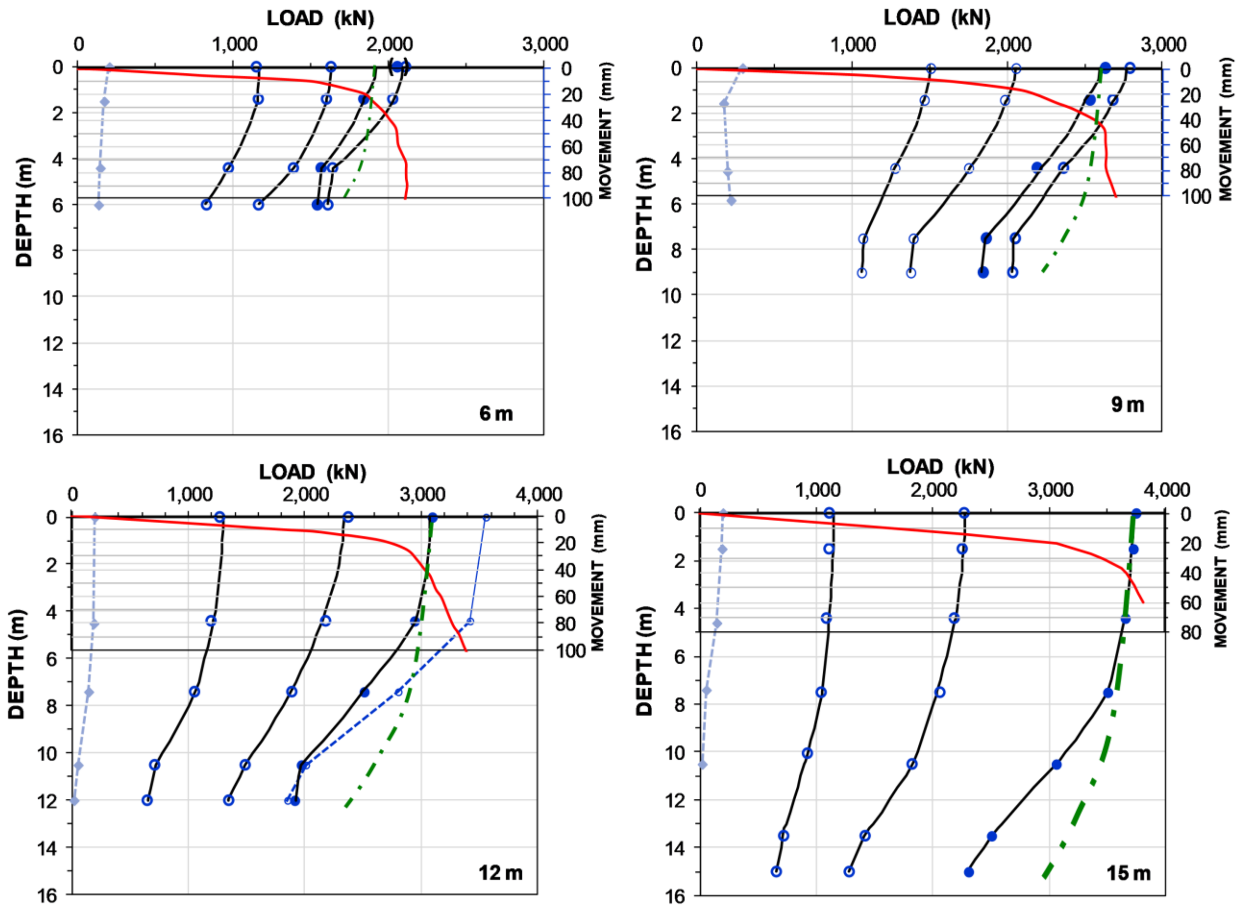

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Case Study

Revisiting Vesic (1970): Tests on Instrumented Piles, Ogeechee River Site

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Abstract: In a paper in the ASCE Journal, Vesic (1970), presented a case history on static loading tests on strain-gage instrumented pipe-piles that gained a wide national and international attention. The paper evaluated the case records and derived a concept later called "critical depth" that became generally accepted as basis for the design of piled foundations. That concept stated that, down to the "critical depth", starting at a depth of about to 10 pile diameters, the ultimate unit shaft and toe resistance of a pile would develop in conformity with the effective overburden stress. Below this depth, the unit resistances would cease to correlate directly to the effective stress and become constant with increasing depth below 20 pile diameters. The concept was oblivious to the presence of residual force, which, had it been considered, would have removed the appearance of a critical depth.

