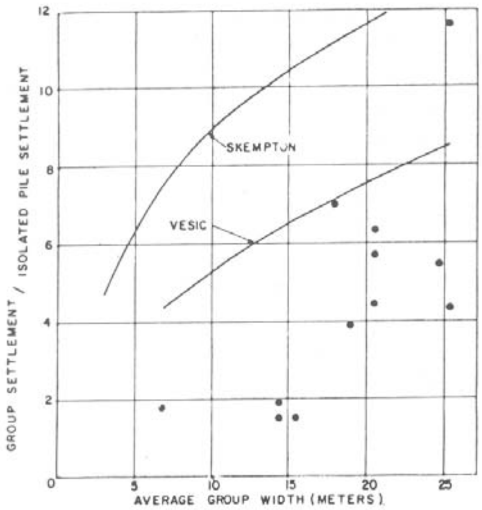
b) In general, there is judicious recognition that the shallow foundation may undergo significant settlements without any real fear of failure, whereas in the case of deep foundations any increased settlement immediately raises suspicion and risks of failure, depending on how shaft friction behaves post--peak.

c) If there is any doubt or insecurity with regard to the shallow foundation, the deep foundation (much more expensive) is used as the way out, and obviously for such a situation the demands on guarantees are more strict.

d) The predictability of deep foundation behavior is confessedly very poor because of depth and installation effects, both of which affect, in insufficiently quantified manners, both the borings and in-situ tests, and principally the widely varying construction procedures and realities of the foundation itself.

e) Whereas the predictability of behavior of any type of deep foundation is thus often subject to very much scatter, in well conceived foundation types the basic concept is that during construction one can resort to automatic adjustments (in depths penetrated) so that guarantee is maintained, even if at incremental cost. The problem should always be transferable to one of incremental cost.

Merely as an illustration of the very wide scatter that obtains in pile behavior I reproduce herewith some data pertaining to one of the very intensely investigated formations of the world, the chalk beds of England (cf. CIRIA Reports). It must be noted, however (cf. Figs. 3,4) that in some respects as regards engineering decisions, some of the



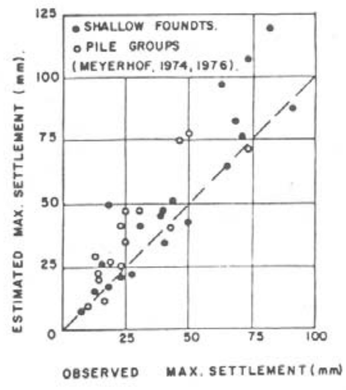


FIG 2 - DISPERSIONS IN PREVISION OF SETTLEMENTS OF PILE GROUPS (REF. 13,19).

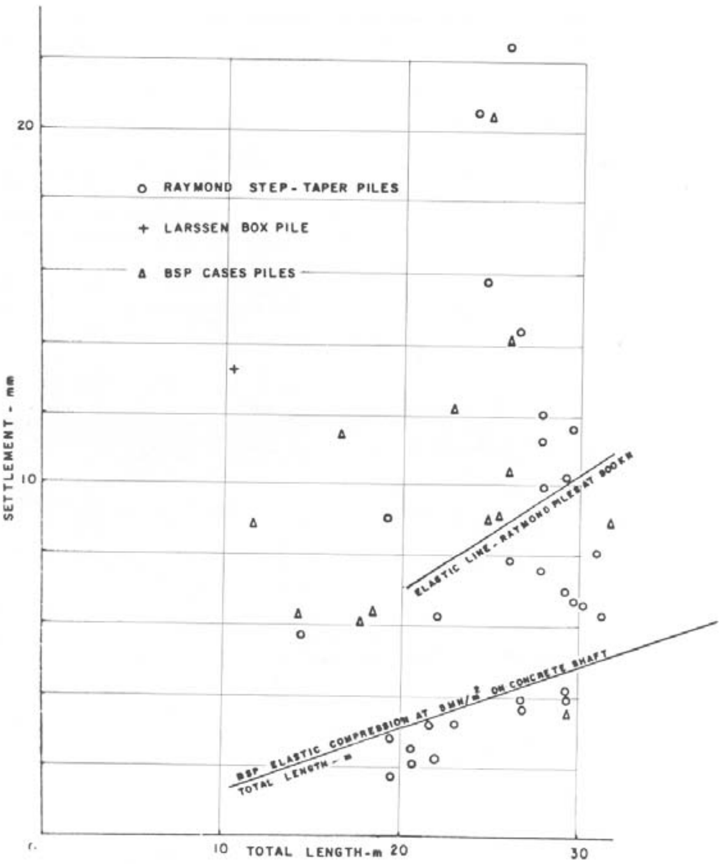


FIG. 3 - PERFORMANCE OF LARGER DISPLALEMENT STEEL PILES. (REF. 8).

scatter is only apparent, since a design decision would not change when a settlement varies between 2 and 10mm, although the error is 5-fold. The importance of such differences would be if they were not erratic, and if thereby a group action in settlement (conditioned by averages) were to result in several centimeters, with a 5-fold error in prediction.

As was briefly mentioned, the subject of corrosion of steel piles will not be discussed. However, Fig. 5 is included herein summarizing the results of a vast number of observations made in Japan on corrosion rates at ten places with severe environmental problems: it can be seen that on an average the corrosion rate is less than 0.015 mm/yr and therefore by increasing the steel thickness by 1.5mm a 100yr life is guaranteed. When such problems are discussed qualitatively, they become a source of mystification: in what civil engineering structures do we have the obligation (nay, even further, the right) to burden the costs to the present generation on the basis of hypotheses of what long-term deleterious effects might impose reinforcements 100 years from now, that is, at a time when the costs would be borne by about 4 generations later?

The fact is that the biggest dispersions occur in "direct" estimates based on borings, e.g. SPT values (cf. Fig. 6). However, there should be some tendency for compensating such wider dispersions either by appropriate adjustments of shaft lengths (total perforated length of bored piles etc., cf. Fig. 7) or by compensating through varying energy expenditures in driving (drivability and penetrability of piles, cf. Fig. 8).

