

# Summary and Review of Part II of the Symposium on Pile Foundations

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•PILE FOUNDATIONS share with cellular cofferdams the distinction of yielding most reluctantly to the advantageous use of theory as a basis for design practice. Papers presented during the second part of the Symposium summarize much of the progress made to date; they offer little comfort to those who view foundation engineering as an applied science. The papers hardly overlap in coverage of subject matter, and it is convenient to consider them separately.

Coyle and Sulaiman review the state of the art concerning the bearing capacity of driven vertical piles subject to static vertical loads. The scope of this paper is indeed comprehensive.

In the discussion of static formulas to predict point-bearing resistance in cohesionless soils, the authors point out that values of the bearing capacity factor  $N_q$  determined theoretically by different investigators vary widely (Fig. 3 in the paper).  $N_q$  is also very sensitive to the angle of shearing resistance  $\phi'$ ; a variation in  $\phi'$  of a few degrees results in  $N_q$  being changed by a factor of two or more, regardless of which theory may be selected for the analysis. Yet, natural variations in state of compaction and subsequent changes due to pile driving preclude an accurate estimate of  $\phi'$  in situ. Coupling this with the uncertainty in  $N_q$  even if  $\phi'$  were known precisely leads the writer to the conclusion that predicting point-bearing from static formulas is largely an academic exercise. Correlations with static cone penetration resistances in specific granular deposits can narrow the margin of error.

Concerning shaft friction in cohesionless soils the authors state, "In general, the (unit) frictional resistance decreases with depth, and is independent of initial overburden pressures." The distribution of unit shaft friction due to butt loads added subsequent to driving can be deduced from measurements of the changes in pile load versus depth. In reality, residual stresses due to driving can be very significant, and little is known of their magnitude and distribution; thus, any generalization regarding the actual distribution of unit shaft friction is tenuous at best. There is some justification, however, for stating that (at ultimate load) the fraction of butt load carried by shaft friction, per unit surface area of embedded pile, increases (more or less) linearly with pile penetration and then approaches a constant value (in a manner similar to that shown for point bearing in Figure 4 of the paper). Present concepts regarding load transfer in piles (point-bearing and distribution of shaft friction) are based almost exclusively on short-term loading tests. What happens in the long run is a matter for speculation.

The authors draw attention to cases where the load-carrying capacity of piles increases significantly with time. Losses in driving resistance (and presumably in bearing capacity) have also been observed for piles driven into dense sands, clay tills, and other hard strata. Negative pore pressures that dissipate with time may be the culprit. On the other hand, relaxation of residual stresses could be a contributing factor. Redriving can increase pile penetrations substantially. The concomitant increase in static load capacity may be important if the design specifies piles of high capacity.

Coyle and Sulaiman also treat the bearing capacity of single piles predicted from dynamic formulas. In this connection, it is useful to summarize the main findings of the Michigan pile study:

1. Depending on the pile capacity and the type of hammer, pile, pile cap, cushion, and soil conditions, the ratio of measured energy transmitted to the top of a pile to the manufacturer's rated hammer energy was 0.25 to 0.65 and averaged around 0.50. Field control of the pile-driving operations was such that little of the energy lost could be attributed to malfunction of the hammer, friction in the leads, and similar extraneous factors.

2. The inconsistencies of dynamic formulas (those based on total energy balance) in predicting ultimate pile capacity as measured by load tests are sufficiently large to make their use for this purpose undesirable, even if the measured transmitted energy is used in the calculation.

As the authors point out, much attention is now being focused on the wave equation as a possible means of predicting pile capacity. The input data for the wave equation include the properties of the hammer, pile cap, cushion, and pile; the static point-bearing and shaft friction resistance offered by the soil, and the deformations at which these resistances are mobilized; and the additional point-bearing and frictional resistance of the soil due to dynamic penetration. The output is the penetration per blow and the stresses in the pile. The inverse procedure—prediction of the static resistances from one output, the penetration per blow—is a much more tenuous matter that, in the writer's opinion, is presently beyond the range of routine application. However, the wave equation has proved its worth for the purpose of matching the pile and driving equipment to achieve desired penetrations efficiently and for guarding against over-stressing in the driving process.

Concerning the bearing capacity of pile groups, the efficiency factor  $E$  for piles in granular soils is of small consequence ( $E$  may exceed two for closely spaced, relatively long piles driven into loose cohesionless soils) because the design of such pile groups is governed by tolerable movements. The prediction of group displacement is often based on the expected movement of a single pile at working load and an estimate of the factor by which this movement is multiplied to obtain the settlement of the group (Fig. 13 in the paper). The movement of a single pile at working load can be estimated from load tests; however, it would be useful if this movement could be predicted by less costly means. Skempton et al. (51 in the paper) suggested that the displacement of a pile in granular soil is (approximately) a unique function of the ratio of applied load to the ultimate capacity of the pile, although this is admittedly an oversimplification. The authors mention the development of an in situ testing device at Texas A&M University that measures skin friction and point-bearing as a function of pile movement. Such a device could prove to be very useful, and it is hoped that the results of this research will soon be published.