Keywords: pipe pile, strain-gages, static loading tests, critical depth, residual force

Introduction

In 1970, Aleksandar Vesic published results of full-scale tests on an instrumented test pile, a 457-mm diameter closed-toe pipe pile with 12.5-mm wall driven 15 m into sand. The main results were compiled in two graphs showing the distribution of ultimate unit toe and shaft resistances—scanned, digitized, and replotted in Figure 1. The test pile was driven to 15.0 m depth and the driving was interrupted every 3.0 m, to carry out a head-down static loading test (the day after) employing a combination of platform load and tension reaction piles. Vesic summed up the test results in terms of shaft and toe resistances as "*both shaft resistances increased approximately linearly with depth over a limited zone not exceeding 10 pile diameters. Beyond a depth of approximately 20 pile diameters, both resistances reach nearly constant final values*". Vesic's paper was recognized far and wide and his conclusion from this test, was accepted as a generally applicable fact. Later on, the depth range of 10 through 20 diameters became known as "the critical depth" and the concept of the unit pile resistances being constant below the critical depth was widely accepted and applied. Soon after 1970, the concept appeared in national and international text books, standards, and manuals. The implication on costs of piled foundations has been large. Although the concept was soon proven wrong, it is still on occasions brought into play in actual pile foundation design. It is therefore important to show how it came about by re-visiting Vesic (1970).

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Soil Profile

The Ogeechee River test site is located in Effingham County about 30 km west of Savannah, Georgia, and comprises deep deposits of medium dense to dense sand. Figure 2 shows the results of CPT sounding and borehole SPT N-indices, both within about 3 m distance from the test pile (the CPT was not pushed to the full depth of the test pile). The borehole sampling and N-count were at every 0.6 m depth. Figure 3 shows the ranges of grain size distributions of the soil deposit determined by sieve analyses of soil samples.

The soil consisted of medium to coarse sand. The q_c-values and N-indices indicated loose to medium density to about 3 m depth, medium to dense density to about 12 m depth, and, then, very dense. The groundwater table during the field investigation was at 2.2 m depth. During the static loading tests, it was at 3.0 m depth.

Test Depths and Gage Positions

The piles were instrumented by means two diametrically opposed pairs of electrical strain gages with a 10-kN stated accuracy. Figure 4 shows the pile toe depths of each of the five static loading tests and the depths of the strain-gages (SG-1 through SG-6) for each test. The q_c and N distributions graph is added to provide reference of the pile and gage depths to the soil profile.

Test Procedure

The schedule involved driving the test pile to the desired depth, and performing the static loading test the next day followed by driving to the next depth. Thus, the testing programme was completed in five days.

