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Piles and other deep foundations

Pieux et autres fondations profondes

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SYNOPSIS This paper attempts to provide insight into the state of international practice for design and installation of axially loaded piles, both driven and bored. Its principal purpose is to report on the state-of-practice as it exists, not as the authors believe that it should be. The paper is based on a worldwide survey of practice conducted in 1983 that asked 36 specific questions on analysis, design and installation of axially loaded driven and bored piles. The responses are evaluated on the basis of geographic area and type of practice of the respondents. Contrasting views of practitioners from different areas and types of practice are highlighted where warranted. The paper concludes with the authors' opinions as to probable future directions of research and practice for design of axially loaded driven and bored piles.

INTRODUCTION

Preparation of a theme lecture for the Jubilee Conference of the ISSMFE is a sobering and humbling assignment. Review of the general reports prepared for the ten previous conferences is even more sobering and humbling. The useful words of J. S. Crandall (First); J. L. Kerisel (Second); R. B. Peck (Third); P. C. Rutledge (Fourth); L. Zeevaert (Fifth and Eighth); A. Kezdi (Sixth); V. F. B. DeMello (Seventh); J. P. Burland, B. B. Broms, and V. F. B. DeMello (Ninth); and B. B. Broms (Tenth) seem to have covered adequately the subject of piles and other deep foundations. Not intended to be a state-of-the-art-type lecture or a summary report of the papers submitted to this conference on this subject, this lecture was instead prepared for the purpose of being a report on the current international state-of-the-practice from a designer's point of view of design and installation of driven piles and bored piles to carry axial loads. It is not a report on practice as the authors perceive that it should be. Neither is its purpose to review the many varied analytical models, field procedures, quality assurance techniques, and strategies that exist for design and construction of pile foundations, as such reviews have been capably done elsewhere.*

A survey of worldwide practice on various aspects of design and construction of axially loaded piles was conducted in 1983. The responses were summarized and evaluated for presentation herein. Personal opinions of the authors are included within the discussion, hopefully well identified. The different practices and procedures indicated by the survey are examined on the basis of the geographic area and the type of practice of the respondents. A discussion of probable future directions of research and practice for the design of axially loaded piles concludes the paper.

* The authors' selections of state-of-the-art papers warranting close attention are indicated in the reference list by asterisks.

SURVEY OF PRACTICE

Questionnaire

A survey of current practice was made by means of a comprehensive questionnaire that asked 36 specific questions concerning analysis, design and construction of axially loaded driven and bored piles. Possible answers were suggested in multiple choice form for some of the questions, but the respondent was given the freedom to answer any or all questions as he or she saw fit and to provide comment.

An effort was made to develop a list of potential recipients to obtain a broad worldwide coverage with a reasonable distribution between consultants, engineer/constructors, owners, and academia. The first column of the upper part of Table I lists the number of questionnaires sent to each of five major geographic regions. Perhaps due to the location bias of the authors, 43 percent of the recipients were in the United States or Canada, with 23 percent in Europe, 19 percent in Asia, 8 percent in Australia or Africa, and 7 percent in Latin America. The lower portion of Table I shows that the categories of consultants and academia both received about 35 percent of the questionnaires with the remainder fairly well distributed between engineer/constructors and owners. All questionnaires were mailed during the Spring of 1983.

A total of 56 completed questionnaires were received in time to be included in the statistical analysis described herein. Other recipients either responded too late to be included in the

TABLE I

Questionnaire Response Rate By Category

Category	Number Sent	Number Returned	Percent Returned of Number Sent	Percent of Total Returned
		By Geographic Area		
Asia	36	10	28	18
Austr/Af	16	4	25	7
Europe	43	17	40	30
Latin Amer	13	1	8	2
US/Canada	81	24	30	43
		By Type of Practice		
E/C	26	9	35	16
Consultant	66	11	17	20
Owner	29	14	48	25
Academia	68	22	32	39

statistical analysis or made contributions through the submission of papers, reports, or other mechanisms that could not be categorized. However, the responses of these individuals were evaluated and have been included in the commentary on the analysis of the categorized responses. The names and affiliations of all contributors, both those included in the statistical analysis and those not included, are contained in Appendix A. The numbers of questionnaires returned in each of the categories are indicated on Table I along with the percent returned and the percent of the total returned. Figure 1 shows the worldwide distribution of the respondents, with numbers in the approximate locations of each respondent's home country. A cross analysis between types of practice and geographic areas reveals a very marked concentration of owners in the U.S. and Canada, 11 out of 14 for the composite group, including three oil companies. Half of the academic responses came from Europe. The other practice types are reasonably well distributed geographically.

On a geographic basis, the returns were approximately in proportion to the number sent. Table I indicates that except for Latin America 25 to 40 percent of the questionnaires sent were returned from the four other major areas. For the practice categories, the percent return ranged from 32 to 48 percent except from consultants, who had only 17 percent return.

Interpretation of Questionnaires

Each of the responses was read, and the answers to each question, where possible, were sorted into not more than ten categories. Each categorical answer was tagged with a category of respondent (type of practice; geographic area; type of pile), and the results were processed on a digital computer. The responses to some questions were either too varied or of insufficient number to allow the procedure described above to be followed. For those questions subjective summaries were developed. Subjective summaries were also developed for statistically processed questions to enhance interpretation of the quantified information and to aid in developing this commentary.

Statistical distributions for answers to the questions that were processed on the computer were developed by considering several independent segments of the data base: (a) distributions according to the entire data base; (b) distributions according to type of practice; (c) distributions according to geographical area; and (d) distributions according to type of pile dealt with. No multivariate frequency distributions were found to be significant due to the limited data base. Those frequency distributions judged to provide significant information on the state-of-practice are included herein in graphical form. Some distributions are described only in the narrative, and responses to a few questions have been omitted.

Evaluation of the State-of-Practice

The state-of-practice was evaluated from the questionnaire described in the preceding section. The questions posed to the respondents can be placed into five general categories: (a) design; (b) construction (including verification of design); (c) accuracy and reliability; (d) codes and standards; and (e) research. These categories will be considered in the order listed.

Fig. 1 Respondent Geographic Distribution

DESIGN

Assessment of Static Capacity

Respondents were asked to indicate which of the common procedures they use to assess the axial compressive capacity of driven piles in cohesive soils or rock and separately of driven piles in cohesionless soils. Figure 2 summarizes the distribution of the principal procedures used by the respondents for cohesive soils, with Figure 3 showing breakdowns of the distribution of principal procedures by geographic area and type of practice. Figure 4 is a summary for cohesionless soils, with breakdowns by geographic area and type of practice given on Figure 5. (The percentages noted on these figures were calculated based only on those 48 respondents who work with driven piles and not on the entire population of respondents. This system is followed throughout: when a question applies to both driven and bored piles, percentages are based on all 56 responses. Percentage responses applying only to driven piles or to bored piles are based only on responses of persons dealing with those respective types of piles.) For both cohesive and cohesionless soils, more than 50 percent of the practitioners prefer static load tests, local experience, and simple empirical correlations to other methods such as direct correlations with in situ tests, effective stress analyses, or capacity assessments based on dynamic measurements made during driving.

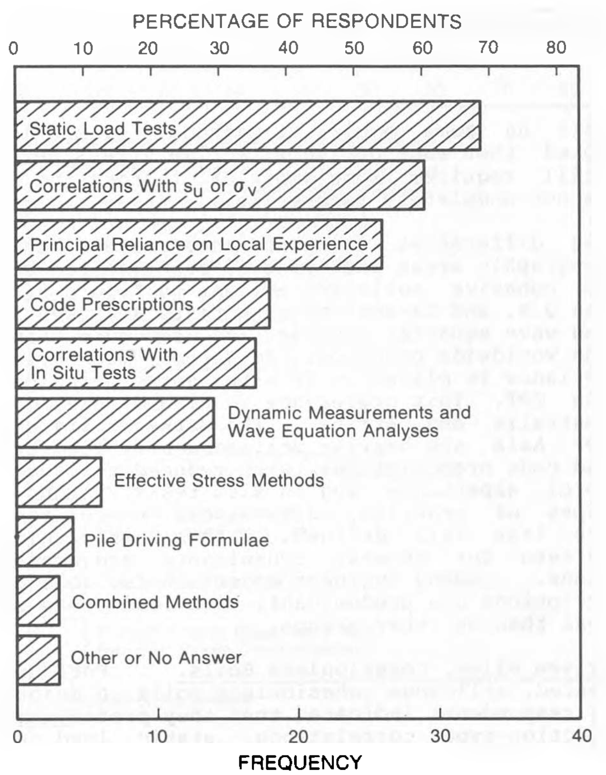

Fig. 2 Assessment Of Axial Compressive Capacity of Driven Piles In Cohesive Soils and Rock

Judging by the composite responses, the assessment of static capacity is often influenced by magnitude and location of the project. For a large, well-organized project in a geographical location for which relatively little experience with piling behavior exists, numerous respondents indicated that different procedures for capacity assessment may be used for different phases of the project. The factor of safety is often systematically decreased as the magnitude of reliability of the site specific data increases. For example, increasing reliability may be obtained in the following sequence:

(a) A feasibility study may utilize general geologic information and historical experience from sites of similar geologic composition to estimate pile lengths, sizes, capacities and numbers.

(b) A preliminary design study may involve collection of subsurface data in a reconnaissance fashion and the use of empirical correlations with the soil properties to assess pile capacities and thereby pile quantities.

(c) A detailed design study may follow where the preliminary study reveals unusual or variable conditions that require further site investigation or warrant more sophisticated analyses. A detailed design study may consist of increasing the coverage of subsurface borings or probes to define irregularities in otherwise well-understood soils; the execution of advanced level analyses; model studies; and/or full scale dynamic or static load tests at the site, with special attention to development of site specific correlations. All of the activities presumably increase reliability of the pile design. The construction contract is usually let based on information from this phase, or occasionally, just prior to this phase.

(d) The final design phase is executed by conducting further static or dynamic load tests using the production contractor's equipment and particular techniques to insure that the capacities established in the detailed design phase can be secured (or perhaps improved upon) and to obtain production control calibrations.

Proceeding from (a) to (d) one might expect the overall factors of safety used to establish allowable pile capacities to decrease from perhaps more than 3.0 to possibly less than 2.0. For smaller projects, phases (c) or (d) or both may be omitted, in which case reliability is lowered and higher factors of safety are used.

