EFFECTS OF TIME ON CAPACITY OF PIPE PILES IN DENSE MARINE SAND[a]

Discussion by John A. Focht Jr.,[5] Fellow, ASCE

The authors of this paper and the several preceeding papers published since about 1990 by one or more of that group are to be commended for their improved procedures for predicting the capacity of pipe piles in sand, particularly in reference to their increase in capacity with time. The data presented in Table 6 of this paper and in Table 1 of the paper by Jardine, et al. (1998) for API RP2A procedures encouraged the discusser to look back at his work in the mid-1950s and early 1960s. During that period, he, Bramlette McClelland, and William J. Emrich at McClelland Engineers, Inc. evolved and tried to improve standardized procedures for predicting the "ultimate" frictional capacity of pipe piles driven into sands. Those procedures that appeared in the McClelland Engineers "Appendix C" from 1953 through at least 1974, modified some with time, were later adopted for API RP2A, which underwent further slight modifications. A brief historical summary of capacity criteria for piles in both sand and clay was presented by Pelletier et al. (1993).

In reference to "ultimate" capacity, I am reasonably sure that we thought then that this capacity would be reached in a relatively short period of time—two to four weeks. To my knowledge there were no data from tests with longer setup times. From the available data (almost all from tests on relatively short piles conducted as verification tests, not research) we knew that actual capacities ranged widely coordinating poorly with our simple procedures. I do not remember making statistical analyses but would expect that the COV values would have been generally similar to the 0.86 in Table 1 of the Jardine et al. (1998) for shaft friction. We also knew at the time that the simple procedure would usually grossly underestimate unit skin friction at shallow penetrations, and thereby the total capacity of short piles. But we had the feeling that it might overestimate unit skin friction at considerable depths.

Out intent was to produce conservative but reasonable predictions for piles with penetrations of 100 to 200 ft anticipating considerable scatter of results. I am pleased that the mean Qc/Qm has been shown to be about 0.8, for that tends to validate what we thought we were doing. Our intuitive judgmental selection of criteria based on meager and widely scattered data was pretty good. Now, predictions can be made with less average conservatism and much greater reliability.

APPENDIX. REFERENCES

Jardine, R. J., Overy, R. F., and Chow, F. C. (1998). "Axial capacity of offshore piles in dense North Sea sands." *J. Geotech. and Geoenvir. Engrg.*, ASCE, 124(2), 171–178.

McClelland Engineers Inc. (1967). "Appendix C, criteria for predetermining pile capacity." *Rep. Section Included for Clients of Offshore Geotech. Investigations.*

Pelletier, J. H., Murff, J. D., and Young, A. C. (1993). "Historical development and assessment of current API design methods for axially loaded piles." *Proc., 25th Offshore Technol. Conf., OTC 7157*, 253–282.

[a]June 1998, Vol. 124, No. 6, by F. C. Chow, R. J. Jardine, F. Brucy, and J. F. Nauroy (Paper 15534).

[5]Focht Consultants Inc., 12226 Perthshire, Houston, TX 77024.

Closure by F. C. Chow,[6] R. J. Jardine,[7] F. Brucy,[8] and J. F. Nauroy[9]

The writers thank the discusser for his contribution and for his reminder of the uncertainties faced by the early offshore pile designers in the 1950s and 1960s. The interpretation of noninstrumented pile tests is difficult, relying on several simplifying assumptions that can mask many important aspects of pile behavior. The recent advances have been achieved through the development and use of reliable on-pile instrumentation, capable of measuring the effective stresses at the pile-soil interface throughout pile installation and testing.

Reliability studies have shown that each reduction of 0.1 in the COV associated with a design method can lead to an order of magnitude improvement in foundation reliability. This highlights the importance of employing design methods with low COV values, particularly when low design factors of safety are adopted, as in the case of offshore pile foundations where factors of 1.5 are often used for the extreme design case.

The writers are pleased to see that new cases demonstrating the effects of time on the capacity of piles are continuing to appear and that pile designers are now able to take account of these effects in the planning and back-analysis of pile tests.

[6]Engr., Geotechnical Consulting Group, 1A Queensberry Place, London SW7 2DL (Formerly Imperial College).

[7]Reader, Dept. of Civ. Engrg., Imperial College Rd., London SW7 2BU.