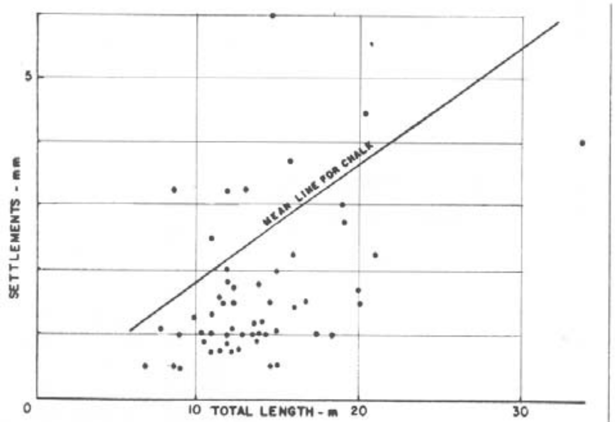


FIG.4 PERFORMANCE OF BORED PILES IN LONDON CLAY (REF. 8).

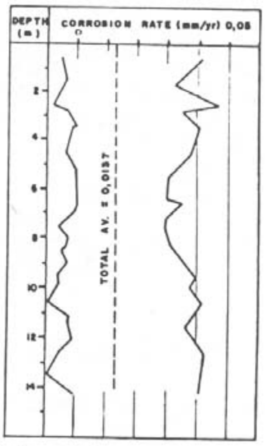


FIG. 5 - CORROSION RATES OF EXTRACTED TEST PILES (REF 10)

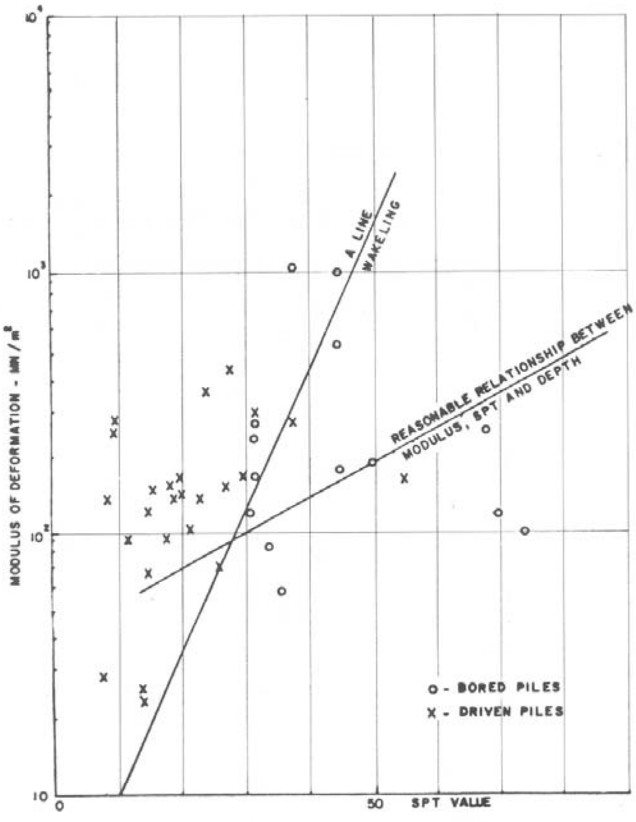


FIG 6 – PLOT OF BASE MODULUS OF PILES AGAINST SPT VALUE AT BASE. (REF 8).

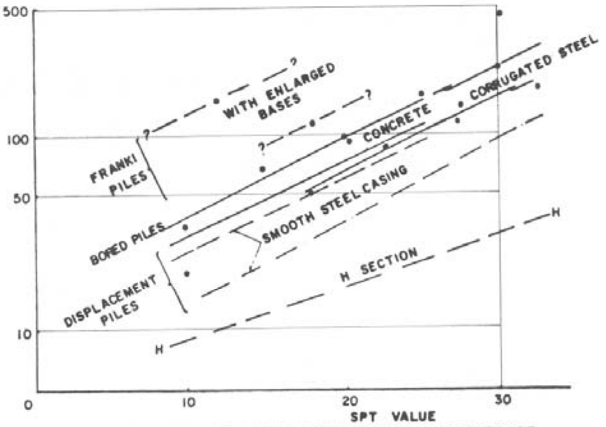


FIG. 7 GENERAL SUMMARY OF ULTIMATE SHAFT RESISTANCE AGAINST SPT VALUE. (REF. 8)

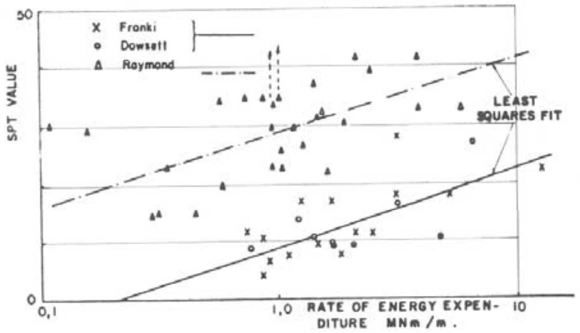


FIG. 8 - VARIATION OF RATE OF ENERGY EXPENDITURE ON DRIVING A PILE WITH SPT VALUE. (REF. 8).

4. STEEL PILES, H SECTION AND PIPE. BASIC DATA.

There is not much to discuss regarding the principles whereby the use of structural steel piles automatically came to the fore with a definite advantage for Pipe piles because of structural criteria (cf. Fig. 9). The principal advantages of the Pipe Pile over the H pile have rapidly pushed it to the forefront among the high-capacity driven piles. These advantages are easily made evident by the comparative data plotted in the following figures.

a) Considering the frequent dominant interest in developing lateral friction (cf. item 5), Fig. 10 evidences the interest in open-ended pipe piles, even though they obviously tend to create a bottom plug after some penetration, as can be visualized from the schematic insert, and can frequently be estimated.

b) Regarding driving energies and specifications of final penetrations per blow (mm/blow) Fig. 11 gives some pertinent indications. Firstly the obvious fact that under the same penetration per blow or energy per unit of penetration, the Pipe Pile gives a much better performance. Indeed, the principal concern with respect to H piles as compared with open-end pipe piles (Fig. 12) is that for a given drivability the penetrability will be very much greater: unless there be a hard underlying layer for very high point driving resistance, the use of driven H piles can lead to uncontrollably increased pile lengths.