Driven Piles, Cohesive Soils. The empirical correlations described by non-European respondents for cohesive soils are typically those of the "alpha" type (Caquot and Kerisel, 1956; Tomlinson, 1957; Whitaker and Cooke, 1966; Tomlinson, 1971; Olson and Dennis, 1983a), the "beta" type (Burland, 1973; Meyerhof, 1976), and the "lambda" type (Vijayvergiya and Focht, 1972;

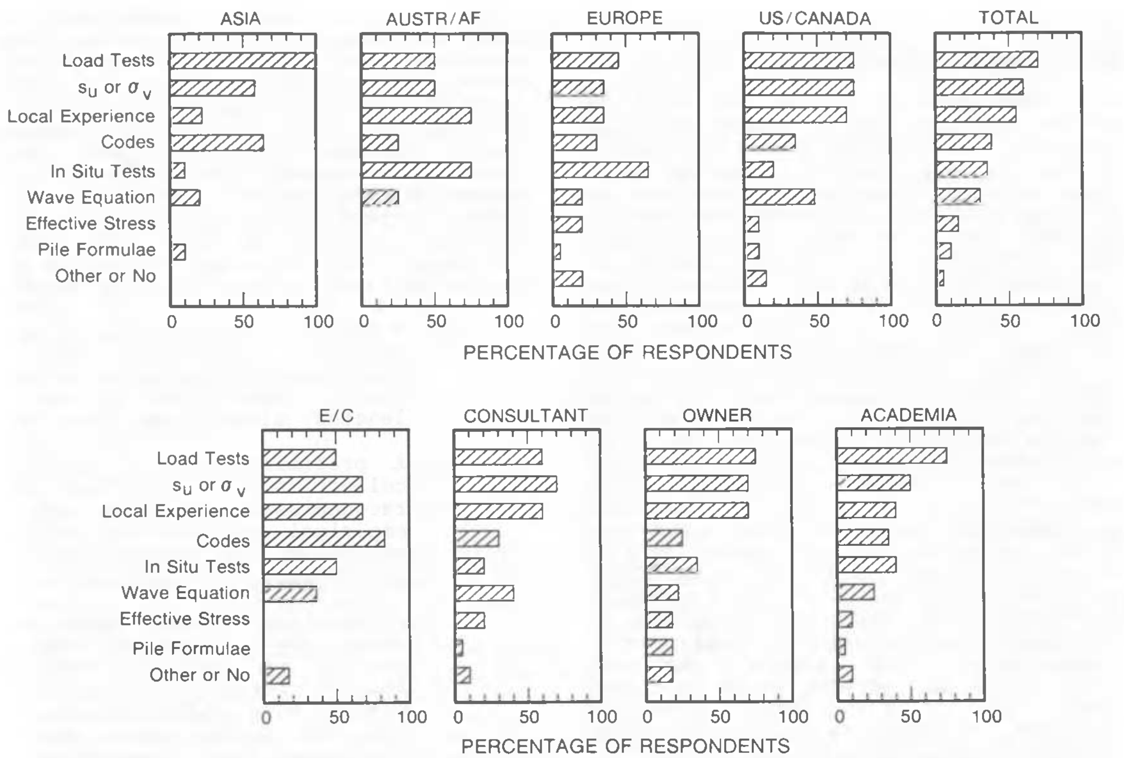

Fig. 3 Assessment Of Axial Compressive Capacity Of Driven Piles In Cohesive Soils And Rock By Geographic Area And Type of Practice

Murff, 1980; Kraft, Focht and Amerasinghe, 1981). There is no strong consensus on how the parameters for these empirical correlations should be obtained other than that thin-walled tube push samplers or piston samplers should be used to recover samples for laboratory testing. The principal parameter used to describe cohesive soils is uniformly "undisturbed, undrained shear strength" measured primarily by means of miniature vane, unconfined compression, and UU triaxial compression tests, with estimates also obtained with a pocket penetrometer or torvane. The cone penetrometer is mentioned, but in situ vane tests are apparently used very little for direct numerical evaluation of the undrained cohesive shear strength. The in situ vane is, however, used routinely for supplementary strength measurements in many offshore site investigations. Almost no use of the pressuremeter test (PMT) is reported by non-European respondents.

Theoretical effective stress methods (Esrig and Kirby, 1979; Kraft, 1982; Kirby et al, 1983), which were quite popular among researchers in the late 70's and early 80's, have not entered design practice to any significant degree. When they are used, they are employed in conjunction with other methods, usually in a secondary manner. Several of the academic respondents indicated that such methods introduce too many assumptions. While they provide insight into the basic mechanism of load transfer characteristics in clays, they do not predict capacity of piles, particularly in overconsolidated clay, as well as some empirical methods. It should be noted that each of the effective stress methods still requires some empirical factors and thus is not completely theoretical.

The differences in preference pattern between geographic areas and types of practice for piles in cohesive soils are evident on Figure 3. In the U.S. and Canada, in situ tests are used less and wave equation analyses are used more than in the worldwide practice. In Europe, much greater reliance is placed on in situ tests, principally the CPT. This preference is also displayed for Australia and Africa. The marked differences for Asia are heavier utilization of load tests and code prescriptions, with reduced reliance on local experience and in situ tests. Among the types of practice, differences in preferences are less well defined, with a fairly uniform pattern for owners, consultants and academicians. Among engineer/constructors, code prescriptions are predominant, with load tests used less than by other groups.

Driven Piles, Cohesionless Soils. For uncemented, siliceous cohesionless soils, a majority of respondents indicated that they prefer simple friction-type correlations, static load tests and reliance on local experience to assess the compressive capacity of driven piles. The simple friction procedures (Nordlund, 1963; American Petroleum Institute, 1982; Dennis and Olson, 1983b) involve the relation of unit shaft friction or end bearing to effective overburden stress without explicit consideration of the effects of soil rigidity, curvature of failure envelopes, residual stresses or other factors. (Dennis and Olson, 1983a and 1983b, have shown through their data base research techniques that simple friction procedures in cohesionless soils yield less reliable results than the various empirical correlations for cohesive soils.) The apparent worldwide preference for simple friction procedures results from an overwhelming usage in the U.S. and Canada and substantial usage in Asia. In Europe, Australia, and Africa, only about one-fourth of the respondents rely on this procedure.

To assess the angle of internal friction for simple friction analyses in cohesionless soils, the standard penetration test (SPT) or some related version of a dynamic penetration test is used almost exclusively. Only modest use of direct correlations with CPT data was reported by non-European respondents.

European respondents, particularly those from the Netherlands, Belgium and France, use established design rules based directly on measurements made by in situ tests, particularly CPT and PMT tests (deBeer, 1971-1972; Gambin, 1979; Baguelin et al, 1982; Begemann et al, 1982). The data base for such rules, especially for the CPT, is so extensive that pile load tests are not routinely conducted and are even dismissed by several respondents as not being cost-effective for most of their projects.

The practitioners who specify dynamic monitoring techniques during pile driving or retapping to

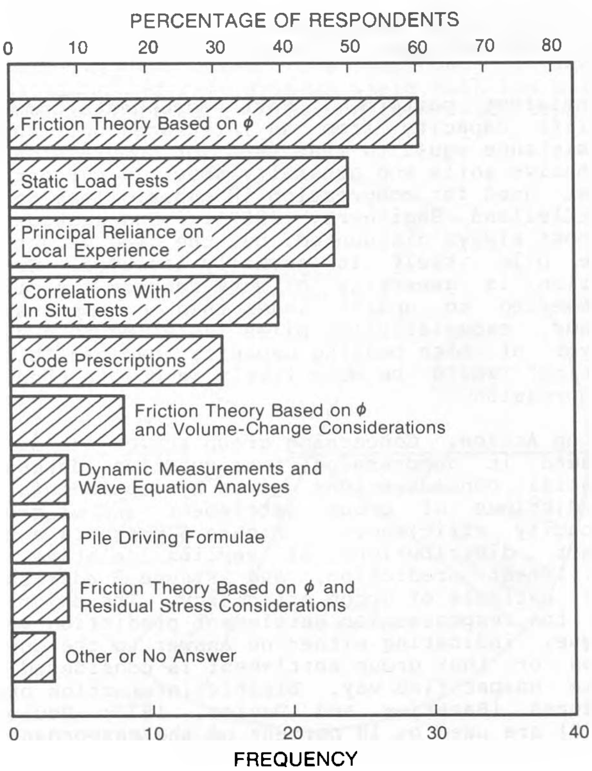

Fig. 4 Assessment Of Axial Compressive Capacity Of Driven Piles In Cohesionless Soils

confirm capacities of driven piles (Rausche et al, 1972) tend to have more confidence in the technique for cohesionless soils than for cohesive soils, even though a larger percentage of respondents use this procedure for cohesive soils. There is a perception that such techniques aid significantly in establishing acceptable refusal blow count in cohesionless soils. These techniques are used in some form on a generally routine basis for capacity evaluation and for driving equipment selection (at least for large or difficult projects) by many U.S. practitioners but have not significantly entered into Asian, Australian, and African practice.

Variations in preference patterns displayed by Figure 5 not previously noted are a strong usage of load tests and code prescriptions in Asia accompanied by a relatively low reliance on local experience. In Australia/Africa, high reliance on local experience was the predominant response. In the U.S. and Canada, code prescriptions are used by only 14 percent of the respondents. The patterns by type of practice show more differences than for cohesive soils. Consultants and engineer/constructors give overwhelming preference to simple friction procedures, whereas owners rely heavily on load tests and academicians turn to in situ tests. Wave equation use is concentrated almost entirely in consulting practice, with pile driving formulae still utilized by 17 percent of engineer/constructors, half of whom also rely on code prescriptions.

Respondents who deal with cemented and/or calcareous sands have no consistent satisfactory solution to the estimation of pile capacity in such materials from either in situ or laboratory tests. Pile designs for such soils thus tend to be approached conservatively but without the assurance of conservatism.

Bored Piles. The design of bored piles is perceived by most respondents as being more dependent on construction technique than is the design of driven piles and thus as more empirical and related to local construction practice. For example, in those areas of the U.S. where rock or hardpan can be easily reached by bored piles, shaft resistance is normally excluded in the overburden. Principal construction attention is given to the socket of the shaft in the bearing layer rather than to the potential capacity in the overburden because of an uncertain condition of the concrete-soil interface along the shaft. In the Southwestern U.S., where bored piles are often installed in formations of relatively consistent strength, shaft resistance is utilized when reasonable forecasts can be made of the critical construction factors such as the method of retaining the soil (casing, slurry, or no retention), the time between drilling and concreting, and the rate of concrete placement.

Bored piles, particularly in cohesionless soils, tend to be designed using smaller unit end bearing and shaft friction values than are used for displacement-type driven piles. When shaft resistance is used in sands, it is generally in the order of one-half of the value used for comparable driven piles, even though it is the authors' opinion that this reduction is too conservative if proper control is exercised in construction of the bored piles. Unit end bearing

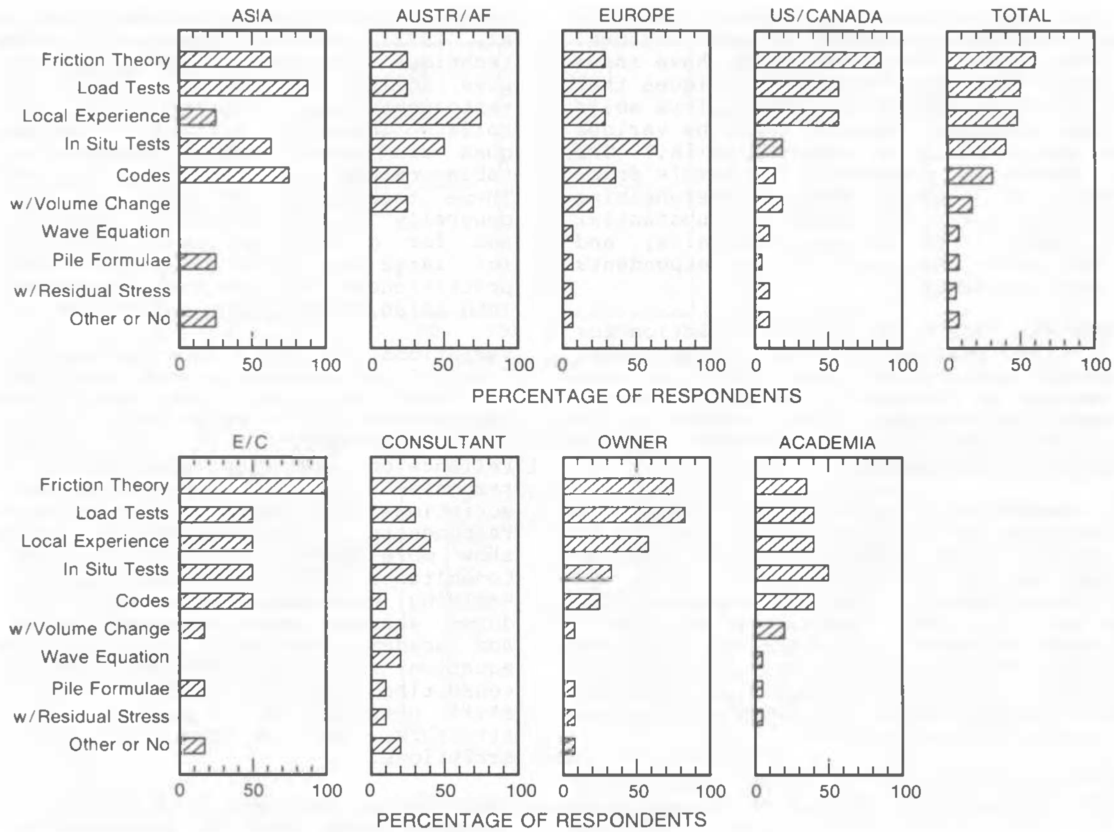

Fig. 5 Assessment Of Axial Compressive Capacity Of Driven Piles In Cohesionless Soils By Geographic Area And Type Of Practice

