Transactions on Engineering Sciences vol 32, © 2001 WIT Press, www.witpress.com, ISSN 1743-3533

Analysis of the ultimate bearing capacity of a single pile in granular soils

F. Kirzhner and G. Rosenhouse
Faculty of Civil Engineering - Technion, Haifa, Israel

Abstract

The bearing capacity of a single pile in granular soil was analyzed piles under vertical loads in the study of the mechanism of shaft friction. The effect of the deformation modulus on the bearing capacity of the single pile in granular materials, and especially in sand, was examined. By using the FLAC program for the finite element analysis it was possible to predict quite accurately the load capacity and load deformation behavior of the pile, for both rigid and deformable piles. The specific results have shown that bearing capacity of the pile changes from 860 to 1025 tones while the field-loading test reached the point 900 tones. The influence of the load on development of stress and displacement fields around the pile was examined as well. A review of current theories for piles in granular soils, practical methods for calculating the bearing capacity and the numerical analysis presented in the paper enable a quick estimation of the bearing capacity of a single pile in granular soils.

1 Background. Current theories for piles in granular soils

Various theoretical solutions have been proposed for the dimensional problem of the bearing capacity of a single pile such as that of Terzaghi in 1943 [28]. Berezantzev et al. [4] postulated in 1961 a failure mechanism for a driven pile in which the shear zones do not progress above base level. The theory of limit equilibrium in granular media that was proposed in 1952 by Berezantzev [3] solved the stress field. Meyerhof [14] proposed in 1953 a formula for the bearing capacity of a pile in soil possessing both cohesion and friction for the

base resistance per unit area. Vesic' [32, 33] derived in 1967 a solution based on the pressure required to expand a cylindrical or spherical cavity, at the level of the base of the pile, and carried out a large scale experimental study. Vesic' concepts were extended in 1984 by Kulhawy [12], who related the combination of rigidity, shape and depth factors to the bearing capacity factors as rigidity index. Norlund [20] has developed in 1963 an empirical method that accounts for the volume of soil displaced by the pile. The bearing capacity of a pile in cohesionless soil is a function of variables such as the toper of pile, roughness, shape of pile surface, and the volume of soil displaced by the pile. Raymond Pile Co. uses Nordlund's equation for example, for calculating confidently the skin friction on tapered piles, but there is little evidence of its reliability when applied to straight-sided piles.
The NAFRAC [19] expresses an ultimate load capacity in compression as:

$$Q_u = P_T N_q A_T + \sum_{H=H_0}^{H=H_0+D} (K_{HC} P_o S \tan\delta), \tag{1}$$

where P_T is the effective vertical stress at pile tip; N_q - the bearing capacity factor in granular soil; A_T - the area of pile tip; K_{HC} - the ratio of horizontal to vertical effective stress on the side; P_0 - the effective stress along the length of embedment, D; δ - the friction angle between pine and soil; S - the surface area of a pile, H_0 - the length of soft soil.

The bearing capacity of single pile may be estimated by following expression presented by Tomlinson [31] in, 1994:

$$Q_P = N_q \sigma^{©}_{vo} A_b + \frac{1}{2} A_S K_S \sigma^{©}_{vo} \tan\delta, \tag{2}$$

where σ'_{vo} is the effective overburden pressure at the pile base level; N_q - the bearing capacity factor; A_b - the area of base of pile; K_S - a coefficient of horizontal soil stress which depends on the relative density and state of consolidation of the soil, the volume displacement of the pile, the material of the pile, and its shape; δ - the angle of friction between pile and soil; A_S - the area of shaft in contact with soil. The factors N_q and K_S are empirical and based on correlations with static loading tests.