Unfortunately it must be conceded (Fig. 11) that because of static vs. dynamic behavior,

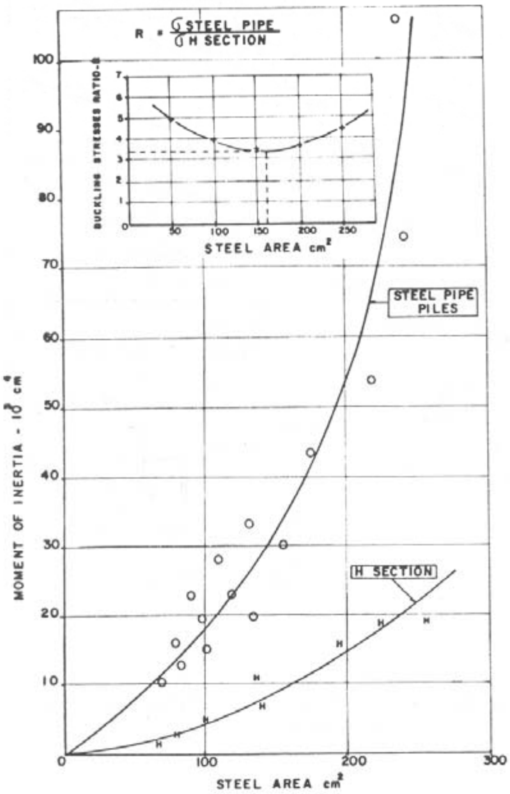


FIG. 9 - THEORETICAL ADVANTAGES OF STEEL PIPE PILES. (REF. 3).

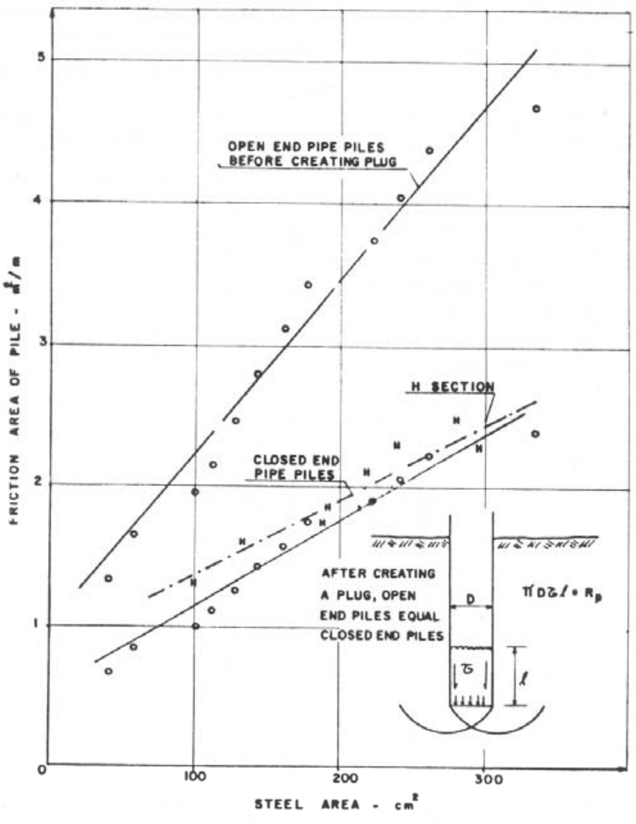


FIG 10 - COMPARISON OF FRICTION AREAS - STEEL PIPE PILES VS. H SECTION PILES (REF. 3).

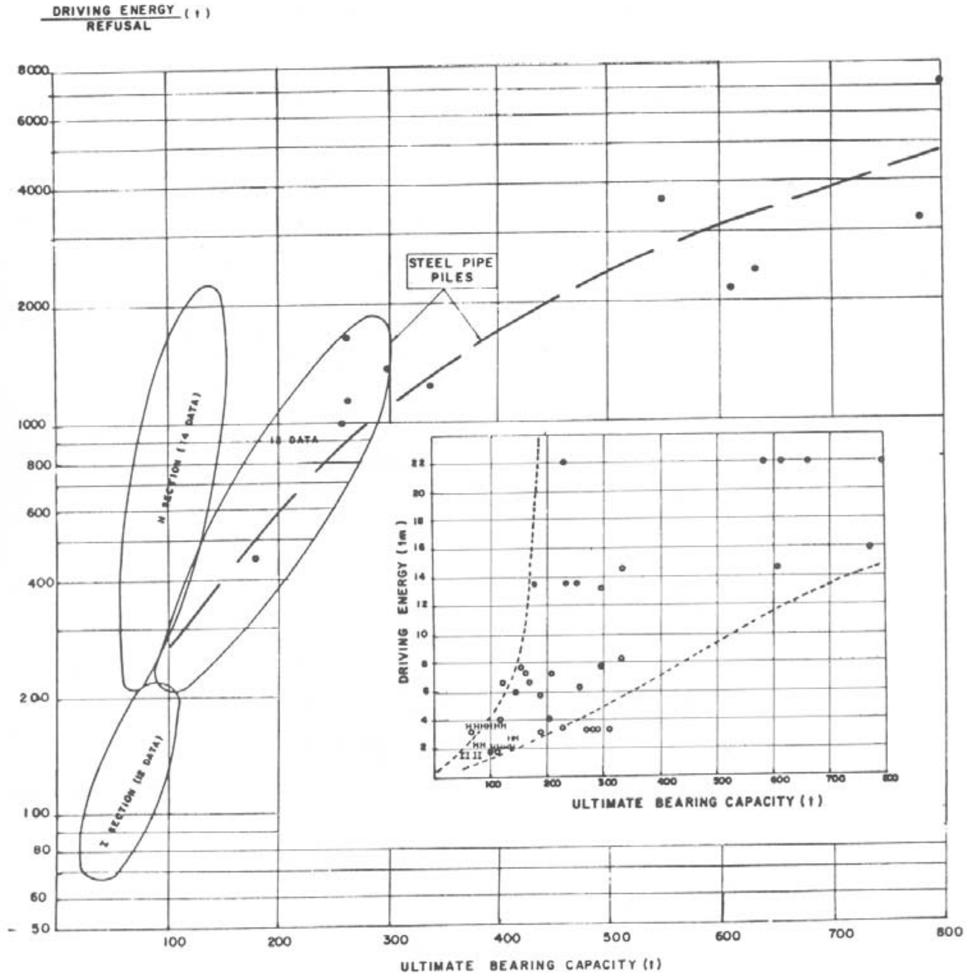


FIG. 11 - SUMMARY OF PILE LOADING TESTS. (REF. 3).