is typically one-third to one-half of that employed for driven piles. Several respondents indicated that the critical issue in their design of bored piles is the anticipated settlement, rather than the pile capacity. Two respondents stated that they have established limiting end bearing values in cohesionless soils in an attempt to limit anticipated settlements to less than 3 to 5 percent of the pile diameter (Reese et al, 1976). One respondent indicated a concern for end bearing capabilities of bored piles in rock that are concreted through a slurry. In this case end bearing is discounted because the quality of the bearing surface cannot be checked, but the full concrete-to-rock bond strength is used in the determination of shaft resistance. Other practitioners who design bored piles in soil or rock utilize semi-empirical procedures that consider both capacity and deflection, and that have been calibrated extensively for local geological and construction conditions (O'Neill and Reese, 1972; Williams et al, 1980). No consistent pattern in rules for the design of bored piles in rock or soil to carry static loads could be discerned, probably because of the significant dependence of individual designers on their knowledge of the influence of both local construction methods and local geological conditions on the capacity of bored piles in their area of practice.

Uplift Loads. Computation of static capacity of piles carrying uplift loads follows a reasonably consistent pattern. Most respondents compute uplift capacity from an integrated unit shaft resistance equal to that used for compression in cohesive soils and generally about 70 percent of that used for compression in cohesionless soils (McClelland Engineers, 1967). End suction is almost always discounted, but the dead weight of the pile itself is normally included. Group action is generally of more concern for piles subjected to uplift loads than to compression loads, especially for piles installed to a deep layer of high bearing capacity, because "block action" would be more likely in uplift than in compression.

Group Action. Concerning group action for piles loaded in compression, one question addressed special considerations given by respondents to predictions of group settlement and of group capacity efficiency. Figure 6 presents a pie chart distribution of replies relative to settlement prediction, and Figure 7 addresses the estimate of group efficiency. The majority of the responses for settlement prediction were vague, indicating either no answer to the question or that group settlement is considered in some unspecified way. Elastic interaction procedures (Banerjee and Davies, 1977; Poulos, 1979) are used by 18 percent of the respondents. Another 13 percent consider group settlement due only to consolidation of soft soils below the pile tips, usually calculated by assuming an equivalent footing at some depth within the group. The settlement of a fictitious shallow footing, usually larger in plan than the real group, is used by 9 percent of the respondents. A few indicated no concern for group settlement, but these are largely practitioners who routinely drive short piles to rock or a dense granular stratum, or who limit pile spacing to large values (i.e., more than 20 percent of the pile length).

With respect to estimation of group efficiency, 31 percent of the respondents either indicated that efficiency was of no concern or gave no answer, suggesting no concern. Many respondents in these two answer groups practice in geologic settings in which efficiency is not a major problem (that is, where rock or high bearing material is present at a relatively shallow depth). Most of those who compute efficiency do so in an unspecified way or by considering the possibility of failure of a block of soil within the pile group, especially in cohesive soils. For driven piles in cohesionless soils, the prevailing trend among respondents is to assume that efficiency is unity regardless of pile spacing, pile length or group size. Slightly lower efficiencies are often specified for groups of bored piles in cohesionless soils. No significant differences in assigned efficiencies between bored and driven piles in cohesive soils were observed in the data base from the questionnaire.

Only 12 percent of the respondents indicated that they consider a pile cap to contribute to the capacity of a closely spaced pile group. Those who do consider the contribution of the cap either use a subgrade modulus approach for design load or consider the cap to act as a spread footing if the piles are short or rigid. If long, flexible piles are used, cap reaction is entirely neglected. One respondent routinely designs piled raft systems where both the piles and the raft are presumed to provide bearing

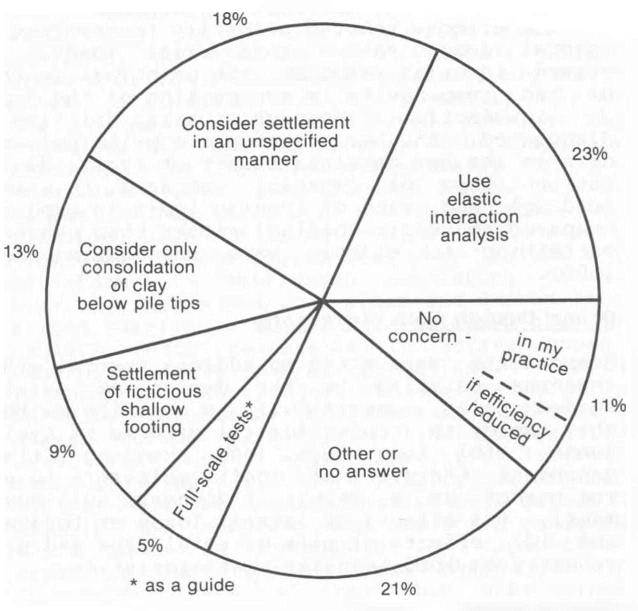

Fig. 6 Considerations For Estimation Of Group Settlement

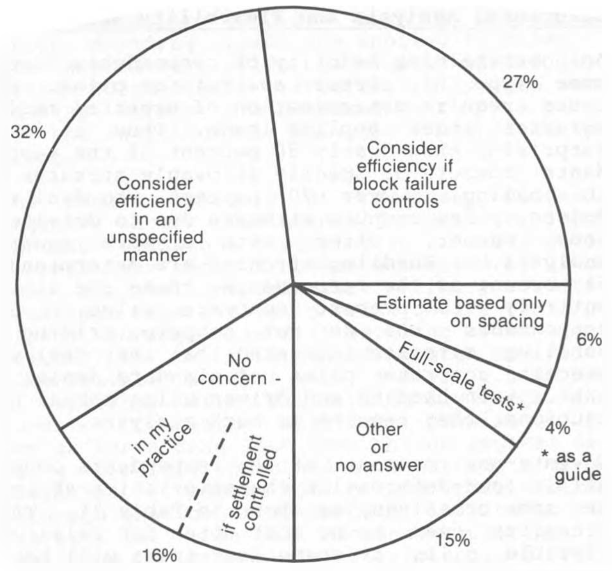

Fig. 7 Considerations For Estimation Of Group Efficiency

support. With such systems, the piles are widely spaced and the bearing surface for the raft is prepared to carry its component of the load. Preparation of the soil surface is characteristically not done for groups of closely spaced piles.

In summary, the state-of-practice in assessment of static capacity presently involves generally unsophisticated methods often coupled with local experience and individual logic supplemented by observations during construction. The authors conclude that many practitioners deliberately, intuitively, or unconsciously are following the oft-repeated admonitions of Terzaghi, Peck and others that local geology must be strongly factored into foundation engineering. Kezdi, in his 1965 general report on piles to the 6th International Conference, pointed out that "geology has influenced the development of soil mechanics in different parts of the world" and that "geologic conditions are responsible for (differences) in application of experiences in different parts of the world." More sophisticated techniques, involving detailed analytical or numerical modeling (Ellison et al, 1971; O'Neill et al, 1977; Esrig and Kirby, 1979), or special model tests such as in a centrifuge (Barton et al, 1983) tend to be used only when they are justified economically, that is, when the potential savings in reduced construction costs are considerably more than the increased cost of analysis or testing. Other justifications cited for utilizing sophisticated techniques of static capacity assessment, even when simple evaluation techniques are judged to be reliable, include the possibility of catastrophic failure consequences and the necessity to construct pile foundations whose capacities cannot be checked by static load tests or by dynamic measurements after setup (as for large-diameter offshore piles in clay).

Structural Analysis and Flexibility Modeling

An overwhelming majority of respondents conduct some type of stress analysis on piles. Many codes require determination of expected maximum stresses under applied loads. Thus, it is not surprising that nearly 80 percent of the respondents compute or specify allowable stresses due to loading. Over 70 percent who deal with driven piles compute stresses due to driving in some manner, often with a wave equation analysis. Handling stresses are determined by 54 percent of the respondents; these are limited entirely to precast concrete piles. Some respondents who do not compute driving or handling stresses indicated that they deal with precast concrete piles of standard design and that, when handled and driven using normal precautions, they require no such analyses.

Seventy-one percent of the respondents compute axial load-deformation characteristics at least on some occasions, as shown in Table II. These occasions seem to be most often for relatively flexible piles or where load tests will not be performed so that there are concerns about either pile-head movement or the possibility of progressive failure under load. The most frequently used method (52 percent) to predict pile head deformation is the "t-z" or unit load transfer function method (Coyle and Reese, 1966; Kraft et al, 1982) with both linear and nonlinear functions. Several respondents indicated that they supplement the "t-z" method with a preliminary calculation of residual stresses obtained from a wave equation analysis (Rausch et al, 1972; Holloway et al, 1978), especially for flexible piles in granular soils, which may develop high residual pile stresses (Briaud et al, 1983). Published elastic solutions including modifications for pile-soil slip (Poulos and Davis, 1980) are used by 36 percent of the respondents. Some respondents from Belgium and The Netherlands indicated that elastic solutions are not used in their practice because of questionable validity. Finite element modeling techniques are used by about 20 percent of the respondents for special problems, usually involving very large piles, piles in highly stratified soils, or offshore pile foundations. Scattered responses were reported for use of computed elastic tip movement plus pile shortening, numerical modeling based on Mindlin's solution, and consideration of only consolidation settlement. A total of 29 percent either do not compute pile head settlement or made no response.

TABLE II

Methods For Computation
Of Pile Head Deformation

Method	Frequency	Percentage
Load Transfer (t-z) Curves	29	52
Elastic Solutions	20	36
Elastic Tip Movement Plus Shortening	6	11
Finite Element Model	5	9
Numerical Model Based on Mindlin	4	7
Consolidation Settlement Only	1	2
Other, None, or No Answer	16	29

A following question pertained to the simulation of pile behavior for analysis of the superstructure. While nearly 60 percent of the respondents do not specify pile flexibility explicitly for superstructure designers, about one-third of the respondents develop an equivalent linear spring for the superstructure designer to represent the pile stiffness. About 18 percent of the respondents compute pile head settlement to satisfy themselves that settlements will be in the tolerable range for the class of structure being supported, especially for pile groups.

Design for Dynamic Loading

Respondents were asked how they designed or analyzed piles under dynamic loads induced by seismic events, ocean waves, or vibrating machinery. A large majority (61 percent) of the respondents do not address this problem at all. The most frequently referenced method of design/analysis (9 percent) is a simulation of piles as springs and dashpots calculated by elastic wave propagation theory and/or procedures that consider hysteretic damping in the soil under cyclic loading (Novak, 1974; Foo and Matlock, 1977; Roesset, 1980).

The small number of respondents making dynamic response computations despite abundant literature on the subject may be a reflection of a general lack of interest in or aptitude for such problems by classically-educated geotechnical engineers, or perhaps a genuinely low importance level of the problem in most pile foundation practice. Several respondents relegate such analyses to the structural engineer, stating that the concern of the geotechnical engineer is only the provision of soil parameter input.