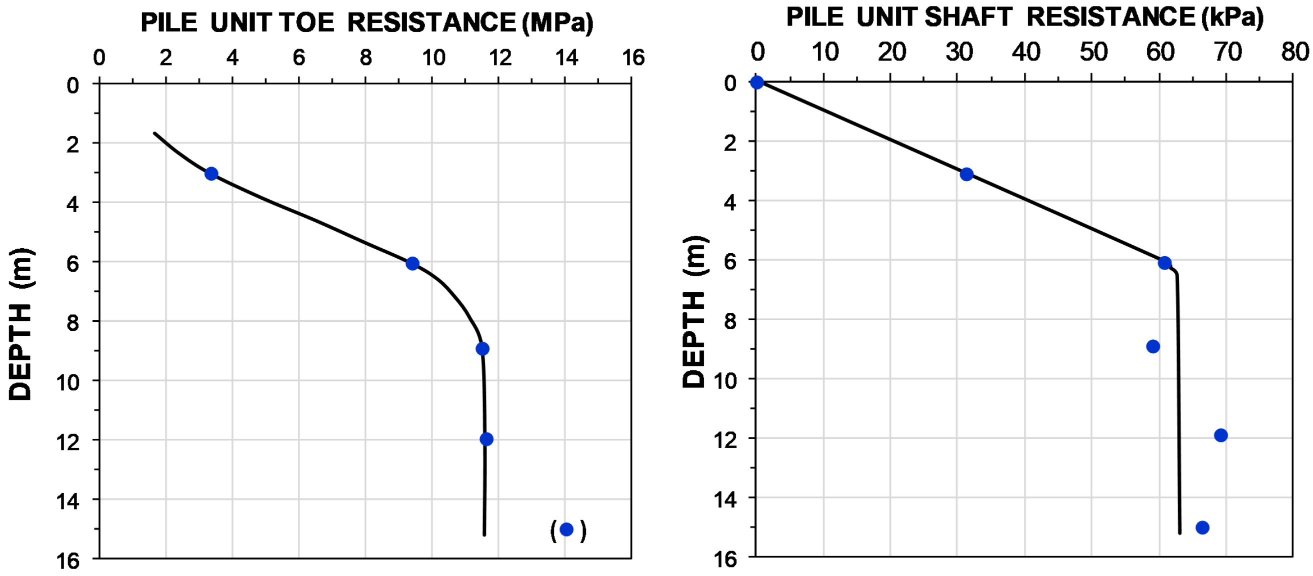

Figure 1. Main 'finding' by Vesic (1970)