2 Practical methods for calculating ultimate bearing capacity

The capacity of a single pile in granular soils can be estimated from results of SPT (Standard Penetration Test) as suggested in 1976 by Meyerhof [16]. A minimum factor of safety of at least 4 is recommended to theoretical computations. Also following [16], the ultimate static resistance of a single pile may be described by the sum of the toe resistance and the shaft resistance. The bearing capacity coefficient of shaft resistance is given in Canadian Foundation Eng. Manual (1985) [7]. The axial capacity of single pile based on static Cone Penetration Test (CPT) in granular soil is given in [7]. Thorburn and Mac Vilar [30] presented in 1970 an empirical formula for the frictional resistance of pile shaft in sand. Meyerhof [15] has shown in 1956 that the skin friction on the pile shaft can also be obtained from cone resistance using the simple empirical relationships established by Te Kamp [27] in 1977. Te Kamp derived the skin friction from results by Dutch cone penetration test (although this method is recognized as the API method and is often specified for use in designing offshore piles (American Petroleum Institute, 1977)). Vesic' [34] suggested in 1975 a simple modification of API-method. Beringen et al. [5] (1979) illustrated computations by these methods, and the results of these were compared with the measured values of loading tests. Their consequence is that prediction of pile capacities obtained by the cone penetration test (CPT) is best, being a reflection of existing in-situ stress conditions.

Ellisson [8] in 1968 and Poulos [22] in 1975 proposed a theory for the bearing capacity of a single pile in sand based on the sum capacities of the pile tip and the maximum skin resistance. The practical use of traditional bearing capacity theories suffers from two main limitations. First, large-scale laboratory experiments -Vesic' [35] and field observations -Meyerhof, [17], indicating that many theories can only be used for short piles, or the so-called critical depth for the bearing stratum. Secondly, further complication in field applications design due to the variability of cohesionless soils and technical sampling problems, causing the need an empirically work, based on in-situ test, using correlation of measured penetration resistance with friction angle.

The use of in-situ test results to evaluate the shaft and the critical base resistance corresponding to the ultimate limit state was discussed by Fioravante et al. [9] in 1995, examining the approaches available for evaluation of the ultimate load capacity of large diameter bored piles in granular soil. E.g., Burland [6] in 1973, Reese et al. in 1988, and O'Neill et al. in 1994 used an empirical earth pressure approach, called also β method, in numerical analysis for estimation of bearing capacity of single pile in sand.

3 Numerical Analysis

Many of the aforementioned deficiencies of the elastic-solid approach can be overcome, at least in principle by the use of finite-element method (FEM). For example Ellisson [8] formulated and built a general program that analyzes load transfer of an arbitrary pile in a soil mass with a bilinear and stress dependent stress-strain response. A similar analysis was introduced by Holloway et al. [11] in 1975, including nonlinear (hyperbolic) stress-dependent stress-strain response, and using different characteristics in compression and tension.

Transactions on Engineering Sciences vol 32, © 2001 WIT Press, www.witpress.com, ISSN 1743-3533

FEM is widely used for calculating and designing all kinds of structures, including pile foundations. It enables numerous factors that influence the behavior of piles to be taken into account, even if there are still many other factors that cannot yet be included. In general we do not know which rheological model or which values of soil characteristics should be taken for the specific soil in study: The FEM can advance engineering solutions considering the rheological behavior of soils. Load transfer for single piles at working loads can be obtained by FEM, using ground resistance parameters, which are in everyday use in soil mechanics, as Smith [25] reviewed in 1982. FEM takes into account linear soils and builds a special contact element between the pile and the soil. Each contact element includes one node of the soil and one node of the pile and is given an area of influence. Initially, the two nodes are linked together by two very stiff springs, one acting normal to the contact surface, and the second one is tangential to this surface (Baguelin et al. [2], 1982). One estimation method for the load-displacement nature of a pile uses a discrete element technique. The soil may be modeled as a series of non-linear ground springs, characterized by 'T-Z curves' relating pile displacement to mobilized shear stress. Smith [25] in 1980 and McAnoy et al. [13] in 1982 used this method. The pile is divided into elastic elements acting on the relevant ground spring. The non-linear elastic finite element calculation is not advised for computing collapse. Instead, initial stress or viscoplastic strain algorithms as shown by Zienkiewicz [36] can be applied.

When applying the FEM to soil mechanics problems, Puech et al. [23] insist on four fundamental aspects: (1) the validity of the rheological law used for the soil; (2) the treatment of the soil-pore water coupling (drainage conditions); (3) the description of interface conditions (soil-structure coupling); (4) taking realistic initial conditions into account. The relative importance of each of these points depends on the problem at hand.