Specific concerns expressed by respondents about seismic effects related primarily to response to lateral loads rather than axial loads. In regard to axial response, the principal concern of two respondents is recognition of the depth of liquefaction, above which axial friction is discounted. Another respondent's principal concern is the net combined effect of cyclic degredation (loss of capacity compared to static loading) and rate of loading (gain in capacity compared to static loading) rather than concerns pertaining to elastic wave propagation (Bea, 1980).

Other Design Considerations

Respondents were asked to address several other concerns relating to the design of axially loaded piles, specifically to describe methods they employ to account for (a) effects of cyclic loads, (b) long term load shedding (time-dependent transfer of shaft resistance to end resistance as a result of downward soil movement), (c) effects of lateral loads or torques, and (d) effects of pile material type and pile geometry on load transfer characteristics.

<u>Cyclic Loads.</u> Most owner/engineers from transportation authorities and power companies indicate that cyclic loading is not a concern as

long as the piles possess adequate factors of safety for static loading. Other owners and some consultants stated that, whenever they are concerned about cyclic loading tending to produce smaller capacities, full-scale cyclic load tests are conducted. This concern surfaces most often when unfamiliar soils are encountered. Two respondents indicate that they also employ model tests in such situations. One respondent in the U.S.S.R. has been acquiring in-service cyclic load data that indicates that displacements increase by as much as 30 percent over displacements under static loading for cyclic loads of moderate magnitude.

Most respondents who have concerns about cyclic loading account for soil degredation effects on axial capacity if either (a) the transient component of load is high (equal to or exceeding about 30 percent of the static component), or (b) the total load (static plus cyclic) is greater than about 90 percent of static capacity for one-way loading or 75 percent of static capacity for two-way loading. Solutions to these problems consist variously of increasing the factor of safety for static load by about 50 percent, computing the capacity for combined loads using residual shear strength parameters and making estimates of lateral effective stresses, and using boundary element and/or finite element models with appropriate constitutive laws for strain-softening soils. The latter solution appeared exclusively in responses from academia.

Load Shedding. The questionnaire requested a description of methods used, if any, to account for a changing load distribution from the pile to the soil due to long term load shedding or due to negative skin friction. The responses were distributed between two categories. The first category consists of considerations of reduced shaft resistance, or occasionally the assumption of downdrag or negative skin friction, even when no significant surface load would be applied at the site that would obviously produce soil consolidation. Only 25 percent of the respondents expressed any concern about this problem and then generally only when the piles being considered have a large component of end resistance. Four respondents consider the effect analytically by using "degraded" unit load transfer functions ("t-z" curves) to represent soft overburden soils for a flexible pile in a soil-structure interaction algorithm. Three stated that they neglect shaft resistance in overburden consisting of soft clays or peats and when piles are driven into a strong bearing stratum. Two indicated that they neglect shaft resistance in active zones of expansive soils and/or permafrost. Another respondent acknowledged the existence of load shedding produced by settlement of soil that had heaved during pile driving. But he also indicated that use of static capacities determined from site specific tests usually yielded capacities that are too low because full set-up has usually not occurred when tests are conducted, which tends to compensate for capacity that might be lost through load shedding. Two respondents revealed that they make dynamic measurements on piles retapped at a substantial time after they have been initially driven and conduct CAPWAP analyses (Rausche et al, 1971) to study redistribution of shaft resistance. Only one respondent actually uses a procedure in which downdrag loads are applied to piles. In his procedure, piles in closely spaced groups driven through soft overburden soils into strong bearing strata are presumed to carry a portion of the weight of the soil block within the pile group.

The second category of responses focused on negative shaft resistance created by imposed surface loads and resulting soil consolidation. Concern over this issue was expressed by 39 percent of the respondents. Primary conditions creating negative skin friction are (a) a pile placed through new fill settling under its own weight, (b) a pile placed through new fill on compressible natural soil, and (c) a pile under a building with extraordinarily high floor loads. Consensus solutions consist of (a) design for a drag load down to the neutral depth using the full undrained shear strength of soft clays or a percentage of the undrained shear strength (as low as 30 percent) in firm to stiff clays; (b) use of factors resulting from continuum solutions for flexible piles (Poulos and Davis, 1975); (c) design for downdrag loads by computing unit shaft resistance by effective stress methods, using $f = \beta\,\sigma'_v$, where β is approximately 0.25 for clays and clayey silts; and (d) destroy shaft resistance in settling zones by appropriate construction techniques.

Scattered responses included the use of finite element methods, the assignment of negative shaft resistance only where soil movement exceeds 50 mm, and the assignment of negative shaft resistance only for soils with low "c/p" ratios. The magnitude of the c/p ratio below which drag loads are considered was not stated, but it would presumably be in the typical range of 0.25 to 0.3 for normally consolidated clays. One respondent, who is responsible for design and maintenance of bridge foundations for a large state in the central U.S., indicated that negative shaft resistance had produced service problems for only two out of several thousand structures.

Lateral Loads and Torques. More comments (68 percent) were received concerning the possible effects of lateral loads and torques on axial pile capacity than on any of the other four issues considered under this subheading. Forty-two percent of those respondents stated that the effects of lateral loads and torques on axial capacity could be neglected when piles are placed in groups that are properly designed to resist the applied lateral, torsional, and vertical loads in a geometric sense. Another 16 percent discount axial resistance above a level of "fixity", which is usually selected as 5 to 6 pile diameters below the soil surface. One respondent in this group excludes or reduces shaft resistance in this zone only if cyclic lateral deflections exceed about 0.75 percent of the pile diameter, and another only discounts axial resistance in granular soils. Other methods of approaching the problem include the use of centrifuge model tests and finite element models for unusual problems, such as laterally cycled tension piles.

Another 21 percent consider the problem only in a structural context; that is, stresses are computed due to combined loading at critical sections and compared to allowable stresses permitted by codes. The possibility of initially bent piles is a concern to 3 respondents when computing stresses.

Pile Material and Geometry. A majority of those who responded to the question on pile material and geometry in the survey typically allow about 20 percent greater shaft resistance on concrete and timber piles than on steel piles, and another 20 percent increase the compressive shaft resistance for tapered piles over straight sided piles. One Canadian professor cited evidence that compression shaft resistance doubles in both cohesive and cohesionless soils when taper is increased from 0 to 7 percent.

Those practioners using analytical methods to assess the effects of taper in cohesionless soil indicated a preference for the method developed by Nordlund (1963). Design procedures based on the CPT used in Belgium and The Netherlands have taper and material effects included in the form of correction factors (Begemann et al, 1982).

Feedback

The questionnaire included a request to describe feedback procedures used to improve design (a) on a project-specific basis or (b) in general practice. Project-specific feedback is used informally by most respondents. The four-step logic sequence for selecting a factor of safety outlined earlier is an example of such feedback from improved soil data, pile tests, and construction observations cited by various respondents.

General practice feedback includes long-term performance monitoring and integration into the respondent's practice of lessons learned on various projects and through research. Only five respondents (9 percent of the total) described long-term studies that they are presently making of load transfer. These are on load shedding, negative shaft resistance, group behavior in uplift, cap-pile-soil interaction, and cyclic effects. However, long-term settlements are monitored by about one-half of the respondents, with owners being more likely to monitor settlements than engineers in other categories. Performance monitoring studies clearly impact the practice of those individuals performing the studies, but, unfortunately, results are more often not effectively fed back into the practice of their organization as a whole. In defense of this posture, one respondent suggested that too much effort has gone into generalizing test or performance data from only a few sites into a set of design rules intended for "universal" application. The study by Dennis and Olson (1982a, 1982b) and comments by several respondents show that the vast majority of pile load test results and performance records are essentially useless to the profession as a whole due to a lack of or incomplete description of one or more critical parameters, most often the parameters describing soil strength. This criticism is essentially unchanged from the General Report of Peck (1953) at the 3rd International Conference.

Ineffective feedback of lessons learned continues to be, in the opinion of the authors, a major weakness in the pile design process on a global basis. Success seems to be limited to the application of practical observations to empirical procedures for local areas of relatively uniform geology, soil stratigraphy, and soil properties.

CONSTRUCTION

Driven Piles

Installation Aids. Respondents were asked several questions regarding installation of pile foundations, particularly with respect to quality assurance. First, a response was solicited regarding the permissible use of installation aids, such as jetting or predrilling. There was a wide diversity of replies, ranging from no concern at all for the problem to altering design procedures to completely discount shaft resistance in the zone of jetting or predrilling. The responses are summarized in Table III. The percentages noted for each response are based on 48 respondents, who indicated that they deal with driven piles.

TABLE III

Permissible Use Of Installation Aids

Method	Frequency	Percentage
To Facilitate Penetration	24	50
To Reduce Soil Heave or Lateral Movement	11	23
Routinely Controlled by Engineer	5	10
Generally Not Allowed	4	8
To Reduce Noise or Vibration	4	8
Through Surface Fill	1	2
Other, None, or No Answer	15	31

Whether installation aids are used or not (and the particular nature of the aid permitted) appears to be very much a matter of local practice and experience. For example, where closely spaced piles are driven through low strength, saturated clays into hard bearing materials, predrilling is viewed by several respondents as being a positive factor in the control of heave and potential lifting of piles from the bearing stratum. Installation aids are also generally accepted as necessary when near-surface obstructions are encountered, when a resistant layer is present at a shallow depth above a weaker formation to be penetrated, and when significant numbers of piles are being driven on a slope.

Predrilling was shown by individual written responses as the most acceptable installation aid. Respondents who adjust pile capacities for predrilling effects indicated that the shaft resistance either is not reduced or is reduced only by a small amount (less than 30 percent) if the pilot holes are undersized.

Controlled jetting is utilized without significant capacity penalty, particularly in granular soils, mainly by owners in the U.S. and especially if tapered piles are used and/or if the piles are driven several meters past the maximum depth of jetting. On major projects jetting effects are evaluated by analyzing driving records and/or by conducting static load tests. There are other owners and/or consultants (principally in Asia) who do not permit jetting or who assign zero shaft resistance to the maximum depth of jetting, with the latter choice appearing in geographic areas where granular soils are frequently cemented in some fashion.

Internal jetting is sometimes employed in large diameter offshore piles, in which case a capacity penalty is usually not assessed.

Installation of Groups. Many respondents expressed concern over the effects of driving piles in closely spaced groups, and predrilling is frequently employed to reduce soil and pile heave. Figure 8 presents the procedures and observations most frequently employed for quality control of closely spaced driven piles. The monitoring of heave of all piles followed by retapping of those that have heaved more than a specified amount, usually 10 to 50 mm, was the most frequent response, coming from 38 percent of the respondents. A few respondents expressed a need to assess the causes of heave prior to undertaking a correction program. The causes indicated in the responses range from general deformation within the major bearing stratum, which usually is of minimal concern, to uplift produced in overburden soils, which is of major concern for piles deriving capacity primarily by end bearing.

Measurement of ground surface movements is employed by 27 percent of the respondents; this technique can sometimes indicate the probable cause of heave. Correction of heaved piles usually consists of redriving them to their original butt elevations in order to reseat the tips. Thin-walled cast-in-situ shell piles that have not been filled with concrete can be retapped to their original tip elevations before filling, which requires the use of telltales or other similar monitoring techniques. For piles that can not be retapped, such as expanded base piles, spacings are generally limited to about six shaft diameters, and at least one percent of reinforcing steel is added to the shafts to resist tensile stresses produced by overburden heave.