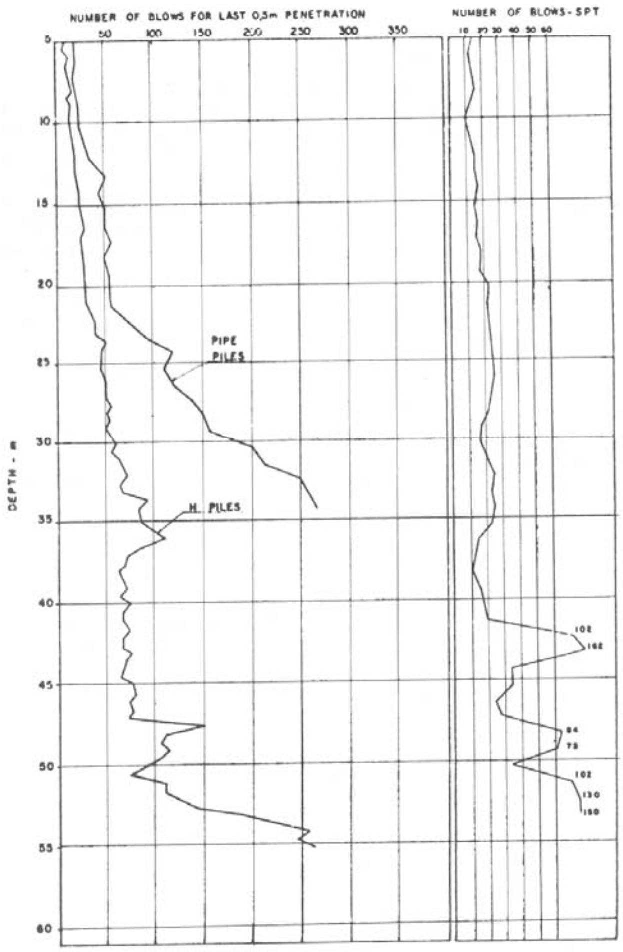


FIG. 12 - COMPARATIVE DRIVING OF H PILES AND PIPE PILES. (REF. 3)

the very important contribution of "set" or "soil freeze" with time in most soils, especially the moderately clayey ones, and also the relatively crude association of hammer energy W H with failure-penetration, the data on driving energy vs. ultimate load Qu in load tests present a very wide scatter. Such scatter is all the greater as the driving energies are greater, i.e. higher--capacity and longer piles are used. It has been proven imperative to use wave equation analysis for better correlation.

c) Finally Fig. 13 summarizes some data extracted from load tests and working loads at specific settlement conditions. The load has been selected corresponding to 0,5 inches of settlement, Q o,5: it can be seen that in the pipe piles for a given Q 0,5 the steel section can be loaded to much higher compressive stresses (thus requiring smaller area of steel for the same load).

5. BIG DIAMETER BORED PILES

In the 1950's the basic design principle on bottom-belled piers was to consider the shaft friction as approximately zero, and the principal contribution as derived from base resistance. In part this was logically associated with the fact the decision (much less frequent in those days) to resort to such deep foundations was generally associated with much softer upper strata and harder bearing strata for the base, and moreover, the construction procedures were reasoned, not unjustifiably, to lower the lateral friction very much.

It will be unnecessary to repeat, however, the presently well recognized fact that in large diameter piles there is a significant distinction (cf. Fig. 14) between the stress--strain curves necessary to develop shaft

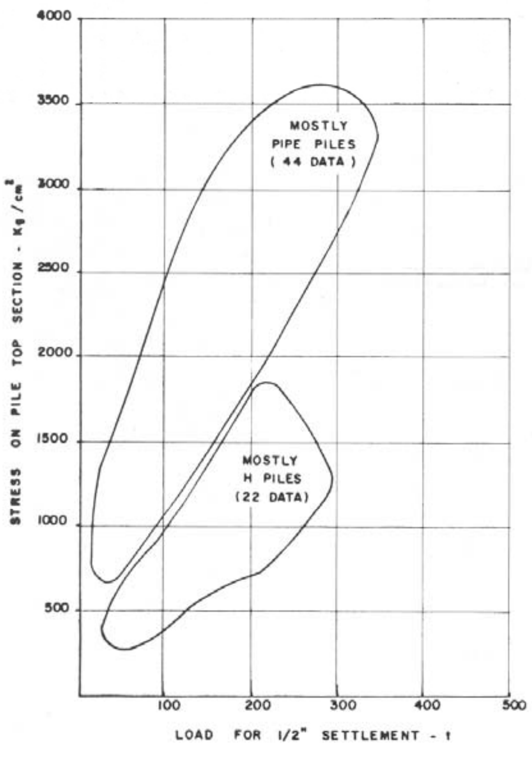


FIG. 13 - COMPARISON OF STRESSES ON PILE TOP SECTION FOR PIPE PILES AND H PILES (REF. 3)

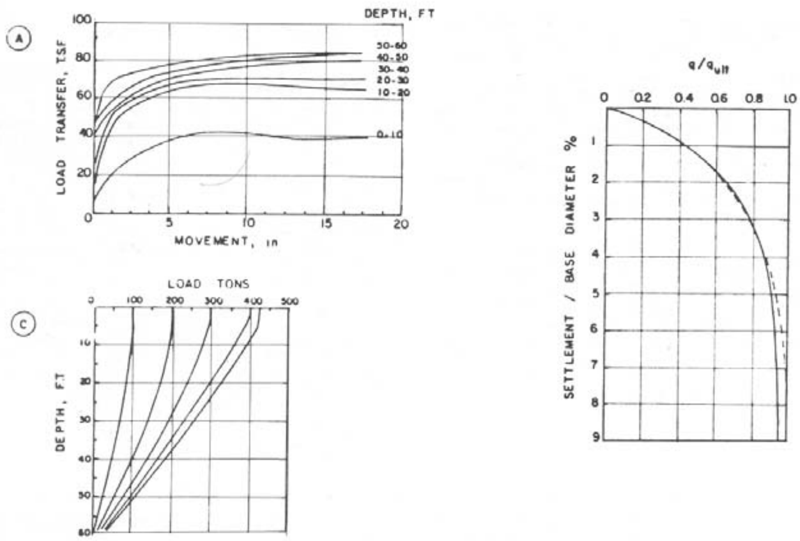


FIG. 14 - SKIN FRICTION AND POINT RESISTANCE OF BORED PILES. (REF. 20)