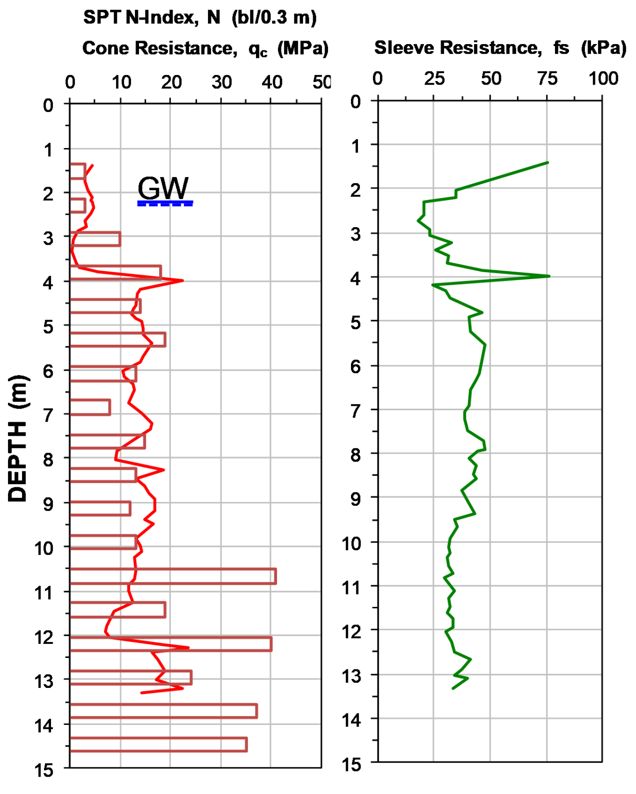

Figure 2. Results of CPT sounding and SPT N-indices near the test pile

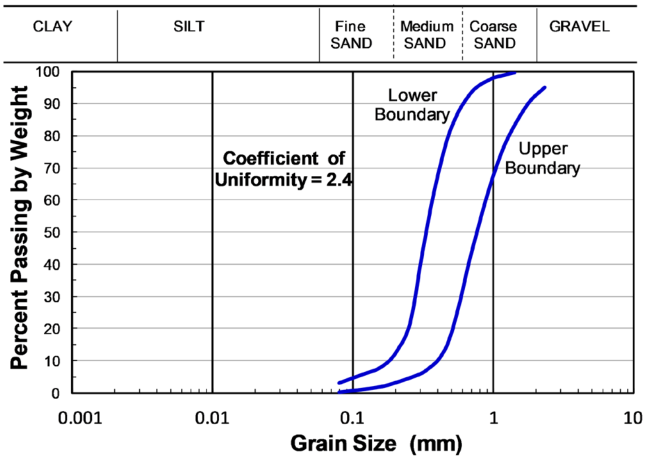

Figure 3. Sieve analysis results

The loading test procedure was by constant-rate-of-penetration, C.R.P.-method (Whitaker and Cooke 1961), with the rate being 1.27 mm/minute (0.05 inch/minute) as guided from the movement of one of the dial gages measuring the pile head movement (dial gage gradation was 0.001 inch). Thus, the time for reaching 50 mm pile-head movement was only 40 minutes. The actual rate was controlled by adjusting the applied jack load (controlling the manually operated pump) aiming to achieve a steady rate of pile head movement as judged from observing a dial gage. Readings of load, pile head movement, and strain-gage records were taken at one-minute intervals and the five-minute readings were recorded. (I have found that with a manually operated pump, a truly constant rate of penetration is very difficult to ensure. A reliable C.R.P. test requires a pump that provides an "automatic" continuous constant rate of oil flow to the jack. Moreover, the continuous build-up of force at the gage levels causes the forces determined at the various gage levels to become less reliable for static reference. Therefore, the C.R.P. method is not suitable for instrumented full-scale pile tests).

Test Results

The main test results are the pile-head load-movement curves and the force distributions recorded by the strain-gages. Figure 5 shows these data for the tests at 6, 9, 12, and 15 m depths, leaving out the test at 3 m depth (the depth-to-diameter