It is not widely recognized that the curve proposed by Skempton et al. (Fig. 13 in the paper) for estimating the settlement of pile groups in granular soils is only tentative and was based on 2 case records presented at the Second International Conference on Soil Mechanics and Foundation Engineering: One was by Feagin and involved Mississippi River silty sands, and the other was by Vargas of Brazil where the granular soils contained significant amounts of clay. The writer is aware of 4 additional case studies where the ratio of the measured settlement of the pile group to that predicted from Skempton's curve ranges from 0.2 to 2.0—hardly a situation that calls for complacency. It appears that the settlement ratio (group to single pile) depends greatly on whether the loads are transmitted in point-bearing or by shaft friction, on the length of the piles, and on the degree to which driving the pile group further densifies or loosens the soil. This is an area where additional research is badly needed.

Concerning the bearing capacity of friction pile groups in soft to medium clays, there is little to add to the recommendations summarized in the paper except to point out that only model tests are available to confirm the proposed procedures and that heaving is likely to be a problem if the pile spacing is 3 diameters or less. To calculate the settlement of such pile groups due to consolidation, Terzaghi's suggestion

of transferring the load to an elevation two-thirds the pile length from the top of the piles is clearly a first approximation. More rational methods for predicting these settlements are still in the research stage.

Fuller and Hoy review qualitatively the procedures commonly used for performing pile load tests and provide a useful checklist of factors to be considered in planning a testing program. A feature of the paper is the description of the quick test method used by the Texas Highway Department, which is a simpler version of the constant rate of penetration test developed at the Building Research Station, England. Correlations are given for the maximum proven design load as interpreted from the quick load test and from the standard AASHTO test. For effective pile penetrations of 20 to 60 ft in sand and clay and silty, sandy clay the correlations are extremely good. (It is interesting to note that the K-factor that multiplies the safe load given by the Engineering-News formula to yield the maximum proven design load ranges from 0.6 to 4.9—another example of the unreliability of dynamic formulas for predicting pile capacity.) If the correlations prove to be valid for a wide range of soil types and pile penetrations, the quick test method can replace conventional load test procedures at substantial savings in time and costs.

Although the defects of dynamic formulas are often cited, the limitations of conventional load test results are seldom mentioned. Such tests usually record the butt load and the corresponding butt deflection on the loading and unloading cycle. In the writer's opinion the specifications should provide a load capacity sufficient to establish the ultimate resistance of the pile. The ultimate resistance is clearly defined if the pile "plunges" into the ground (the deflection per unit time, at constant load, remains constant or begins to increase with time). If this does not happen an interpretation such as that given in Figure 6 can be made. In the latter case the test should not be stopped unless the pile deflection exceeds 20 percent of the tip diameter (or the structural strength of the pile is being approached) to ensure that full point resistance has been mobilized. Such criteria as "the load at 0.25 in. net deflection" are merely empirical procedures intended to estimate the pile load at which the settlement of the pile group is likely to be tolerable. They may bear no relation to the ultimate pile capacity.

A conventional load test can only establish the capacity of a pile at the time of testing under conditions where the ultimate point-bearing and ultimate shaft friction are mobilized simultaneously. The following factors obscure the relation between such test results and the subsequent performance of a pile foundation:

1. The net settlement curve obtained from rebound data gives only an indication of tip penetrations because the stresses in the pile and in the surrounding soil during unloading can be very different from those during loading. The discrepancy may be reduced, but probably not eliminated, by cycling the load several times.
2. The settlement of the pile at a given butt load can change drastically with time if a transfer of load from shaft friction to point-bearing takes place. Such load transfers can occur because of (a) creep of the surrounding soil, (b) consolidation of clay layers caused by placement of fill, groundwater lowering, or remolding or excess pore pressures or both caused by pile driving, and (c) minor vibrations associated with normal occupancy of the structure.
3. There is no consistent relationship between the settlement of a single pile and the settlement of the pile group at the same load per pile. Therefore, selecting a design load on the basis of the load at a given gross or net deflection, or at a given fraction of the ultimate pile capacity, is equivalent to accepting an unknown factor of safety with respect to satisfactory performance of the foundation.