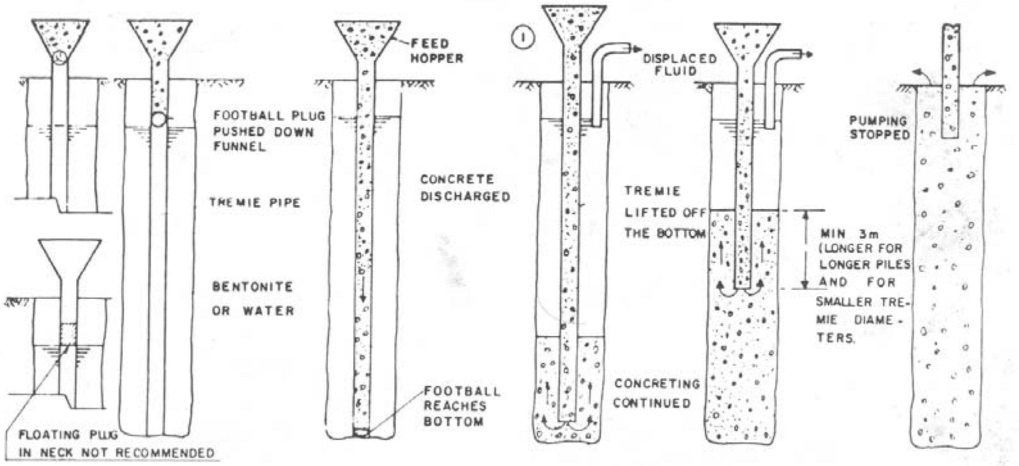
friction resistance and base bearing resistance. Whereas full shaft adhesion is developed with a top settlement of about5 to 10 mm (thereafter most often remaining fairly constant up to some cms of settlement), the load-deformation behavior of the base requires a settlement proportional to the base diameter (5 to 10% D) to develop to failure load. The two contributions cannot be simply added, and especially, since seldom would a settlement of some centimeters be considered allowable, the interest in bottom-belling to bigger diameters decreased considerably, in comparison with construction techniques capable of assuring a perfect cleanup and good embedment in hard material.

As is well recognized the typical load transfer behavior under increased loads on the large-diameter bored piles is given in Fig. 14. The participation of the base only begins as the loading increases. Considering that bored piles up to 4m diameter have been used, one can understand that only a very small base load will have developed at all with as much as 2 cms of base settlement, which corresponds to 0,5% of the shaft-base diameter. Obviously much depends on the length to diameter ratios, directly affecting the load transfer. Merely as one example of constructive solutions we should mention the special base that employed pressure grouting to prestress the base so as to force it into early load sharing.

Many bored piles have suffered from concreting defects. Fig. 15 illustrates some of the fundamental precautions regarding tremie concreting so as to avoid either the mixing of water and/or slurry into the concrete, or the trapping of air pockets in the funnel. The equilibrium equations can be solved once one has the viscous drag coefficient of concrete-steel depending on the slump.

Since shaft friction has become the principal contributor, three points must be emphasized which one should believe should have already been clearly established but which one repeatedly finds associated with lapses and unsatisfactory behaviors. Firstly if we express the in situ shear resistance at any elevation as Sis, and therefore the shaft adhesion as α Sis, it must be well understood that different construction procedures and different soils will inevitably generate much variation of α (cf. Fig. 16).

A second point that should have been cleared quite definitely is that the shaft adhesion does not develop as a function of deformations proportional to the diameter, but as a consequence of deformations of X mm, essentially analogous to a condition of direct shear. Prior to about 15 years ago it used be thought that in the same way as the development of the base resistance was (justifiably) in function of settlements as a % of the diameter, so would also the shaft adhesion merit being expressed as a % of the diameter. It would seem that both theoretical reasoning (under the applicable simplifications of considering the dominance of shear along the wall) and innumerable results of observations in piles of very different diameters, have clearly proven that the maximum adhesion



(1) - CARE NEVER TO RAISE TREMIE SO MUCH AS TO ALLOW CONCRETE COLUMN TO DESCEND BELLOW FUNNEL NECK, OTHERWISE AIR POCKETS ARE TRAPPED UNDER NEXT HOPPER CONCRETE DISCHARGE.

FIG 15 - COMMENTS ON STAGES OF CONCRETING BY TREMIE. (REF. 5)

develops consistently with about 5-10 mm. We must record that some authoritative entities (e.g. apparently the BRE, UK) are yet retaining the former tendency to express adhesion as a function of percentages of the diameter: I personally disagree with such a policy.

A third very important point concerns the use of bentonite slurry and its effect on adhesion. Fig. 16 presents representative data. It is presently well accepted that bentonite-stabilizing slurry, in bored pile construction does not affect unfavourably the lateral friction. It must be recalled however that there have been cases where an excessive delay before concreting has permitted forming a cake and damaging the expected soil adhesion: it has so happened that because of the time taken to lower carefully some instrumentation cages etc., there has been a coincidence that the very bored piles selected for instrumented monitoring and interpretative research, have been the ones badly affected in a manner that does not affect the piles routinely bored and concreted at industrialized construction rates. In Fig. 16 it is shown that some conditions of use of a casing do cause a loss of shaft friction. It need hardly be emphasized that in many unsaturated soils the use of bentonite slurries far from being a stabilizing procedure can be a cause of great loss of strength, and local failures of the shaft wall. In some clays that tend to absorb water rapidly, with consequent loss of strength, one should not only avoid the use of bentonite slurries but should investigate the influence of higher slump concretes (water: cement ratios)) in reducing the shaft adhesion.

6. PREDICTION AND CONTROL METHODS FOR DRIVEN AND BORED PILES

As has been mentioned, the published methods of predicting allowable loadings and estimated settlements for driven piles generally seem to give a very broad scatter. In part, however, it appears that individual and local professional "experience" is very much better than as reflected by such publications: unfortunately practising professionals have not been sufficiently induced to analyse their "experience intuitions" or to furnish their vast volumes of data (that generated such intuitions) for the purpose of closer analysis by research professionals.