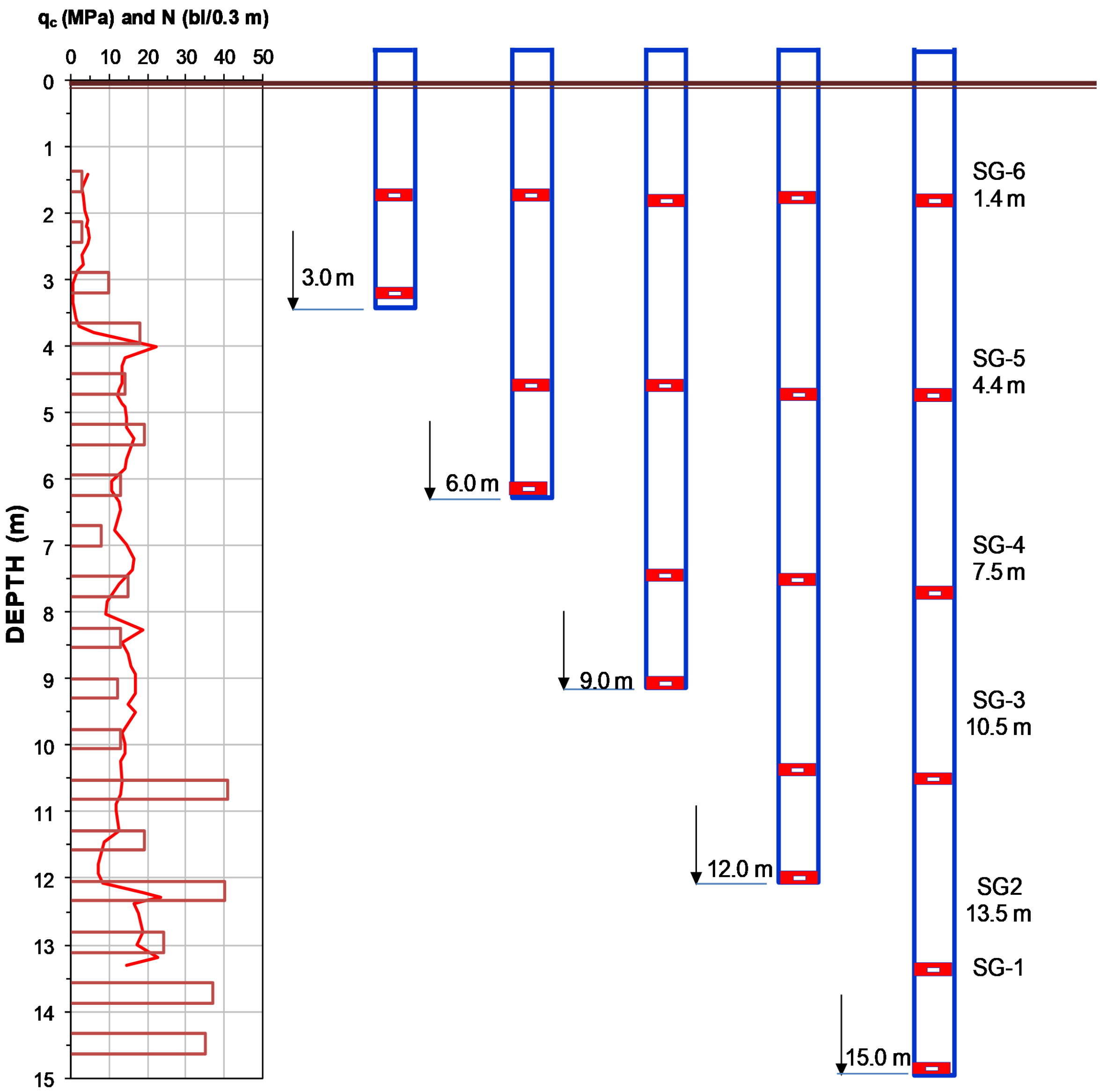

Figure 4. Pile length and gage levels for the five loading tests

ratio being more similar to a pier than a pile for this shallow pile length). The paper limited the force distribution displays to four, with the fourth showing the maximum pile-head load applied. The third force distribution was for the load that caused a load-movement of 10 % of the pile diameter, which was defined as the ultimate resistance. Vesic(1970) here referenced Terzaghi (1942) as the proponent of this definition. However, Terzaghi did not propose this. The statement, repeated by many, that Terzaghi proposed this definition is a misinterpretation of his stating that ultimate resistance should not be considered until the pile toe has moved 10 % of the pile diameter and that the ultimate resistance can then be larger or smaller than the load that resulted in a movement equal to 10-% of the pile-head diameter (Likins *et al*., 2012).

The distributions show very small shaft resistance along the lowest part of the test piles, which is commensurate with presence of residual force and positive unit shaft resistance along this pile length. The dashed-dot distribution curves starting from the third load distribution curve in each figure suggest a possible true force distribution that includes a presumed residual force along the pile and demonstrate the consequence of disregarding presence of residual force when interpreting force distribution measured in instrumented tests.

Unfortunately, the accuracy of the strain-gage records is not of a reliability that would allow a detailed analysis,. Moreover, the paper mentions that "*a few missing readings from non-recording gages were interpolated*". However those values are not identified.

Had Vesic (1970) been aware of the Mansur and Kaufmann (1956) paper, he would have been able to show that those records also indicated a response additionally supporting the critical depth concept. That is, provided that the presence of residual force would be disregarded. That Vesic (1970) did not reference the Mansur and Kauffman