Two rheological laws may suit soil (monotone calculations): 1. An elastic law defined by Young's modulus E and a Poisson's coefficient - ν, 2. An ideal elastoplastic law with an associated Mohr-Coulomb yield surface defined by the elastic properties E, ν and the plastic properties c and ϕ.

4 Results

4.1 Experimental evidence

The results of the loading test which where performed in 1982 in Ashkelon South - Power Station in Israel [26] were taken for estimation of the ultimate bearing capacity of single pile in sand (pile length - 20 m; pile diameter - 0.9 m. The data included the mechanical properties of the sand.

For the analysis, a two-dimensional explicit finite-difference model which simulates the behavior of structures built of soil, rock or other materials that may undergo a plastic flow until reaching their yield limit was applied (FLAC, [10], 1991). The program FLAC can currently analyze two-dimensional plane or axisymmetric shapes, and since real interaction between pile and soil is three-dimensional, some approximations are required.

4.1.1 Basic assumptions for the finite difference simulation

The following presupposings have been implemented in the FLAC simulation:

- The medium is continuous, non-homogeneous and isotropic.
- An axisymmetric configuration has been implemented, with the axis of symmetry aligned with the axis of pile.
- A method has been developed for analysis of axially loaded single pile embedded in soil layer whose modulus increases linearly with depth.
- The modulus of deformation for the analysis was obtained by pressuremeter test [26] in a single borehole. A magnification of modulus of deformation from 7.5 to 12.5 times is obtained [26]. It is based on the analogy derived by Mahmoud et al. [18] in 1990, between the modulus of deformation obtained by plate load test and the modulus of deformation, obtained by pressuremeter test.
- A constitutive elasto-plastic relation was chosen to represent the mechanical behavior of the intact soil, using the Mohr-Coulomb failure criterion.

Two main schemes for calculation of the bearing capacity of pile were developed. The 1st for rigid piles in sand, and the 2nd for deformable piles.

The maximum displacements were obtained by the integration:

$$u = \int_0^T A * e^{-\alpha x} dx \quad ; \quad \text{A–parameter.} \tag{3}$$

α is the convergence parameter. Integrating (3) with respect to time we obtain:

$$u = A\left[\frac{Te^{-\alpha T}}{\alpha} + \frac{1}{\alpha}\int_0^T e^{-\alpha x} dx\right] = \frac{A}{\alpha}\left[Te^{-\alpha T} + \frac{1}{\alpha}(1 - e^{-\alpha T})\right]. \tag{4}$$

At $T \to \infty$, eqn (4) takes the form:

$$u = \frac{A}{\alpha^2}. \tag{5}$$

Fixing the maximum value of α we can obtain the value of the coefficient A using the given maximum displacement u_{max}.

A loading was applied at the cap of pile, using the 2nd scheme. It is well known that under slow cyclic loading, engineering materials degrade and become softer and weaker. Examples of displacement fields at collapse of deep foundations in sand are shown in Fig.1 together with load-displacement graphs. Fig. 1 shows that the bearing capacity derived by numerical analysis changes from 860 to 1025 tons while the field loading test has reached the point 900 tons. Using eqns (1, 2) for bearing capacity of pile in the same soil show that the values of bearing capacity are 760 tons and 925 tons, respectively.

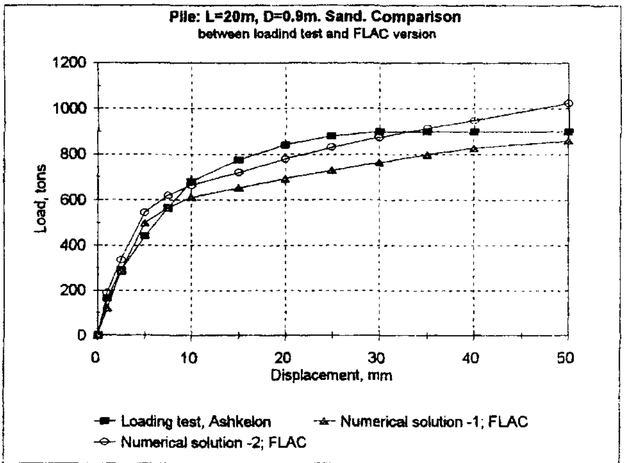

Figure 1: Comparison between loading test (Ashkelon) and numerical solution.1- magnification of the modulus deformation -7.5 times; 2 - magnification of the modulus deformation -12.5 times.