If properly conducted, load tests can provide important information regarding the performance of a single pile for load transfer and soil conditions extant at the time of the test. Although this information is superior to that which can be deduced from static or dynamic formulas, it is not sufficient for rational design of pile foundations. In situations where experience with given types of pile foundations is lacking or very limited, the designer is compelled either to be very conservative or to take an "uncalculated risk" with respect to satisfactory performance.

Davisson presents an exceptionally lucid summary of the nondimensional procedure for estimating the shears, moments, and deflections in laterally loaded vertical piles



based on the subgrade modulus  $k$  model. As this model is clearly an approximation to the actual soil response, the selection of appropriate magnitudes and distributions of  $k$ -values is crucial to the potential success of the analysis. In this connection, Davisson's suggestions for selecting  $k$ -values (Table 1 in the paper) are welcome because they represent a synthesis of considerable experience.

Recent studies indicate that  $k$  decreases significantly with increasing deflection especially near the ground surface. As the deflections are maximum near the surface, and this is the zone where  $k$  has its greatest influence on the stresses in the pile, the practical importance of the nonlinearity in  $k$  deserves further study. The author suggests that a load test could be conducted, which is strongly endorsed by the writer. In fact, there is at least as much to be gained from a lateral load test as from the corresponding vertical load test; however, the writer would prefer to conduct the test on a prototype pile. (It is often assumed that the lateral resistance of the ground is minimum when the pile supports no vertical loads. Is there any field evidence to justify this assumption?) Not enough is known about the effect of pile width and nonlinearity of  $k$  with deflection; elimination of these uncertainties should more than compensate for any additional cost of testing the prototype pile.

The author correctly emphasizes the importance of the effects on  $k$  of repeated loading and pile spacing if the performance of a pile foundation is to be assessed, and states that their combined effect can result in  $k_{\text{effective}}$  being as low as 10 percent of that applicable to initial loading of a single pile. This statement is based on small-scale model studies of the group effect while the repeated load effect stems largely from the behavior of a single pile. The suggestion that  $k_{\text{eff}} = 0.1k$  may be unduly conservative, as indicated by the following example. Lateral load tests were performed on 2 groups of 150 piles. The lower portions of the piles were embedded in soft rock and the upper portions extended through submerged, relatively loose, uniform sand. Pile spacing in the direction of the load was  $2\frac{1}{2}$  to 3 diameters. The design load was applied and cycled 5 times. Because of cycling,  $k_{\text{eff}}$  reduced to 55 and 80 percent of  $k$  respectively (30 percent was indicated in the paper). Actually, maintaining the lateral load for 24 hours decreased  $k_{\text{eff}}$  almost as much as load cycling. Unfortunately, a load test on a single pile was not conducted, hence no definitive statement can be made regarding group action. Judging from the observed deflection of the pile cap and the loose nature of the sand, however, it is unlikely that  $k_{\text{eff}}$  was  $0.25k$  as indicated in the paper. More field measurements on laterally loaded pile groups are badly needed.

York presents an interesting survey of the structural behavior of driven piles, including factors such as pile stresses during and after driving, the stresses in and load capacity of bent and damaged piles, the potential magnitude of negative friction (drag-down), and the likelihood that many pile foundations may never be subjected to their design loads. Attention is also directed to the wide variation of accepted values for allowable stresses in concrete, steel pipe and steel H-piles. To the writer's knowledge this paper is unique in the literature.

Although the examples cited are most instructive and a valuable guide to good judgment, it is difficult to draw general conclusions from the survey. For example, it is true that steel pipes filled with concrete, when tested in the laboratory, have ultimate strengths that suggest that working stresses allowed by most codes are too conservative. On the other hand, definitive methods for assessing the corrosion that may occur during the intended life of the piling are difficult to come by. In this connection, the study by the National Bureau of Standards on the corrosion of H-piling is encouraging but not entirely comforting. Moreover, the corrosive activity of the ground can change with time often because of factors that are difficult to anticipate at the time of construction. In another vein, bent and damaged piles have consistently shown surprisingly high load capacities under short-term loading. If it is argued that their capacity may not deteriorate much with time on the grounds that many existing foundations supported by such bent or damaged piles or both have performed satisfactorily, consideration should be given to the high probability that in most instances the pile stresses were correspondingly low. Would they perform equally well in the long run at higher working stresses? Foundation performance may be likened to a chain with many links; changing any one link (such as using higher working stresses) may lead to unexpected failures in other

links. The "good field data" that the author recommends be collected must be comprehensive in scope and long term in nature. Such data are difficult to generate, but they are a prerequisite to more rational design methods. Unless (a) the loads supported by the piles are clearly defined, (b) the piles can be inspected and only straight, undamaged piles are accepted, and (c) pile deterioration can be evaluated accurately, conservatism in structural design of piles is fully justified.