At any rate, in the case of driven piles there is always an immediate (Bayesian) adjustment of predictions to realities as construction progresses.

The obvious bases for preliminary design predictions are by (a) empirical correlations with penetration tests, (b) by use of static load-settlement equations after estimating the

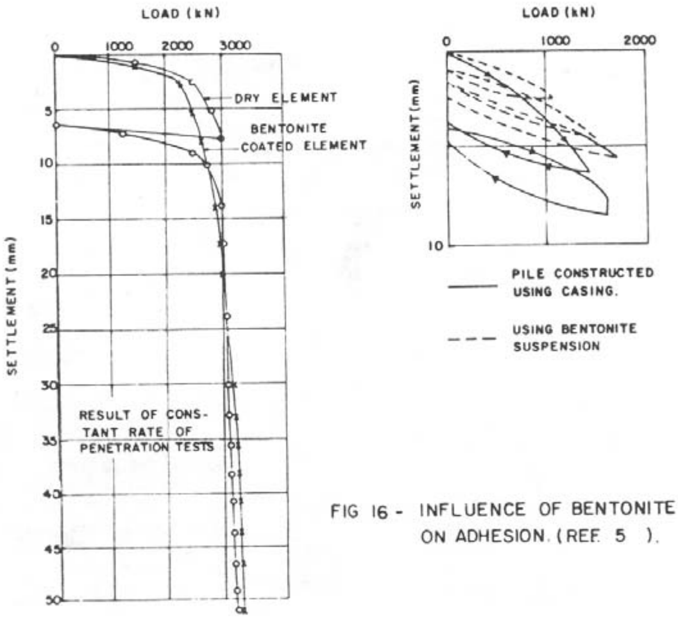


FIG 16 - INFLUENCE OF BENTONITE ON ADHESION. (REF. 5).

presumed fundamental parameters along the profiles.

6.1 Estimates via SPT, CPT etc.

There have been many simple proposals that merit the qualification of PRESCRIPTIONS. Some of them are summarized in Fig. 17. In the same figure we summarize data from F.B. Newman et al. (drilled Piers and Caissons, ASCE, St. Louis, 1981) on a specially instrumented and load caisson in which the inexorable indication is that all the recommended formulae give widely scattered results, but well on the conservative side,as is recognized to be a valid concept for safe PRESCRIPTIONS: similar situations repeat themselves throughout published case-histories

6.2 Estimates from static formulae

Both approaches have been followed, via undrained strengths (cohesion in clays and S in situ in general), and also via effective stresses. Also in each case the soil profile parameters have been obtained either "directly" through in situ tests (e.g. vane shear tests, interpretations of CPT and CPTU static penetration tests, pressiometer tests, etc.), or also estimated through "statistical correlations" with regard to the index tests from borings and in-situ determinations (SPT, CPT etc.). The ESOPT II, Amsterdam 1982 Proceedings are full of such statistical correlations, which once again, must be criticized regarding the scatter being too broad for professional practice: in the eagerness to obtain a great number of sets of data for statistical regressions it always happens that the net result is a much broader scatter than would be found valid in the local professional practice connected with each specific type of soil.

For the purpose of load-settlement predictions and analyses of soil-pile interaction in distribution of stresses, the present routines justifiably employ elastic analyses (Davis and Poulos, etc.) and, in some cases of somewhat greater displacements, elasto-plastic analyses, all of them provided with suitable charts for the practising professional.

In Fig. 18 I have taken the liberty to reproduce the data from Meyerhof 1976 (Terzaghi Lecture) in order to illustrate the method that is being employed by aythoritative pioneers of the teaching-research-consulting sector of the profession, to provide first--aid PRESCRIPTIONS. There have been no perceptible changes in theoretical approaches and formulae since Kerisel, Berezantzev, Vesic, but the efforts in the past decade have been towards gathering field and laboratory data, and plotting graphs of the necessary parameters, in order to provide the desired first-aid PRESCRIPTIONS for professional practice, despite all the scatter.

SKIN FRICTION (MEASURED 1100 TONS)

PRESCRIPTION	CALCULATED (TONS)
$f_{sz} = 0{,}026\,\bar{N}$ (SAND) $= 0{,}054\,\bar{N}$ (SILT)	950
$f_{sz} = \bar{N}/100$	355
$f_{sz} = \alpha\,\bar{p}\,\tan\phi$ $\alpha = 0{,}5$ FOR PENETRATION GREATER THAN 40 FEET	360
$f_{sz} = k\bar{p}\,tg\phi$, $k = 0{,}7$ $= \frac{\bar{N}}{34}$ FOR $\bar{N} < 55$ $= \frac{\bar{N} - 55}{450} - 1{,}6$ FOR $55 \leqslant \bar{N} \leqslant 100$	504
$f_{sz} = B\bar{p}$ B = FUNCTION OF RATIO OF PENETRATION TO SHAFT DIAMETER	735
$f_{sz} = \frac{\bar{N}}{55}$ (REF. 10)	645

f_{sz} - ULTIMATE UNIT SKIN FRICTION IN TSF

q_p = ULTIMATE POINT RESISTANCE IN TSF

$\bar{N}$ = AVERAGE SPT

N = SPT AT PILE TIP

P = EFFECTIVE OVERBURDEN PRESSURE IN TSF.