Figure 8 indicates that only 15 percent of the respondents occasionally or routinely monitor excess pore water pressures in the soil mass within or surrounding the pile group. Generally such monitoring is done for groups of piles on clay slopes with the purpose of monitoring the stability of the slope. These responses suggest relatively little concern that group efficiency in clays may be influenced significantly by the rate of pore water pressure dissipation (O'Neill, 1983).

Capacity Assurance. Methods for field verification of static pile capacity during production were the object of another question. Figure 9 shows the distribution of responses from the respondents dealing with driven piles. This chart shows equal numbers of responses for driving to a specified blow count and for conducting load tests. Individual comments revealed that many respondents utilize these two procedures in combination rather than separately. Static load tests are conducted, either as a direct design step or during production to verify static calculations, and then production piles are driven to a final blow count derived from the test piles. This practice is presumably more prevalent where piles are not driven to rock or to a near-impenetrable stratum and for in situ piles formed by driving low slump concrete to form a bearing bulb. The third most

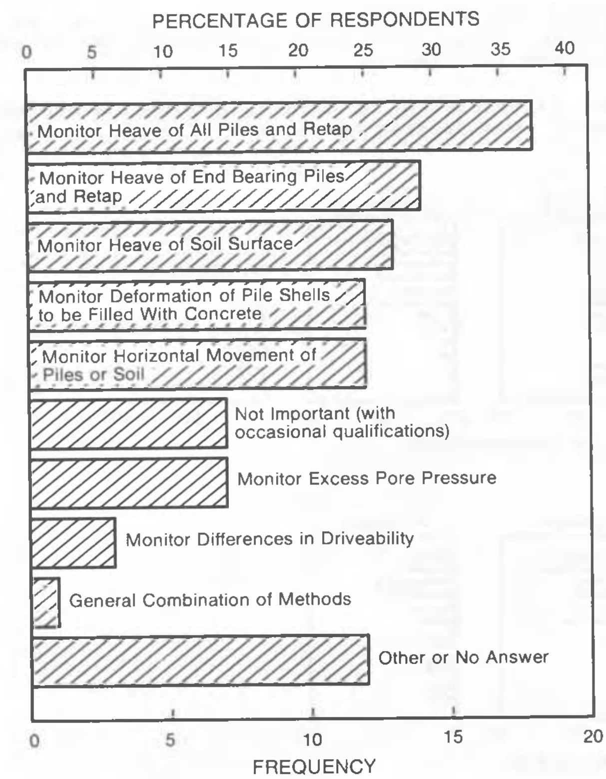

Fig. 8 Observations During Driving Of Pile Groups

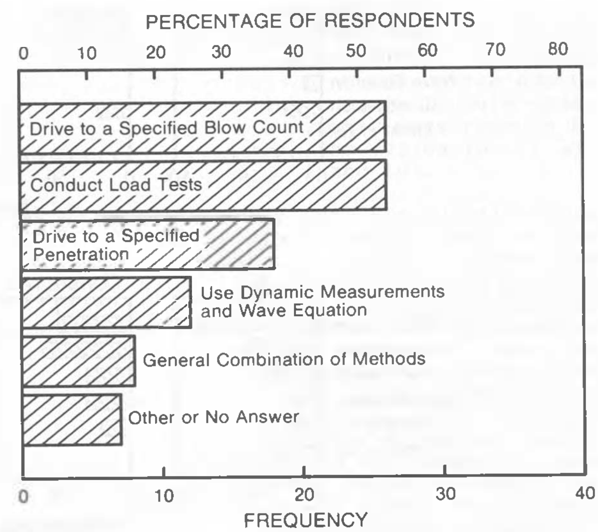

Fig. 9 Verification Of Static Capacity During Pile Driving

popular procedure is to drive piles to a specified penetration; respondents following this procedure tend to be active primarily in geographic areas onshore where the prevailing practice is to drive piles to rock or other well-identified strong bearing strata.

Only 25 percent of the respondents indicate that they verify capacity during driving by making dynamic measurements of strains and accelerations at the pile head followed by systematic computation of pile capacity from such measurements (Rausche et al, 1972; Vines and Amar, 1980). Two respondents who utilize dynamic measurements to assess capacity suggest that the method is inappropriate for clay profiles but is reliable in granular deposits because the method works best when time-dependent load transfer is minimal and where the greatest proportion of driving resistance is in end bearing. One U.S. geotechnical consultant indicated that, although he was personally satisfied with available dynamic stress wave methods for assessing capacity (if used during retap), considerable reluctance to substituting dynamic tests for static tests exists among owners and building officials.

Differences in the pattern of procedures is evident when replies are broken down by geographic area and type of practice (Figure 10). The relative preferences by owners and by respondents in the U.S. and Canada form a trend very similar to that of the total. In Asia only modest reliance is placed on final blow count during driving or any procedure other than a static load test to verify static capacity. Final blow count is the primary verification technique in Europe, influenced by almost exclusive reliance by Belgian and Dutch respondents on penetration resistance rules developed from CPT tests. Engineer/constructors also rely heavily on the blow count pattern and final blow counts. Consultants and academicians have a preference for load tests over final blow count. Owners, perhaps reflecting offshore practice of oil companies, indicated that piles are frequently driven to a predetermined penetration based on an office static analysis. In the U.S., some engineers in the transportation area use pile driving formulae in lieu of wave equation analyses for selection of a desired final penetration resistance. Several respondents use a combination of methods, either formally or informally.

Although no specific methodologies were provided by respondents, Figure 11 outlines a formal acceptance logic for production pile control developed in the senior author's firm that utilizes both static capacity computation and dynamic measurements. The reconciliation procedure referred to in Figure 11 typically involves analysis of driving records in comparison to the anticipated stratigraphy based on site investigations. When warranted and economically feasible, new borings or penetration tests may be made adjacent to questionable piles. If the stratigraphy is indicated by the driving record or new borings to be different from that assumed for the static capacity computations, new computations are made for revised layer dimensions or indicated soil types. As an alternate to dynamic measurements and wave equation analyses, blow counts may be related to blow counts on piles that have been statically load tested at the site or to blow counts on "indicator" piles that have been dynamically monitored at the site and whose capacities have been computed from wave equation procedures. Even if a formal acceptance logic pattern is not followed, it is the practice of many respondents specifying load tests as a means of capacity evaluation to select piles with questionable driving records for the load tests.

An alternate philosophy to a formal acceptance logic was offered by two North American consul-

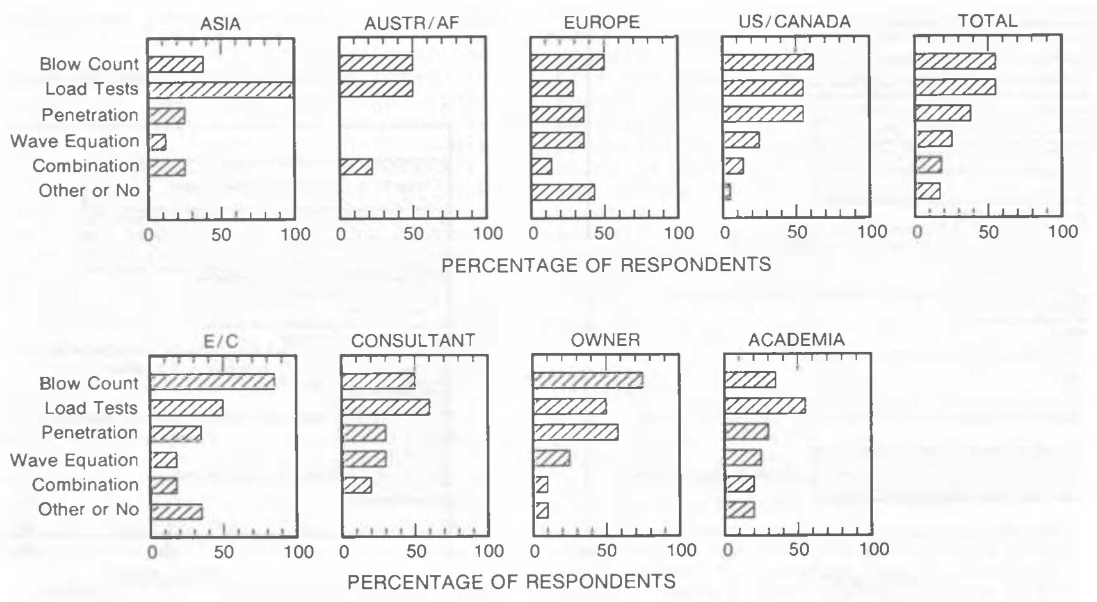

Fig. 10 Verification Of Static Capacity During Pile Driving By Geographic Area And Type Of Practice

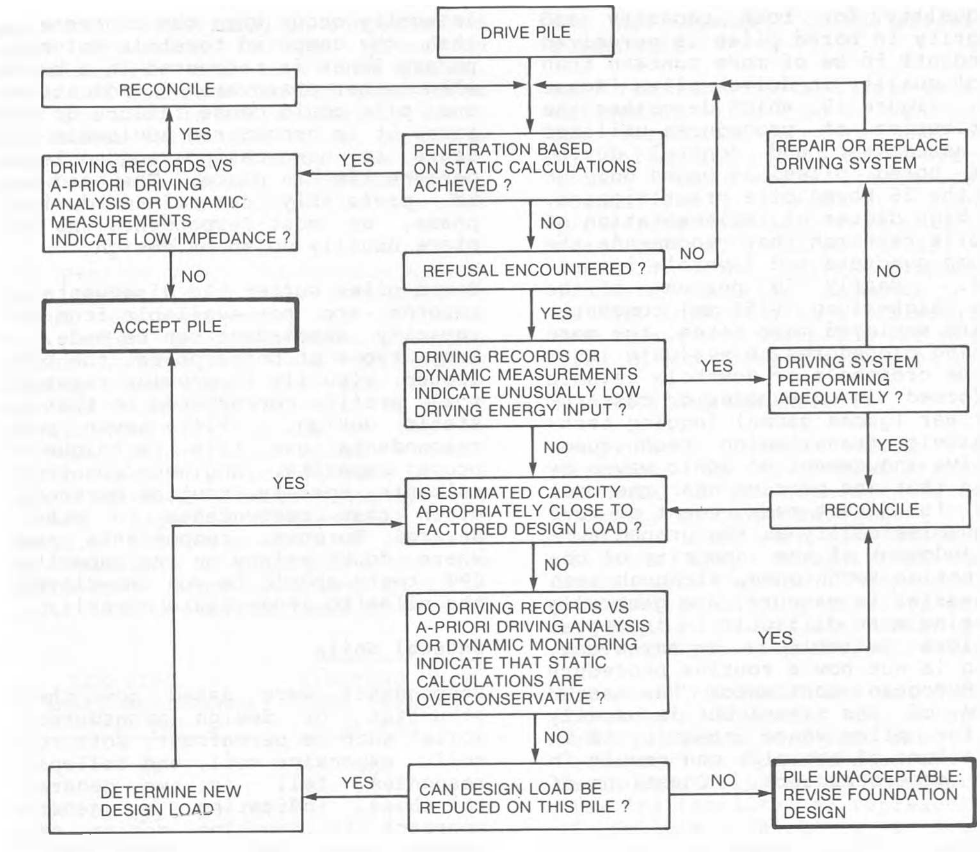

Fig. 11 Acceptance Logic

tants involved exclusively in onshore practice. Their statements are that each pile installation project is unique and that fundamental reasons for unexpected behavior should be addressed on a problem-by-problem basis by engineers who understand the phenomena involved. They believe that attempts to over-formalize the acceptance process can be counter-productive. These individuals also warned that reliance on a single capacity verification method is not good practice.