[8]Engr., Institut Français du Pétrole, BP 311, 92506 Rueil Malmaison, France.

[9]Engr., Institut Français du Pétrole, BP 311, 92506 Rueil Malmaison, France.

149

UNDRAINED LIMIT ANALYSES FOR COMBINED LOADING OF STRIP FOOTINGS ON CLAY[a]

Discussion by Jean Salençon,[4] Member, ASCE, and Alain Pecker[5]

In their paper the authors occasionally and briefly refer to the contributions by Salençon and Pecker (1995a,b) on the evaluation of the bearing capacity of a strip footing under combined loading. A comparison between the results obtained by the authors and those obtained by the discussers (apparently the best results available previously to the paper) is worth making, and may help point out the breakthrough of the authors' very valuable contribution from the theoretical point of view of the application of limit analysis to the study of the bearing capacity of strip footings.

With the paper's notations the problem studied by the discussers corresponds to Mode II combined loading. Lower-

[a]March 1998, Vol. 124, No. 3, by Boonchai Ukritchon, Andrew J. Whittle, and Scott W. Sloan (Paper 15586).

[4]Prof., Dept. of Mech., École Polytechnique, F91128 Palaiseau cedex, France.

[5]Chair. and Managing Dir., Géodynamique et Structure, 157 rue des Blains, F92220 Bagneux, France.

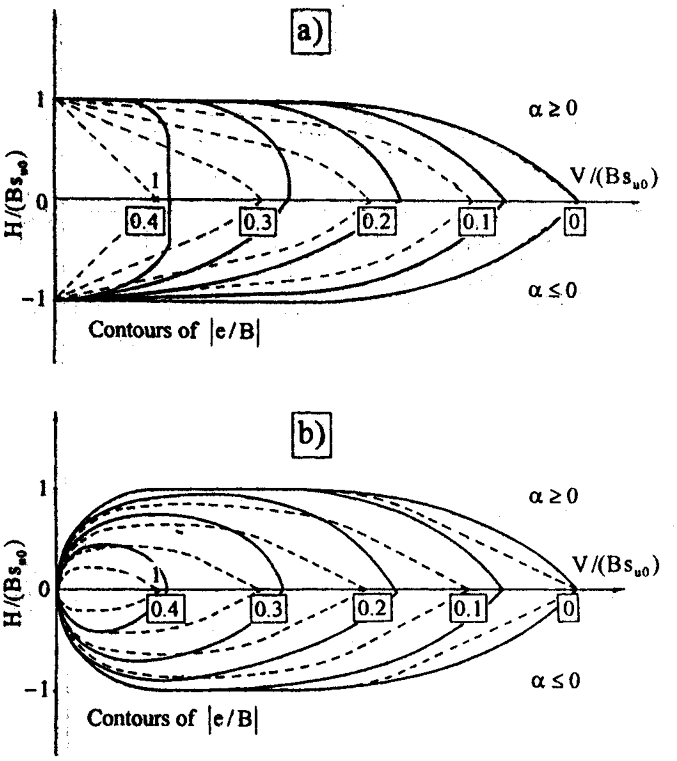

FIG. 21. Projections of Failure Surface for Combined Loading of Footing: (a) on Homogeneous Clay; (b) on Homogeneous Clay with Zero Tensile Strength

bound and upper-bound estimates have been produced through analytical procedures associated with numerical minimization on a few parameters as regards the kinematic approach; exact solutions have been obtained in some cases. With the notations of the paper, Fig. 21 shows the results obtained by the discussers to be compared with those presented in Fig. 12 (where the charts seem to be mislabeled).

The upper-bound estimates obtained in both papers are very close to each other when they don't coincide. It is noticeable that the dissymmetry of the failure surface predicted by the discussers' upper-bound results is now confirmed.

For zero eccentricity ($e/B = 0$) the lower bound in Fig. 12 is not as good as the one shown in Fig. 21. As a matter of fact it was proven that the lower bound in Fig. 21 matches the upper bound and yields the exact failure surface for $90° > |\alpha| > 7°$ and for $\alpha = 0°$; for $0° < |\alpha| < 7°$ the gap between the upper- and lower-bound estimates is very narrow. For increasing values of the eccentricity it had been observed by the discussers that the lower bound in Fig. 21 was getting poorer and the conjecture had been put forward that the exact failure surface was most likely in the form of the upper-bound estimate. It is a major contribution of the paper to produce reliable statically admissible stress fields for the lower-bound approach, thanks to which, in the considered case, the validity of the conjecture is now definitely proven; all the comments issued by the discussers about the assessment of the classical correction factors are corroborated and reformulated in the paper.