4.2 The effect of the shape of the pile

The bearing capacity of circular piles, of four shapes and the same volume (62.8 m^3) shown in figure 2 is compared.

The displacements of the end of the pile are given in figure 3 and in table 1.

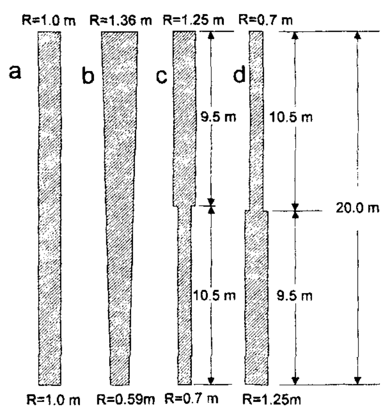

Figure 2: Various shapes of circular pipes. a. cylinder. b. conical. c. combined cylinder. d. combined inverse cylinder.

Transactions on Engineering Sciences vol 32, © 2001 WIT Press, www.witpress.com, ISSN 1743-3533

Table 1: Load- deflection relationships in piles

LOAD (T)	DISPLACEMENT OF EACH PILE TYPE (M)			
	Cylinder	Cone	Cyl Co.	Cyl Co.Ob.
0	0.00000	0.0000	0.000000	0.00000
100	0.00496	0.0103	0.008343	0.003
200	0.00978	0.0386	0.002967	0.00546
300	0.02820	0.0827	0.063	0.00802
400	0.04151	0.137	0.104	0.0107
500	0.06382	0.205	0.1521	0.01396
600	0.08954	0.2771	0.2054	0.02267
700	0.1185	0.3546	0.263	0.03177
800	0.1504	0.4369	0.3237	0.0422
900	0.1847	0.5228	0.3872	0.05367
1000	0.2211	0.6116	0.4535	0.06616
1100	0.2595	0.7039	0.522	0.07944
1200	0.2994	0.8002	0.5919	0.09361

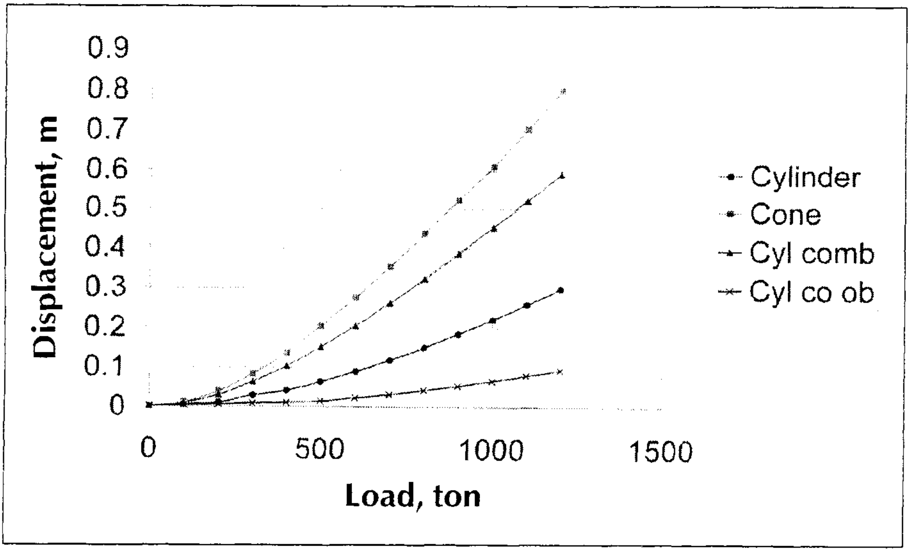

Figure 3: Load- deflection relationships in piles

It is observed from the table that the larger the base area is the lower is the deflection of the end of the pile and the higher is its bearing capacity.