The response to a following question revealed that the most common revision in a foundation design based on adjustments of assigned capacity during the installation process is to modify the penetration of future piles, particularly when the capacity is primarily generated by skin friction or when the position of a bearing stratum is variable. Only occasionally is the number of piles revised, because such a change usually involves a change in design of the pile, superstructure, or cap. About 10 percent of the respondents, who utilize piles that cannot be easily spliced, may adjust the number of piles. Also, the base size of driven bulb-base piles can be increased if capacity is believed to be inadequate according to energy input rules (Nordlund, 1982). Numerous respondents stated that alterations in foundation design must be a last resort since it usually causes difficulty in the working relationship between owner, contractor, geotechnical consultant, and engineer.

Verification of the structural integrity of driven piles is another part of the overall capacity evaluation process. Four techniques of integrity evaluation are each employed by 50 to 60 percent of the respondents. These are (a) observe piles for evidence of cracks or curvature, (b) compare blow counts against blow counts computed from wave equation, (c) conduct load tests on suspicious piles, and (d) monitor dynamic strains and accelerations on representative or suspicious piles. Practitioners appear more prepared to use dynamic measurements for integrity evaluation than for evaluation of load capacity (see Figures 2, 4 and 10).

Bored Piles. One-half of the overall survey sample indicated that their practice was partially or completely concerned with some form of bored piles. Construction of bored piles in granular soils by use of bentonitic slurry has become increasingly common in Southwestern United States, Belgium, Germany, and The Netherlands. Most respondents who expressed any opinion with regard to installation-related group effects for such piles were concerned about construction in granular deposits. Detrimental effects are reduced by installing bored piles in a staggered sequence and/or by installing newly placed piles to slightly shallower depths than those already installed in order to maintain relatively large end bearing capacities.

Assurance of quality for both capacity and structural integrity in bored piles is perceived by many respondents to be of more concern than the assurance of quality in driven piles (Reese et al, 1981). Figure 12, which describes the frequency distribution of procedures utilized for integrity assessment and control during installation of bored piles, is based only on responses from the 26 bored pile practitioners. It suggests a high degree of implementation of recent bored pile research that recommends the use of high-slump concrete and downhole inspection techniques. Nearly 70 percent of the respondents use high-slump (150 mm) concrete. Because they are employed most often, the more successful testing procedures to evaluate integrity appear to be crosshole or downhole seismic techniques performed in core holes or cast-in-place tubes, nuclear (gamma gamma) logging techniques, and seismic transmission techniques. The latter involve inducement of sonic waves by pulse generators that are precast near the base of bored piles for direct measurement of wave velocity and transmissibility at the ground surface to permit judgment of the integrity of the pile. Wave reflection techniques, although less expensive and easier to execute, are generally perceived as being more difficult to interpret and therefore less reliable. In any event, integrity testing is not now a routine procedure (although one European contractor has tested several thousand of its sites) but is usually employed only for piles where integrity is in doubt or where failure of one pile can result in failure of the superstructure. Questions of

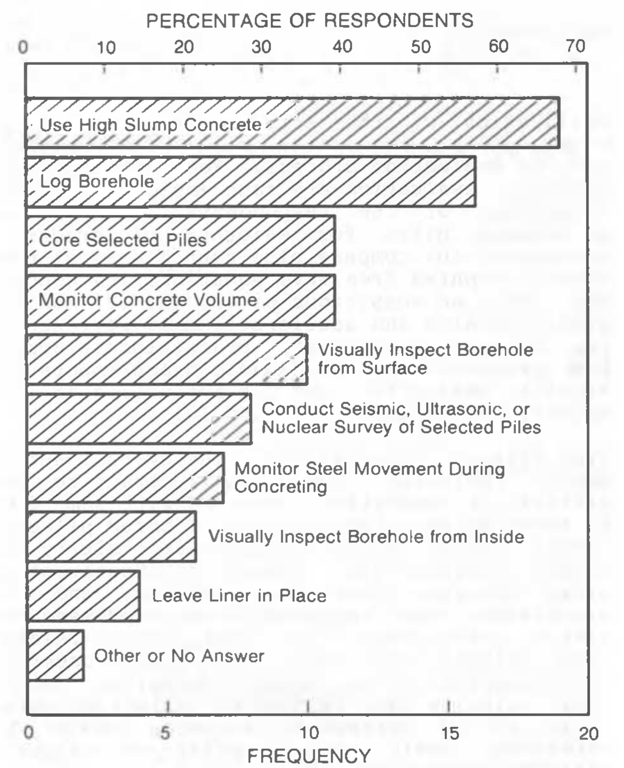

Fig. 12 Integrity Assessment And Control For Bored Piles

integrity occur when the concrete volume is less than the computed borehole volume, where a temporary liner is recovered in a buckled state, or when other observations indicate a problem. If one pile could cause failure of the superstructure, it is considered advisable by many respondents to use casings and to leave the casings permanently in place. The need for such action is preferably determined during the design phase, as most respondents who leave casing in place usually do so by design.

Bored piles suffer the disadvantage that driving records are not available from which a general capacity assessment can be made. However, with many types of bored piles, the boreholes can be logged visually to provide reassurance that the soil profile corresponds to that assumed in the static design. Fifty-seven percent of the respondents use this technique to help assure proper capacity. Engineer/constructors and consultants specify routine borehole logging more often than respondents in other categories. Several European respondents recommend that, where doubt exists on the capacity of the soil, CPT tests should be run immediately adjacent to the piles to investigate capacity.

Unusual Soils

Respondents were asked how they altered construction or design procedures in "unusual soils" such as permafrost, soft rock, calcareous soil, expansive soil, and collapsing soil. The responses fell in two general categories: (a) those indicating a general procedural approach to insuring design capacity in all unusual soils, and (b) those indicating specific design or construction considerations for particular types of unusual soil. In the former category most respondents indicated that they would develop construction and/or design procedures on a site-specific basis by installing test piles by various means and then conducting static load tests, often on instrumented piles, as appropriate. A few respondents would permit substitution of capacity or integrity evaluation by use of dynamic monitoring and wave equation analyses to reduce or eliminate static load tests. In addition, respondents reiterated a need for better soil characterization than is usually required for well-known soils. There was, however, considerable disagreement on how to develop such characterization. This disagreement was centered around the acceptability of laboratory tests vis-a-vis in situ tests, with no clear preference apparent.

The responses pertinent to specific types of unusual soils will be discussed in sequence. Very few comments were received relative to the construction and design of piles in permafrost. One respondent indicated that proprietary methods are available and are being employed in his practice. Another indicated that he designs piles by assessing the bonding characteristics of the permafrost before determining shaft capacity (presumably disallowing shaft resistance in unbonded permafrost). The tendency for upward jacking forces created in the frost-active zone is considered.

For soft rock, most responses were concerned with construction methods and control. For example, careful consideration should be given to tip protection, especially on concrete and timber piles. Attention is also given to evidence of relaxation and heave in shales. Most respondents use the same general procedures for design that are used in stiff cohesive soils such as pressuremeter rules, CPT rules, or alpha factors. A few respondents, mostly from academia, suggested that local experience is necessary in order to apply such rules correctly.

The responses pertinent to "calcareous soils" indicated that there is no clear consensus on the definition of this term, which was variously interpreted to refer to chalk, cohesive soils with calcareous concretions, and calcareous sands (presumably depending upon the respondent's geographical area of practice). No great concern was expressed for soils in the two former categories, other than those already cited for "soft rock." Considerable uncertainity exists regarding the most appropriate construction and capacity assessment for calcareous sands. A sliding scale of limiting shaft and base resistance of pipe piles that are driven in calcareous sands has been given by Agarwal et al, (1977) and is used by several practitioners:

PERCENT CARBONATE CONTENT	LIMITING SHAFT RESISTANCE (k Pa)	LIMITING UNIT END BEARING (k Pa)
$\leq$30	96	960
30-45	31	770
>45	27	570

Several respondents indicated that these general values are used in their practice, but if the construction procedure is altered to permit grouting of a pile into a oversized pilot hole, a shaft resistance value of 96 kPa can be used for any carbonate content. The interaction of piles and calcareous soils is indicated by the respondents as very poorly understood, and significant deviations from tabulated values are possible. Crushed calcareous soil may recrystalize and bond firmly to the pile under some combinations of soil and groundwater physico-chemical conditions, thus greatly increasing pile capacity. In other soils, piles driven open ended create a "cookie cutter" action where the pile and soil are not adhesively bonded and where lateral earth pressures are extremely low, possibly even reducing shaft resistance values below those presented above. Conservatism is advocated without the assurance of conservatism being achieved.

Responses pertaining to expansive soils were largely associated with bored piles. The most frequent response was that shaft resistance is discounted in the zone of seasonal moisture change (depth of 2 to 6 meters). Those respondents who practice in environments conducive to extreme shrink-swell behavior (South Africa, Australia, Texas) either estimate the shear strength of the soil in the zone of moisture change and apply the resulting shear stress as an upward force on the pile to be resisted at a lower level (O'Neill and Poormoayed, 1980) or "sleeve through" the expansive zone taking care to provide watertight boundary materials between the pile and the soil. Generally, respondents did not comment on the effectiveness of these procedures.

Very few comments were made relevant to piles in collapsing soils such as alluvial fans or loess. One respondent suggested that piles not be used in collapsing soils but prefers instead to use ground improvement procedures and shallow foundations. Another respondent suggested protecting the soil from moisture variation and then utilizing bored piles. Other responses included using driven displacement piles in a staggered driving sequence to compact the soil so that interstitial piles would not be affected and designing the piles for negative skin friction in and above the zone of collapsing soils.

ACCURACY AND RELIABILITY

Respondents were asked "considering the variables in the present state of practice, how reliable do you consider the estimation of pile capacity and pile settlement to be in your practice." They were also asked to comment on how reliable they thought that the imposed service loads were known. Responses to the perceived reliability of pile capacity estimates are given in Figure 13 and are broken down by geographic area and type of practice in Table IV.

The word "reliable" was not defined in the questionnaire; however, an example was given in which reliability was used in the sense of "accuracy" or "probable error." Several respondents were critical of this wording because they view "reliability" as representing a probability of failure, which is a separate issue from accuracy of capacity, settlement, or load estimates. Nevertheless, most respondents did estimate errors in assessment of capacity, settlement, and load corresponding to the procedures followed in their practice. Based on comments received, the "errors" reported probably approximate each respondent's ±90 percent confidence limits. Just over 80 percent of the respondents expressed an opinion on capacity; most of these were fairly evenly distributed in the brackets of 10 to 20, 20 to 30, and 30 to 50 percent error. Seven percent indicated a probable error larger than 50 percent. Despite the uncertainty on the part of the authors in interpreting the responses, we

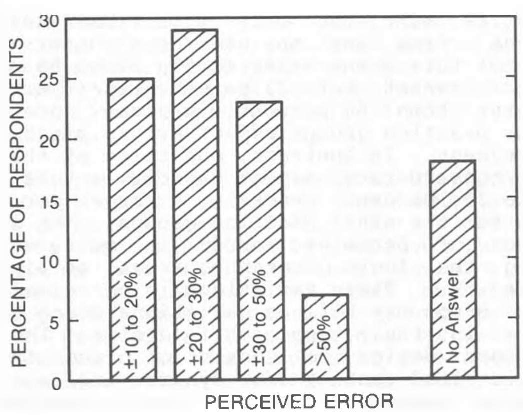

Fig. 13 Perceived Reliability Of Pile Capacity Estimates

TABLE IV

Distribution Of Perceived Reliability Of Pile Capacity Estimates By Geographic Area And Type Of Practice

PERCENTAGE OF RESPONDENTS

Respondent Category \ Bracket of Perceived Error	±10 to 20%	±20 to 30%	±30 to 50%	>50%	No Answer
			By Geographic Area		
Asia	0	40	20	20	20
Austr/Af	25	0	25	0	50
Europe	41	18	23	6	12
US/Canada	25	33	21	4	17
			By Type of Practice		
E/C	34	22	0	22	22
Consultant	27	27	27	0	19
Owner	22	14	29	0	14
Academia	23	32	23	0	18
Total	25	27	23	7	18

judge that the average perceived error is about 25 to 30 percent for all types of piles.