POINT RESISTANCE

$q_p = 4N$ DRIVEN (REF. 12)
$= 1{,}2N$ BORED

$q_p = 3{,}6N$ DRIVEN IN SAND (REF. 10)

FIG.17-SUMARY OF PRESCRIPTIONS FOR PILES BEARING CAPACITY.
(REF. 04)

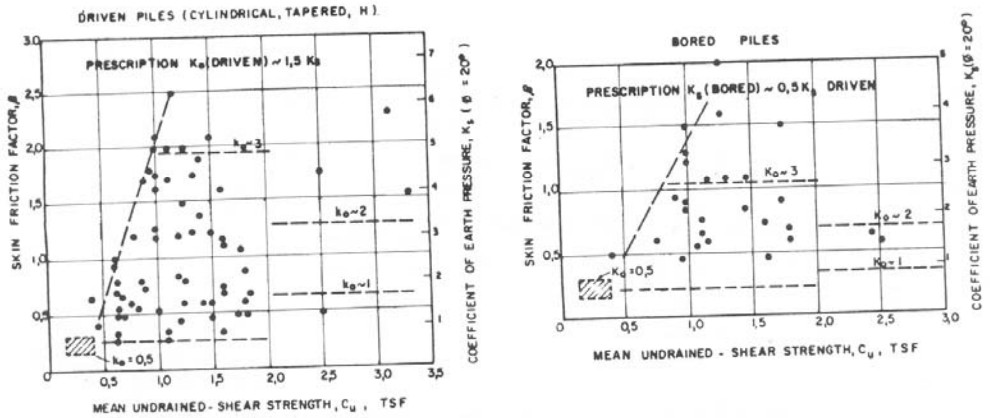


FIG. 18 – SKIN FRICTION FACTOR OF DRIVEN AND BORED PILES IN STIFF CLAY (REF.12)

In my personal experience and view, all of such efforts must be examined with caution: three points stand out:

a) sweeping PRESCRIPTIONS serve merely for first-aid preliminary design approximations where a professional or consultant is forced to face for the first time a new region and typical subsoil profile;

(b) the publications serve well to demonstrate the methods that each one of us might and should employ, to develop his own independent parameters and coefficients of experience;

(c) and, most important of all, the wide scatter serves to emphasize that no matter how authoritative an "expert opinion",unless it is based on specific experience associated with really similar soils and subsoil profiles (and construction equipment, with its installation effects), the scatter of opinions can be very great, or the Factor of Safety of decisions to cover the FACTOR OF IGNORANCE will have to be very great. The effect is predominantly on significantly increased costs. In choosing a Consultant, the choice is no longer dependent on presumed greater theoretical advances and merits, but principally on the degree to which the individual has real demonstrable experience with the specific type of region and soil.

6.3 Quality control in execution of bored piles. Predictability of working load and settlement estimates.

As generally happens in any Technology, attention first concentrates on the "dramatic failure" conditions. Bored piles have been subject to several types of construction defects and these have been responsible for the "dramatic failures". I beg leave to emphasize that the conditions leading to such construction defects are well enough known, have been well enough explained by "physics" and common sense" and "elementary principles of geotechnical engineering". The conscientious professionals of geotechnical engineering are doing a great disservice to the progress of the profession, and to their image in the eyes of society, by mystifying each additional case of a gross error: errors should be recognized as such, even though in all humility and charity the specific agents associated with the error should be spared any disproportionate penalty. I really think that it is time for us to set aside further publication of gross errors as if they were case-histories from which we learn something: the apprentice who committed the error should have learnt before subjecting his client to the devastating experience, and the profession to the shame. I shall therefore not expatiate any further on the classically recognized construction defects of bored piling.

Our interest is in the possibility of using the very perforation of the bored pile as a means of obtaining direct information on the quality of the soil that will participate in the soil-pile interaction. To begin with, since the shaft friction is of greatest importance, the principal information should be extracted from the shaft wall conditions: further, the conditions of the base within an "equivalent pressure bulb" should also be easily assessed. The recent Symposium on In Situ Testing, Paris, May 1983, revealed the great advances that have been and are being made in multiple profiling by diverse geophysical emissions and diagraphs.The weak point in the confident use of bored piles was the lack of such pile-by-pile confirmation of as-built conditions at the critical time of weakening of the soil interface just before concreting: by appropriate use of such new geophysical techniques of multiple profiling, this weak point should be properly excluded in future projects.

6.4 Driven piles. Wave equation control of driving and predictability of pile behavior.

The use of wave equation analyses of penetration behavior during driving of piles has been developed significantly during the past decade, for a much-improved substitution of the historic dynamic formulae that attempted to provide a quality control of each pile as-built. The great advances have been required and spurted because of such difficult and responsible conditions of pile driving and foundation guarantee as offshore platforms. Moreover, these advances have been aided by high resolution electronic equipment that records the minutes details of wave-transmissions, and by the computer advances that have permitted continuous automatic interpretative analyses and feed-back as the driving and penetration progresses.

In short, whereas the classic dynamic formulae may still continue in use for preliminary estimates, for the conscientious quality-control of pile driving they should be substituted by the well recognized wave equation analyses.

In routine practice it will probably continue to be expedient to use concomitantly the wave equation analyses and some classical dynamic formulae on a first few pilot piles, and, thereupon, after some minimum satisfactory correlation, the remainder of the piling under those site conditions may be moderately controlled by the classical formulae with correction factor.

Presently wave equation analyses constitute the most reliable technique for prediction and field behavior interpretation. In sequence of reliability we may list the following uses:-

(1) Optimization of (hammer assembly-pile-soil) combination.

(2) Prediction of peak driving stresses (often a controlling feature regarding damage to piles).

(3) Prediction of driveability.

(4) Prediction of ultimate bearing capacity based on blow counts.

The frequent frustrations in use of these sophisticated techniques make it imperative to repeat the insistent admonitions that in this as in other geomechanical problems, a sine-qua-non requisite, is for proper geologic assessment (including in-situ states of stress etc.) and for "experience" under similar conditions. Specific requirements seem to be at this moment: attention to when plug fixity occurs in penetration of open pipe piles and its simulation in the computer modeling; when there is soil set-up or relaxation (remolding etc.); the simulation of different proposed hammers is required during design. Good field measurements and engineering interpretation are indispensable to calibrate the model, whereupon the reliability for field control applications improves dramatically.

Two types of sophisticated modeling comprise:

(a) Single blow analysis: of limited value, often misleading, for the hard driving conditions that generally terminate a pile penetration.

(b) Multiple blow analysis: residual stresses due to driving are incorporated, obtaining a statically equilibrated solution at the completion of a hammer blow.

As regards computer modeling one should mention at present:

(1) Two principle programs available, at this moment: TIDYWAVE and CAPWAP, both utilizing single blow analysis, and destined principally for use of penetration data and instrumentation observations long before final resistances, in order to minimize pile-hammer simulation errors, and to adjust the unknown soil rheology;

(2) Multiple-blow programs, PSI and DUKFOR (extended), capable of incorporating non-linear soil-pile interface behavior (hyperbolic equation assumed). These are the programs presently more recommended for the analysis of final driving-penetration conditions, such as will permit simulating axial load test behavior.