5 Conclusion

The finite-difference model can be used to accurately predict the load capacity and load-deformation behavior of pile embedded in sand. The calculations by FLAC for a specific case show that the bearing capacity derived from numerical analysis changes from 860 to 1025 tons while in field loading test has reached a point 900 tons. For piles under vertical loads, the mechanism of shaft friction becomes clear. Mechanical behavior of sand is highly nonlinear and it must be admitted that a new approach should develop. The effect of the shape of a cylindrical pile was examined, and it was found that for the same volume of pile that the larger the base area is the lower is the deflection of the end of the pile and the higher is its bearing capacity. We have found that the cylindrical pile was better than the conical one, because it resists better the penetration. The inverse combined cylinder is better than the regular, since it has a wider base. It should be noted that the examined shapes are only theoretical, and some shapes can not be applied for practical reasons. However, in special cases

Further numerical developments may include constitutive relationships for soil, especially during cyclic loading and penetrating of piles.

6. References

[1] American Petroleum Institute, Recommended practice for planning, designing and constructing fixed offshore platforms. API RP 2A. 1977.

[2] Baguelin, F., Frank, R. & Jezequel, J.F. Parameters for friction piles in marine soils. *Proc. 2nd Int. Conf. on Num. Meth. in Offshore Piling*, College of Engng, Univ. of Texas, Austin and the Inst. of Civil Engineers, London, pp. 197-214, 1982.

[3] Berezantzev, V.G. Axial symmetrical problem of limit equilibrium theory of earthy medium, Gostekhisdat, 1952.

[4] Berezantzev, V.G., Khrisofov, & Golubkov, V.N. Load bearing capacity and deformation of piled foundations, *Proc. 5th Int. Conf. Soil Mech.*, Paris, 2, pp. 11-15, 1961.

[5] Beringen, F.L., Windle, D. & Van Hooydonk W.R. Results of loading tests on driven piles in sand. *Proc. ICE, Recent Developments in the Design and Construction of Piles*, London, pp. 213-225, 1979.

[6] Burland, J.B. Shaft friction of piles in clay - a simple fundamental approach. *Ground Engineering*, **6** (3), pp. 30,32,37,38,41,42, 1973.

[7] Canadian Geotechnical Society, *Canadian Foundation Engineering Manual*, 2nd Edition, Vancouver, 1985.

[8] Ellisson, R.D. *An Analytical Study of the Mechanics of Single pile Foundation.* Master's Thesis. Carnegie-Mellon University, Pittsburgh, Pennsylvania, 1968.

[9] Fioravante, V., Ghionna, V.N., Jamiolkowski, M. & Pedroni, S. Load carrying capacity of large diameter bored piles in sand and gravel, *Proc. 9th Asian Regional Conf. on Soil Mech. and Foundation Engng*, 1995.

[10] FLAC, Fast Lagrangian Analysis of Continua. Version 3.0, ITASCA Consulting Group, INC, Minneapolis, Minnesota, 1991.

[11] Holloway, D.M., Glough, G.W. & Vesic' A.S. *Mechanics of Pile-Soil Interaction in Cohesionless Soil.* Duke Soil Mech. Ser., 39, 280 pp, 1975.

[12] Kulhawy, F.H. Limiting tip and side resistance, fact or fallacy. Proc. *Symp. on Analysis and Design of Pile Foundations.* American Society of Civil Engineers, San Francisko, pp. 80-98, 1984.

[13] McAnoy, R.P.L., Cashman A.C. & Purvis, D. Cyclic tensile testing of a pile in glacial till. *Proc. 2nd Int. Conf. on Numerical Method in Offshore Piling, College of Engineering*, The Univ. of Texas at Austin and the Inst. of Civil Engineers, London, pp. 257-292, 1982.

[14] Meyerhof, G.G. Recherches sur la force portante des pieux. *Annales de l'institut technique du Batiment et des Travauz Publics*, 6(63-64), pp. 371-374, 1953.