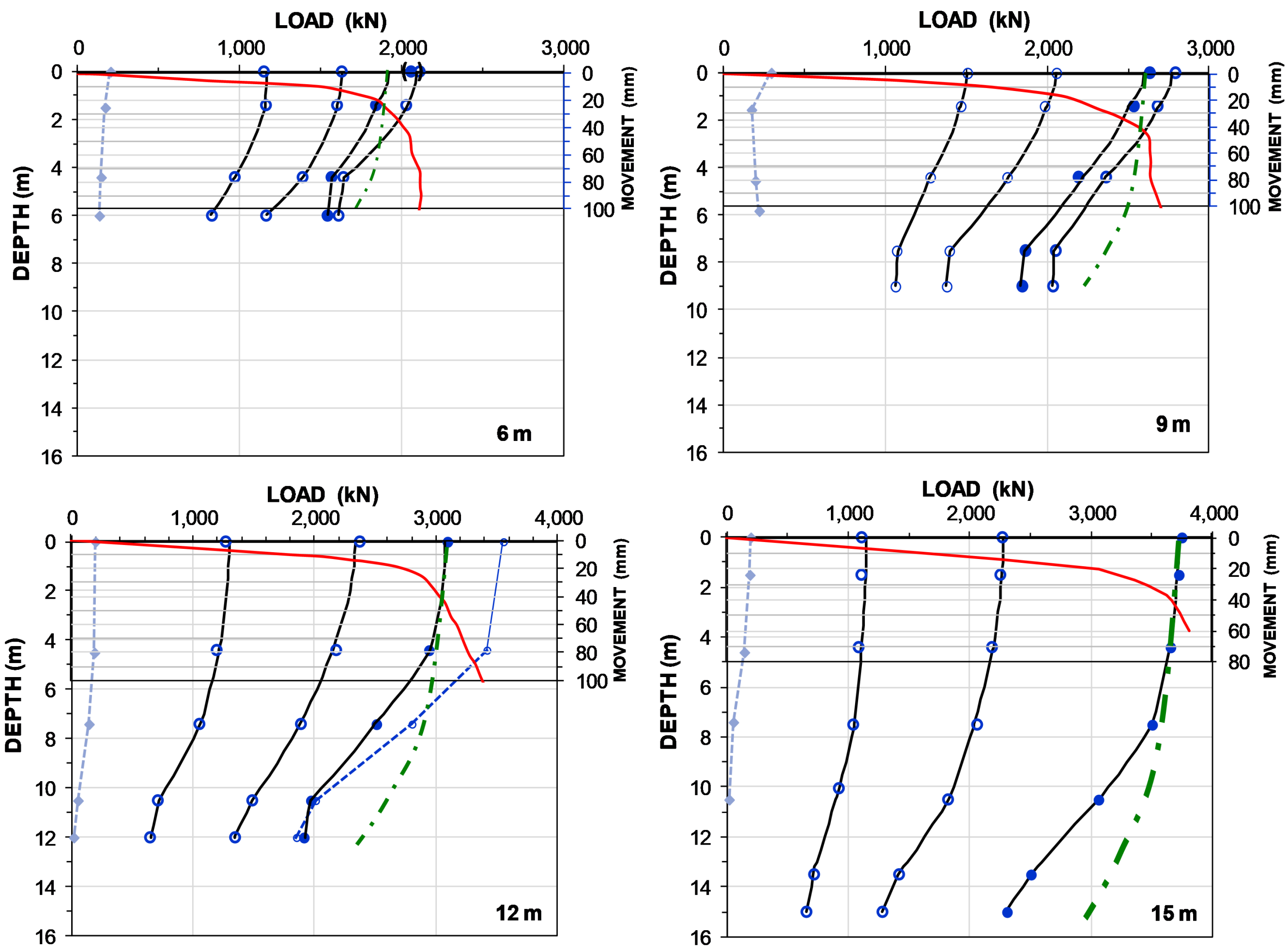

Figure 5. Force distributions measured in the test on the 6,9,12, and 15 m embedment piles with a pile head load movement curve added for reference

(1956) paper can be understood in the light of the latter paper did not include a CPT sounding, which the subject paper emphasized. Neither was Vesic (1970) aware of the paper by Nordlund (1963), who pointed out that Mansur and Kaufman had disregarded the presence of residual force and therefore, their test interpretation was incorrect, as, then, also Vesic's interpretation of the subject case would be.

Nordlund assumed that the residual toe force was equal to the pile-toe force evaluated from the pull test, that is, that the presence of residual force would be equal for the push and pull test. In reality, the residual force present at the start of the push test is not equal to that after the unloading from the push test, that is, the residual force at the start of the pull test cannot be assumed equal to that at the start of the push test. Nor would the residual force remaining in the pile after the end of the pull test be equal to the one at the start of the pull test. It was left to Hunter and Davisson (1969) to be the first to fully discuss the effect of presence of residual force in evaluating results of loading tests on instrumented piles.

Nordlund, as did Vesic and many others, based the analysis on a single value of applied load, the one corresponding to capacity. But people do not employ the same definition for capacity. Mansur and Kaufmann (1956) considered the value to be the average of three capacity definitions based on the load-movement curve. Vesic (1970) defined capacity as the applied load that caused a movement equal to 10 % of the pile diameter, and Nordlund (1963) defined capacity as the pile-head load for which the tangent to the load-movement curve was 1 mm/7 kN (0.05 inch/ ton).

The Nordlund (1963) paper spent most effort on general shaft resistance response and, in particular, the effect of pile taper, so the alert in the paper in regard to residual force can easily be missed. In due course, obviously, Vesic became aware of the importance of presence of residual force. In Vesic (1977), he addressed the effect and referenced both Mansur and Kauffman (1956) and Hunter and Davisson (1969).