Eleven respondents gave alternate error estimates. Eight indicated a belief that errors are larger in cohesionless soils (average error of nearly 50 percent), with only one suggesting errors are larger in cohesive soils. Three expect larger errors with bored piles than with driven piles. Dutch and Belgian practitioners state that their high degree of confidence in capacity predictions is related to the reliability of CPT data and would have less confidence for other situations.

The distributions of perceived reliability based on geographic area and type of practice are shown by Table IV. The pattern of responses is fairly consistent among the different types of practice with the only significant deviation coming from the engineer/constructor group. None of this group selected the 30 to 50 percent error bracket but 22 percent expect errors of greater than 50 percent, whereas none of the other practice groups expect errors greater than 30 percent. In contrast, one-third of the engineer/constructors expect capacity errors in the 10 to 20 percent bracket, a larger percentage than for the other practice groups. The distribution of perceived error is quite variable among the four geographic areas, as listed in Table IV. These variations in perceived capacity error may reflect variations among respondents in their approach to design. Those who consider design to consist of preconstruction static load tests and/or dynamic indicator pile driving tests were generally more confident of their ability to estimate capacity than those who generally do not perform such tests. Furthermore, European practitioners who have developed in situ test rules for their area have the greatest confidence of all respondents in their ability to predict capacity. Their responses were all in the 10 to 20 percent category. One U.S. respondent cautioned that general estimates of error are not meaningful. Instead, he argued that the goal of the designer is to consider the site conditions and construction methods, and then to develop a reliable lower bound for capacity. The data and comments contained herein may serve towards this objective.

Less confidence in the prediction of settlement was reported than in the prediction of capacity: 14 percent of the responses in the 10 to 20 percent range, 16 percent in the 20 to 30 percent range, 13 percent in the 30 to 50 percent range, and 23 percent expecting over a 50 percent error. There was no trend toward greater uncertainty with any given soil type or with bored piles in comparison to driven piles. One respondent felt that large percentage errors in settlement of individual piles were a relatively minor concern because settlements are usually small. More concern was raised about settlement of pile groups, for which more reliable soil mechanics procedures for estimating settlements are available.

The vast majority of respondents made no estimate of errors in the design loads that the piles will have to carry, suggesting that the development of such information is not within the purview of geotechnical engineers. The few answers received indicate a belief that loads are overestimated by 20 to 30 percent. A respondent from the U.S.S.R. has extensive measurements indicating that most loads are overestimated by an average of 20 percent. One respondent from The Netherlands has measured errors in pile loads as large as 50 percent in statically indeterminant structures but has found that such errors are consistently smaller in statically determinant structures.

The manner in which a margin of safety is established is perhaps more related to reliability. Seventy percent of the respondents use an overall global factor of safety for pile foundation design. Partial factors of safety are used by 27 percent of the respondents, and 5 percent use a formal reliability approach for foundations of critical structures such as offshore platforms in a seismically active region.

Global factors of safety typically range from 2.0 to 3.0 for onshore projects to relatively standard values for offshore projects of 2.0 for operating conditions to 1.5 for extreme environmental loading conditions. In seismically active areas, the trend is to assign a factor of safety ranging from 1.2 to 1.5 for the static load plus the increased load due to dynamic loads imposed by the structure on the pile foundation. One respondent suggested that the factor of safety be chosen based on superstructure flexibility. He recommended global factors of safety of 3.0 for stiff superstructures, which can promote progressive failure of foundation piles, and 1.5 for flexible superstructures.

Several respondents suggested that the choice of a factor of safety logically should be based on the amount of information about the soil, the piles, and the installation equipment as it becomes available to the engineer. Therefore, the magnitude of the safety factor may vary during various design stages of a project, as suggested in an earlier section. Most of these respondents felt that global factors of safety should be no lower than 2.5 onshore if the design is based only on a static analysis, perhaps higher if the site geology is not familiar to the designer. Global factors of safety are decreased 10 to 20 percent by some respondents for short term loads, increased by 10 to 20 percent for tensile loads, and also increased for two-way loads. One respondent also indicated that he varies the global factor of safety based on the type of pile used (1.7 for tapered piles vs. 2.0 to 2.5 for concrete and expanded base piles). Another approach to the selection of global factors of safety is to use 2.0 for the worst combination of dead and live loads, and 3.0 for dead loads and permanent live loads, establishing pile capacity on the more critical of the two conditions. This approach corresponds to that in offshore practice of using 2.0 for operating loads and 1.5 for maximum environmental loads. Two academicians are beginning to define limiting service loads (as opposed to "failure loads") based on load tests and analytical methods, especially for large-diameter piles. With such service load limits defined presumably in connection with the nature of the supported structure, load factors as opposed to safety factors are established which allow the foundation to be included in an ultimate strength design of the total system. Load factors are generally taken to be in the range of 1.35 to 1.5 for dead load and 1.8 to 2.0 for live load. While establishment of limiting service loads and corresponding load factors appears logical from the structural analysis point of view with a trend toward ultimate strength design, relatively few respondents advocate this approach. Those who oppose it cite the lack of accuracy with which load-settlement behavior of piles can be forecast given all of the uncertainties inherent in the design and construction of pile foundations.

The preceeding discussion of global factors of safety pertains to soil failure. Most respondents are of the opinion that factors of safety against pile material failure should exceed those for soil failure by a factor of as large as 1.5 to account for misalignment, unanticipated bending due to obstructions, lateral soil creep, and other similar effects. Many also advocate higher material factors of safety to account indirectly for stresses induced during driving, which are almost always higher than stresses imposed by structural loads. More logically, driving stresses should be analyzed separately, and material factors of safety should be based on the considerations enumerated above.

Partial factors of safety are employed in two general ways: (a) individual factors for base and side resistances are applied separately to the respective capacities and the resulting "allowable" capacities are added together to obtain an "allowable" pile load, and (b) separate factors of safety for base and side resistances that express the uncertainty of such resistances tied to an overall factor of safety against pile collapse. In the former approach, usual factors for end resistance are in the range of 2.5 to 6.0 (with the largest factors for large-diameter bored piles), and usual factors for shaft resistance in the range of 1.5 to 2.5, with no significant observable difference based on pile type or diameter. The selection of larger factors of safety for the base is generally an acknowledgement of the widely accepted hypothesis that a pile base must settle more than a shaft before failure is reached. An example is the selection of allowable load for a relatively shallow underreamed bored pile in cohesive soil from

$$(Q)_{allowable} = (Q_{ultimate})\ \text{shaft} + \frac{1}{F}\ (Q_{ultimate})\ \text{base}$$

where F is about 3 (Reese et al, 1976). The implied factor of safety for the shaft is 1.0 because it will have developed its maximum resistance by the time the base load reaches one-third of its ultimate value. With this approach an overall factor of safety in the order of 2 to 2.5 is also enforced.

Other partial safety factors were described. One respondent applies separate factors of safety to the soil parameters of ϕ and c, and to load. Another respondent uses formal reliability approaches in which partial factors are chosen based on loading details, superstructure details, soil environment, and the procedures that have been used to analyze the piles deterministically.

CODES AND STANDARDS

Building codes are classified by Fuller (1979) as either "prescriptive" or "performance" codes. Prescriptive codes give generally detailed procedures for assessment of pile capacity, material quality verification, and construction quality control. Performance codes specify only how the foundation is to perform but not the details of design and construction, which are left to a presumably qualified engineer. The ideal code as argued by Fuller should be at least partially performance-based, since the state-of-the-art is not advanced sufficiently to permit code-writing agencies to include requirements that insure both economy and public safety in all situations. On the other hand, a purely performance-based code requires specialists in the enforcement agency for evaluation and does not insure that unqualified engineers will not design either inadequate or ultraconservative foundations.

Only 14 percent of the respondents indicated that their practice is not constrained in some way by codes. A total of 29 different codes and standards were specifically mentioned by the 48 respondents who feel some code restraint. The responses indicate that the overwhelming majority of these codes are prescriptive in form. Seven codes in complete or extract form were returned with the responses. While each of these codes would be classified as prescriptive, each permits the engineer alternative procedures, provided they are rational and backed by evidence that the procedures are appropriate. The American Society of Civil Engineers has proposed a standard for pile foundations that is weighted towards performance (ASCE, 1984).

Twenty-five percent of the respondents who feel constrained by codes further believe that their codes discourage innovation. Several respondents in North America indicated that the codes themselves are not unduly constraining, but that enforcement agencies and owners, increasingly concerned with legal issues and lacking technical understanding of foundations, are using codes to constrain engineers and to dampen the rate of integration of state-of-the-art technology into practice. It is apparent that geotechnical engineers need to become more involved in the preparation, revision, and enforcement of codes and standards that pertain to pile foundations.

RESEARCH

Our state-of-practice advances only as and when the state-of-the-art advances. Respondents were asked to describe those specific elements of deep foundation research performed in the past decade that have had the greatest impact on their present practice. Ninety-three percent provided one or more replies. The most significant research was perceived to be the development of more or less standard procedures for dynamic monitoring and wave equation analyses for capacity and integrity evaluation, as noted by 12 respondents, which was 21 percent of the total. Three other activities that were each nominated by about 10 percent of the respondents are: (a) analytical studies of complex pile-soil interaction problems leading to simple equations and graphs (especially for settlement predictions), (b) development of effective stress procedures for prediction of pile capacity, and (c) acquisition and synthesis of data on performance of pile groups.

Nominations were also requested for those areas in which respondents perceived the greatest need for future research. A total of 27 different topics were submitted. Three generalized recommendations were suggested in one form or another, each by 20 to 27 percent of the respondents. These are:

(a) conduct definitive experimental studies, preferably in the field, on the effects of loading rate and cyclic loading, especially as they influence uplift capacity and long-term settlement,

(b) conduct assessments of the reliability of dynamic driving measurements and analyses to predict the static capacity of piles and modify the techniques if and as necessary, and

(c) develop better methods to evaluate the integrity of bored piles.

Many of the other suggestions, while directed at different objectives, involved the acquisition of high-quality full-scale performance data, including load transfer information and accurate, statistically-significant soil property data.

The general nature of the responses suggests a lack of confidence in present static design methods and a positive desire to apply more fundamental principles of soil mechanics to static capacity evaluation. It also reflects on the past short-sightedness of many owners, research agencies, and geotechnical consultants themselves who have not collected test and performance data nor conducted analyses of observed pile behavior in a manner as to be relevant, not just to the specific foundation project at hand but also to the advancement of the state-of-the-art and the state-of-the-practice. Perhaps too much research emphasis has recently been placed on the study of pile-soil interaction in the context of mathematical models that are conveniently formulated in terms of conventional mechanics but which are frequently based on axioms that have an inadequate or improper physical basis for soil mechanics. The abundance of empirical design methods is evidence of the fact that we as a profession do not fully understand what happens physically to an element of the soil and to the assembly of elements that constitutes the soil profile when a pile is driven or bored into place, as time passes, and as the pile is later subjected to a random pattern of loads. Phenomenological understanding must be obtained through appropriate (probably full-scale or centrifuge) experimentation prior to or at least in parallel with the development of improved mathematical models if such models are ultimately to represent properly as complex a problem as soil-pile interaction.