Salençon and Pecker (1995b) also drew attention to the importance of the tensile strength assumed for the clay layer and produced the corresponding results in the case of a zero tensile strength cohesive soil [Fig. 21(b)]. It is likely that the application of the very efficient numerical approaches of the authors would help fill the gap between the obtained lower- and upper-bound estimates of the failure surfaces.

It must be emphasized that the numerical procedures presented in the paper are of the highest interest in the production of good and reliable upper and lower bounds in limit analysis. One may even venture to say that the lower-bound aspect is the most important as the construction of "efficient" statically admissible stress fields is always a challenge.

Closure by Boonchai Ukritchon,[6] Andrew J. Whittle,[7] and Scott W. Sloan[8]

The writers would like to thank the discussers for their most generous recognition of our paper. The writers agree with their observation that the key to the success of the proposed numerical limit analyses lies in the calculation of high-quality lower-bound solutions, which are not easily found by other methods (as shown in Fig. 21). Although the numerical solutions are able to give very tight bounds on the true collapse loads (all of the cited examples are within ±5%), they are clearly no substitute for exact solutions as derived by the discussers for cases with $|e/B| = 0$.

Erratum. The following correction should be made to the original paper: in Fig. 12(b), the sequence of contour labels $|e/B| = 0.0–0.35$ should be reversed such that Figs. 12(a) and 12(b) are compatible. A corrected Fig. 12 is shown here.

[6]Grad. Res. Fellow, Dept. of Civ. and Envir. Engrg., Massachusetts Inst. of Technol., Cambridge, MA 02139.

[7]Assoc. Prof., Dept. of Civ. and Envir. Engrg., Massachusetts Inst. of Technol., Cambridge, MA.

[8]Assoc. Prof., Dept. of Civ. Engrg. and Surv., Univ. of Newcastle, Newcastle, NSW 2308, Australia.

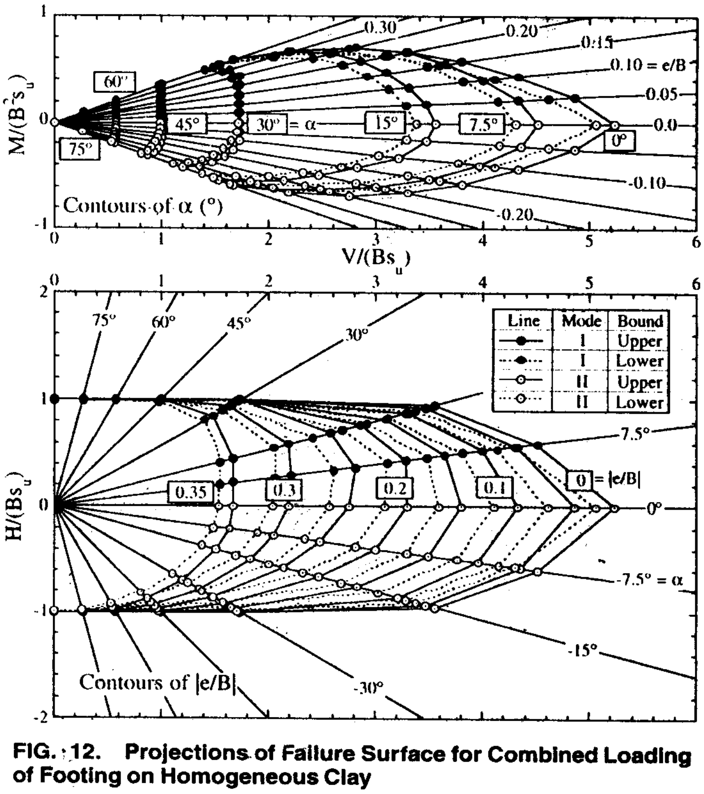

FIG. 12. Projections of Failure Surface for Combined Loading of Footing on Homogeneous Clay