[15] Meyerhof, G.G. Penetration test and bearing capacity of cohesionless soil. *Proc. of the Am. Society of Civil Engineers*, 82, SM 1, 866-1-19, 1956.

[16] Meyerhof, G.G. Bearing capacity and settlement of pile foundations. *ASCE J. of Geotechnical Eng. Division*, 102(GT 3), pp. 195-228, 1976.

[17] Meyerhof, G.G. Scale effects of ultimate pile capacity. *J. Geotech. Eng., ASCE*, 109(6), pp. 797-806, 1983.

[18] Mahmoud, M., Evans, R.S. & Trenter, N.A. The use of the pressuremeter test in weak rocks at the Port of Jebel Ali. Proc. 3rd Int. Symp. of Pressuremeters, the British Geotechnical Society, Telford, London, 1990.

[19] NAVFAC, DM-7.2, Foundations and earth structures, dep. of the Navy Naval Facilities Engineering Command, Alexandria, VA., 1982.

[20] Norlund, R.L. *Bearing capacity of piles in cohesionless soils.* J.S.M.F.D., ASCE, 89(SM 3), pp. 1-35, 1963.

[21] O'Neill, M.W. & Hassan, K.M. Drilled Shafts: Effects of construction on performance and design criteria. *Proc. Int. Conf. on Design and Construction of Deep Foundations, US-FNWA*, 1994.

[22] Poulos, H.G. *Design of Pile Foundation.* Research Report No 271. The University of Sydney School of Civil Engineering, Sydney, 1975.

[23] Puech, A., Boulon, M. & Meimon, Y. Tension piles: field data and numerical modeling. *Proc. 2nd Int. Conf. on Numerucal Method in Offshore Piling*, College of Engineering the Univ. of Texas at Austin and the Inst. of Civil Engineers, London, pp. 293-312, 1982.

[24] Reese, L.C. & O'Neill, M.W. *Drilled Shafts: Construction Procedures and Design Methods.* FHWA-HI-88-042, FHWA, Washington, D.C. 1988.

[25] Smith, M. A survey of numerical methods of offshore piling. Proc. Conf. on Num. Meth. in Offshore Piling, Inst. of Civil Engineers, London, pp.1-8, 1980.
[26] Summary of loading test on cast in-situ concrete piles and slurry walls, Ashkelon South - Power Station. Technion, Soil and Road Testing Laboratory and Soil Consultants of Soil Engineering Ltd., Haifa, 1982.
[27] Te Kamp, W.G.B. Static cone penetration testing and foundations on piles in sand. Fugro *Sounding Symp.*, Ulrecht, 1977.
[28] Terzaghi, K. *Theoretical soil mechanics.* John Wiley, New York, 1943.
[29] Terzaghi, K. & Peck, R.B. *Soil mechanics in Engineering Practice*, 2nd ed. John Wiley, New York, 1967.
[30] Thorburn, S. & R.S.L. Mac Vilar, Discussion. Proc. *Conf. of Behavior of Piles.* Inst. C.E. p. 50, 1970.
[31] Tomlinson, M.J. *Pile Design and Construction Practice*, 4th ed., E & FN SPON, 1994.
[32] Vesic', A.S. A study of the bearing capacity of deep foundation, Project B-189, Georgia Inst.Technol., Atlanta, Ga., xvi + 264 (final report), 1967.
[33] Vesic', A.S. Ultimate loads and settlements of deep foundations in sand. *Proc. Symp. on Bearing Capacity and Settlement of Foundations.* Duke University, Durham, North Carolina, pp. 53-68, 1967.
[34] Vesic', A.S. Principles of pile foundation design. Soil Mechanics Series No 38, Duke University, 1975.
[35] Vesic', A.S. *Design of Pile Foundations.* Report 42, National Cooperative Highway Research Program Synthesis of Highway Practice, Transportation Research Board, National Research Council, Washington, D.C., 1977
[36] Zienkiewicz, O.C. et al., Associated and non-associated viscoplasticity and plasticity in soil mechanics. *Geotechnique*, 25(4), pp. 671-689, 1975.

Acknowledgment

The author to acknowledges Professor Amos Komornik without whose encouragement and support this research work would not have materialized.