The senior author has been advocating (Focht and Kraft, 1981; Focht, 1983) that more research attention be given to predictive models relying on t-z curves rather than models that sum peak capacity contributions from segments of the pile shaft and end area. Such models will require improved understanding of the constitutive properties of the thin, highly remolded soil zone immediately adjacent to the pile. Recognition of this need is not new, having been pointed out in previous international conferences. Kezdi (1965) wrote "the addition of parts taking the sum as the ultimate bearing capacity is definitely a bad procedure" and "this process is not independent of the manner of installing the pile or of construction methods." Then later, "We have to find out the changes in installation of a pile or any other element of a deep foundation will cause in the soil and how this will influence its behavior." At the 8th Conference, Tomlinson (1974) in a discussion wrote, "It must be recognized that .... we are concerned with the failure condition which is not one of relative movement between the pile and the adjacent soil but is one of slipping between the skin of soil carried down by the pile and the adjacent soil which is itself heavily sheared and compacted." In the panel discussion at the 10th Conference, Mazurkiewicz (1982) proposed major research, "Estimation of the reduction of the shear strength in cohesive soils due to remolding during driving, taking into consideration when estimating the ultimate skin friction, the increase of the shear strength of the remolded clay with time, the changes of pore pressure during driving, and the influence of the permeability of the pile material." Considerable research effort in the oil-industry-sponsored ACAPP and ESACC programs (Kraft, 1982; Kirby et al, 1983) focused on effective stress procedures for clays and in continuing academic studies (Randolph, 1983) has advanced our knowledge but has not adequately examined that thin but real zone of reoriented remolded soil and its response to the varied sequence of events imposed upon it. The recent progress in these effective stress programs is encouraging that the present "first approximation" t-z models (Kraft, 1981) or degradation models (Randolph, 1983) can be extended and expanded to permit specific consideration of different construction parameters, cyclic loading, rate of loading, time after installation, time after last maximum loading, and other construction/loading variables as well as the constitutive properties of each soil layer and the compressibility of the pile.

CONCLUSIONS

The design of piles to carry axial static loads appears to remain essentially an art. This is evidenced because the survey found that there is not a universal set of static design rules, based on a fundamental understanding of pile-soil interaction phenomena. The art of the practice is suggested to be heavily influenced by local geology and at least five components of general engineering/construction practice, which are (1) prevalent construction materials and techniques, (2) governmental influence in the form of codes, (3) relative risk accepted by owners, (4) precedents, and (5) economic factors. For some circumstances, the art is very highly developed, producing a high degree of confidence in the expected performance of pile foundations, for example driven displacement piles in the sands of Belgium and The Netherlands.

While considerable research into the static capacity of piles has been conducted in the past decade, no real breakthough has been made that has universally impacted design. Designers have continued to rely essentially on simple empirical formulas heavily weighted by their own experience and philosophy. It is not surprising, therefore, that the evolution of static design procedures has been driven by local geology and other local factors producing a divergence in the detail of procedures utilized. The authors conclude from the survey that most of the respondents are applying the observational method of Terzaghi to their practice of pile foundation engineering.

The present empirical approach appears to be hampered by a shortage of appropriate full-scale data, especially with regard to geotechnical characterization, from which design rules can be refined and which would assist researchers in pursuit of fundamental phenomenological understanding of pile-soil interaction. Some attempts are being made to conduct static tests and to monitor long-term performance with a view toward improving practice and not just insuring an adequate foundation for a specific project. In the past, design of high penetration, large diameter, heavily-loaded pipe piles for offshore structures has relied heavily on gross extrapolation of onshore experience. From research conducted in the past few years and major tests now being planned, significant improvements in our basic knowledge of pile-soil interaction are now expected to be derived from pile research focused on offshore problems. The improvements are likely to result from the need to consider the progressive development at a point on a long compressible pile of maximum load transfer potential followed by degradation of load transfer at that point and "load shedding" down the pile, and to consider variation of loading sequence on both incremental load transfer and ultimate pile capacity.

In the area of field control including production testing, rapid advances have been made in the past decade that have been incorporated into the total design process. The advent of dynamic monitoring techniques coupled with wave equation analyses have made it possible to "test" more piles at a site at a lower cost to evaluate static capacity and pile integrity than was possible by earlier static-load-test-only methods. Such techniques are also useful in the control of construction by providing indications of hammer performance, which allow for better interpretation of blow count records. The promulgation of dynamic capacity evaluation techniques will, however, have a negative impact on the development of an improved-quality data base of capacity information from which better static design rules can be formulated. These techniques will encourage the omission of static load tests in which careful measurement of pile-soil load transfer and deformation are made along with detailed characterization of soil properties. The authors hope that the profession will be able to accommodate this new approach while continuing to seek acquisition, analysis, and public dissimination of static load test and performance data as major objectives because at least the initial estimate of pile design must continue to be made on the basis of static design procedures.

Progress in the design and construction of bored piles has been considerable throughout the world. It is clear that the usage of bored piles has propagated significantly, due largely to improved confidence in the structural integrity of bored piles. This has resulted from development of improved construction methods, integrity evaluation procedures, and increased availability of test records, which itself has created an improved understanding of the effects of differences in construction techniques. The expansion of bored pile usage will probably continue into geographic areas where they are not now used.

Pile design for a specific project almost always falls under the umbrella of one or more codes or "guidelines." Most of the world pile codes are "prescriptive," although the details of the prescriptions vary considerably. Most prescriptive codes allow for design approaches that differ from those prescribed, as long as the substituted procedures are rational, correct, and sufficient to protect public welfare. The survey suggests that some building officials and certifying agencies are reluctant to permit variances, to the extent that codes in some areas tend to impede the development of new and more appropriate procedures for design and installation of pile foundations.

Despite the lack of research breakthroughs and the impediments associated with some codes, progress in the design and construction of piles is proceeding at a steady pace. Greatest progress worldwide is, in the opinion of the authors, being made in the adaptation of general empirical procedures to the local geology and engineering practice of the region where the foundation is to be installed. Progress can continue, perhaps at a more rapid pace, if all practitioners including designers, contractors, and owners will focus on acquisition and analysis of fundamental data. They must to the maximum extent possible relax proprietary constraints in the interest of free exchange of information. The greatest opportunity for research advancement in capacity predictions is believed to lie in detailed study of the constitutive properties of the thin reoriented soil zone adjacent to the pile. The greatest need is continued recognition that "most foundation failures .... have not been caused by erroneous estimates of bearing capacity or settlement, but by defects in the foundation elements themselves" (Peck, 1974), and "in many cases where pile-supported structures have been damaged, the cause can be traced to faulty workmanship during installation" (Burland et al, 1982). The greatest hazards are present in the extrapolation of empirical procedures from one geologic area or from one construction technique to a different geologic setting or to a different construction situation.

The good geotechnical engineer is the one who knows the limits of his experience with problems and soil conditions comparable to those of his current assignment and makes appropriate extrapolations. He knows what he knows and uses it confidently. More importantly, he knows what he does not know, seeks all available knowledge, and then proceeds fully acknowledging his limitations and uncertainties.

Acknowledgement

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* An asterisk by a reference indicates the authors' recommendation of it as a state-of-the-art paper warranting attention. No attempt has been made to compile a comprehensive reference list. Instead interested readers are referred to in the lists in each S-O-A paper.

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APPENDIX A: LIST OF RESPONDENTS

Adams, J. I., and Pellew, T.; Ontario Hydro; Canada.

Akinmusuru, J. O.; University of Ife; Nigeria.

Al-Salloum, N.; Ministry of Communications; Saudi Arabia.

Anonymous, ; Anonymous Oil Company; U.S.A.

Baguelin, F.; Laboratorie Central des Ponts et Chausses; France.

Baker, C. N., Jr.; Soil Testing Services, Inc.; U.S.A.

Bartolomey, A. A.; Perm Polytechnical Institute; U.S.S.R.

Bea, R. G.; PMB Systems Engineering, Inc.; U.S.A.

Bozozuk, M.; National Research Council, Div. of Building Research; Canada.

Braun, J. D.; Jacques Whitford and Associates; Canada.

Bredenberg, H.; Royal Institute of Technology; Sweden.

Cheney, R. S.; Federal Highway Administration; U.S.A.

Cragg, G.; Ontario Hydro; Canada.

Dastidar, A. G.; Foundation Consultant; India.

Davisson, M. T.; Consulting Engineer; U.S.A.

Debedin, F.; Ontario Hydro; Canada.

DeBeer, E.; State University of Ghent; Belgium.

Demars, K. R.; University of Connecticut; U.S.A.

Demsky, E. C., and Moore, B.H.; Corps of Engineers; U.S.A.

Donaldson, G. W.; Council for Scientific and Industrial Research; South Africa.

Douglas, D. J.; Frankipile Australia Pty., Ltd.; Australia.

Duoto, T.L., and Stripling, C.M.; Houston Lighting and Power Co.; U.S.A.

Engeling, P.; Raymond-Kaiser Engineers; U.S.A.

Fellenius, B. H.; University of Ottawa; Canada.

Gammon, J. R. A.; Fugro (Hong Kong), Ltd.; Hong Kong.

Guha, S.; Simplex Concrete Piles (India), Pvt. Ltd.; India.

Heijnen, W. J.; Delft Soil Mechanics Laboratory; Netherlands.

Holloway, D. M.; In-Situ Tech, Inc.; U.S.A.

Howell, C. T.; Houston Lighting and Power Co.; U.S.A.

Insley, A.E.; Thurber Consultants, Ltd.; Canada.

Iyengar, M.; Engineers India, Ltd.; India.

Jamiolkowski, M., Lancellota, R., and Pasqualini, E.; Technical University of Torino; Italy.

Kerisel, J.; L'Ecole Nationale des Ponts et Chaussees; France.

Kisheda, H.; Tokyo Institute of Technology; Japan.

Kulhawy, F. H.; Cornell University; U.S.A.

Laughter, C. N.; Wisconsin D.O.T.; U.S.A.

Li, J. C.; National Central University; Republic of China.

Mello, L.G., Cepollina, M., and Oliveira, F.; Victor F.B. de Mello and Associates; Brazil.

Nordlund, R. L.; Franki Foundation Company; U.S.A.

Olson, R. E.; University of Texas; U.S.A.

Pelletier, J. M.; Shell Oil Company; U.S.A.

Poskitt, T. J.; Queen Mary College, Univ. of London; U.K.

Poulos, H. G.; University of Sydney; Australia.

Quigley, D. W.; Harding Lawson Associates; U.S.A.

Randolph, M. F.; Cambridge University; U.K.

Rausche, F.; Goble and Associates, Inc.; U.S.A.

Rigden, W. J.; British Petroleum, Ltd.; U.K.

Sahrmann, G. J.; McDermott, Inc.; U.S.A.

Schmidt, H. G.; Bilfinger and Berger Bauakien-Gesellschaft; Federal Republic of Germany.

Smith, I. M.; Private Consultant; U.K.

Sommer, H.; University of Kassel; Federal Republic of Germany.

Stocker, M.; Karl Bauer Special Foundation Company; Federal Republic of Germany.

Tagaya, K.; Mitsubishi Heavy Industries, Ltd.; Japan.

Tang, Nian-Ci; Nanjing Institute of Technology; People's Republic of China.

Tavenas, F.; Laval University; Canada.

Tejchman, A.; Technical University of Gdansk; Poland.

Tong, Yi-Xiang; East China Power Design Institute; People's Republic of China.

Touma, F. T.; Rashid Geotechnical and Materials Engineers; Saudi Arabia.

vanWeele, A.F., Beringen, F., and TeKamp, W.G.B.; Foundacon B.V. and Fugro Consultants; Netherlands.

Wallays, M.; Franki Foundation Company; Belgium.

Webb, J. D.; Mississippi State Highway Department; U.S.A.

Yamane, G.; Shannon and Wilson, Inc.; U.S.A.

York, D. L.; Port Authority of New York and New Jersey; U.S.A.

Yu, Tiao-Mei; Tongji University; People's Republic of China.