Fig. 19 summarizes the basic concept of the wave equation analyses, and some of the data published on comparisons between predicted and observed pile behaviors.

7. FACTORS OF SAFETY IN PILE FOUNDATION DESIGN

One of the problems that the profession presently faces is that of somewhat premature standardizations of nominal factors of safety regarding complex "end-product" behavior parameters, such as bearing capacity, either computed or as deduced from load test data. As a result the conscientious geotechnical specialist would be forced to insert, into the basic index or fundamental parameters used for the computations, the principal adjustments regarding varying conditions of risk assessed.

One first question is the ratio of incremental live load to original dead load:generally due to construction time the dead load comes on gradually over an extended time,

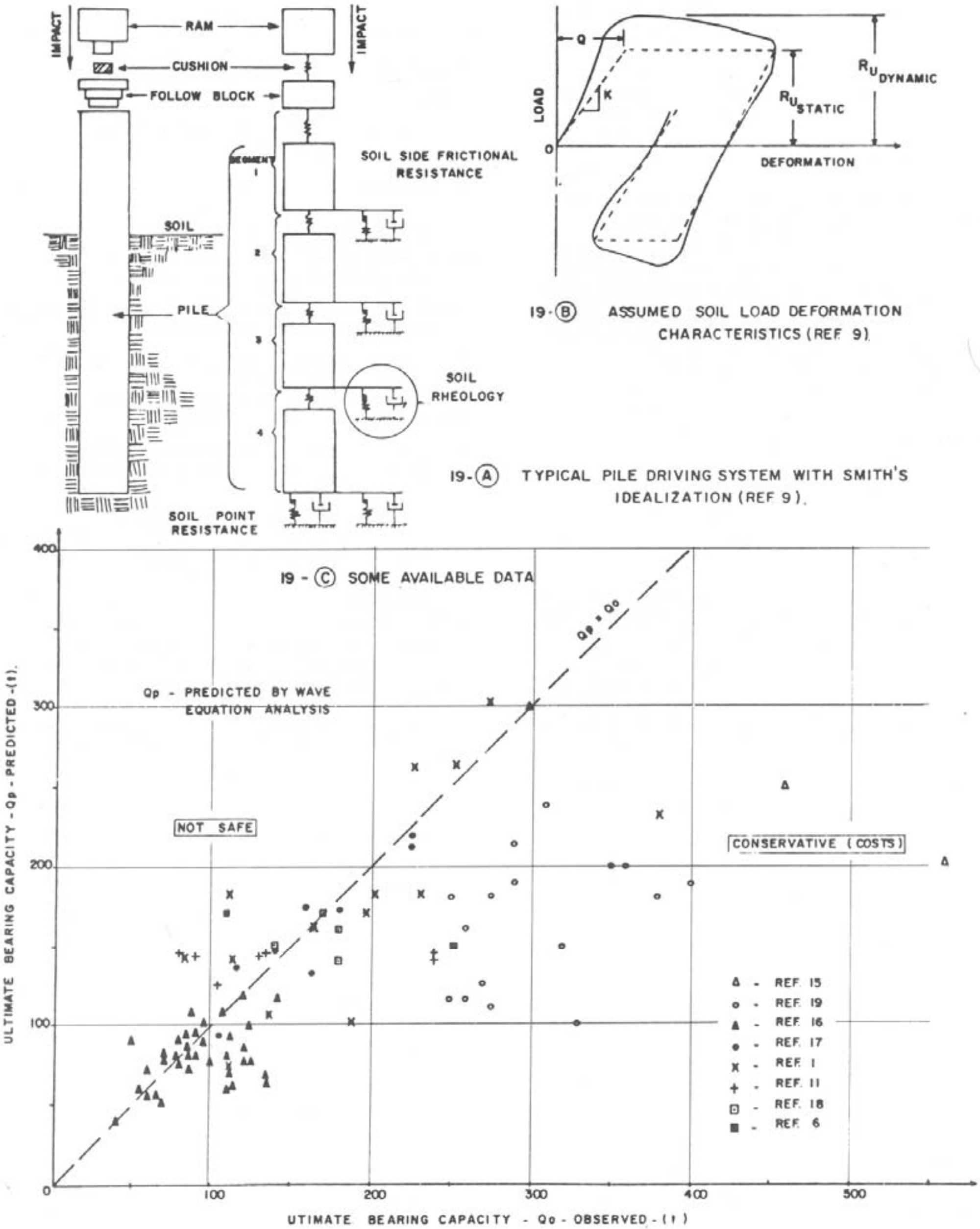


FIG. 19 - SUMMARY OF WAVE EQUATION ANALYSIS

whereas for live load one has to admit the probability of rapid application. In the case of concrete high buildings the ratio is often about 1/7, in the case of steel buildings about 1/2 to 1.5, whereas in the cases of industrial structures, tanks and silos, it can well be 10/1 or even much worse. Obviously the factor of safety required should be perceptibly higher in the latter cases.

Another question already discussed is the distinction between nominal Factors of Safety FS, Factors of Garantee FG, and Factors of Insurance FI. To a large extent many problems with bored piles to date have been due to the fact that installation effect deteriorations would make such piles subject to FI or FS conditions, whereas in principle driven piles can be considered "pre-tested" as to penetrability (although not quite similar, in distinguishing between dynamic and static loadings) and thereby driven piles could merit the priviledges of FG criteria. It is understood that for a similar degree of comfort we should require numerically different values FI >> FS > FG.

Moreover, in considering the group behavior of a foundation, two conflicting trends must be recognized. On the one hand, the probabilities of "erratic misbehaviors" of several individual piles coinciding become gradually smaller, and therefore the nominal factor of safety on individual pile behavior could be less stringent. On the other hand, there should be a trend (which has nothing to do with statistics and probabilities) of piles affecting each other, either favourably or unfavourably: the most unlikely hypothesis in nature is that conditions do not suffer any effect, remain insensitive to nearby additional piles. Therefore such a trend must be taken into account, deterministically, according to geomechanical reasoning. The net effect can end up being quite different from case to case.

In short, therefore, it is emphasized that one should be very carefull with regard to direct application of codified factors of safety with pile foundations. This becomes all the more important when high-capacity piles are chosen, as is often the case, because of the need to reduce the number of piles per column.

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