At the time, Vesic was not alone in being oblivious of the presence of residual force. For example, O'Neill and Reese (1972) published results of tests on 760 mm diameter, 7.6 m long bored piles in Beaumont Clay and to explain the force distributions measured for one test pile shown in Figure 6, stated that the shape of the distribution could be analogous to those measured for the Vesic (1970) test pile in sand (although the pile was only 10 diameter in length). The

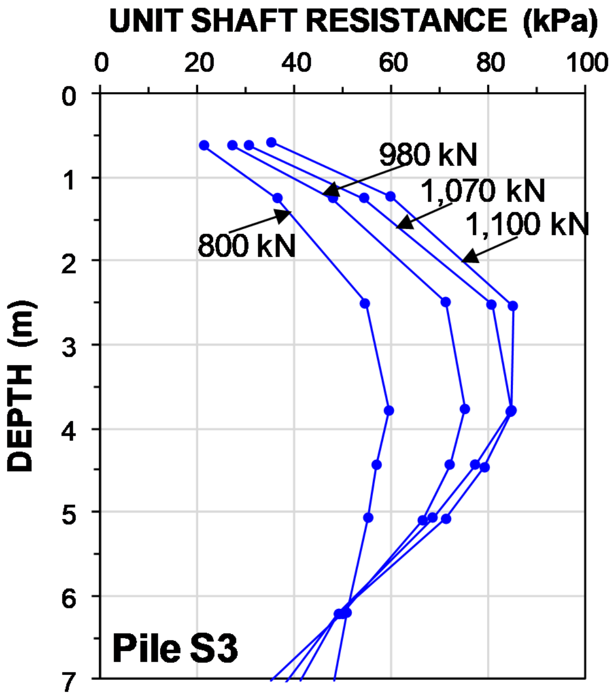

Figure 6. Distribution of unit shaft resistance (after O'Neill and Reese 1972)

distributions were measured for applied loads beyond the pile capacity ranging from 800 through 1,100 kN.

The mistake in interpretation of test records that resulted in the critical depth concept must not distract from Aleksandar Vesic's numerous outstanding contributions to foundation engineering knowledge. We all have baggage in our geotechnical contributions. For example, Vesic's omission of presence of residual force is minimal in reference to my own gaffe, albeit 50 years ago, in my then incorporating drag force with the sustained load in determining a maximum allowable applied load.

References

Hunter A.H. and Davisson M.T. (1969). Measurements of pile load transfer. *Proc. of Symposium on Performance of Deep Foundations*, San Francisco, June 23-28, 1968

Likins, G.E., Fellenius, B.H., and Holtz, R.D. (2012). Pile Driving Formulas—Past and Present. Full-Scale Testing and Foundation Design: Honoring Bengt H. Fellenius https://doi.org/10.1061/9780784412084.0051

Mansur, C. L., and Kaufman, R. I. (1956). Pile tests, low sill structure, Old River, Louisiana. Pile tests, low sill structure, Old River, Louisiana, Transactions, *ASCE J*. 82(10) 715–744.

Nordlund, R.L. (1963). Bearing capacity of piles in cohesionless soils. *Journal of the Soil Mechanics and Foundations Division*, Volume 89, Issue 3, pp 1-35. https://doi.org/10.1061/JSFEAQ.0000507

O'Neill, M.W., and Reese, L.C. (1972). Behavior of bored piles in Beaumont Clay. *J. Soil Mech. Found. Div*., ASCE, 98(2), 195–213.

Terzaghi, K. (1942). Discussion of the Progress Report of the Committee on the Bearing Capacity of Pile Foundations. ASCE 68(2) 311 323.

Vesic, A.S. (1970). Test on instrumented piles, Ogeechee River site. *Journal of the Soil Mechanics and Foundations Division*, Volume 96, Issue 2, pp561-584. https://doi.org/10.1061/JSFEAQ.0001404.

Vesic, A.S. (1977). Design of pile foundations. NCHRP Synthesis 42, 80 p., retrieved from: https://onlinepubs.trb.org/Onlinepubs/nchrp/nchrp_syn_42.pdf.

Whitaker, T. and Cooke, R.W. (1961). A new approach to pile testing. *Proc. 5th ICSMFE*, Paris, July 17-21, Vol. 2, pp. 171-176.