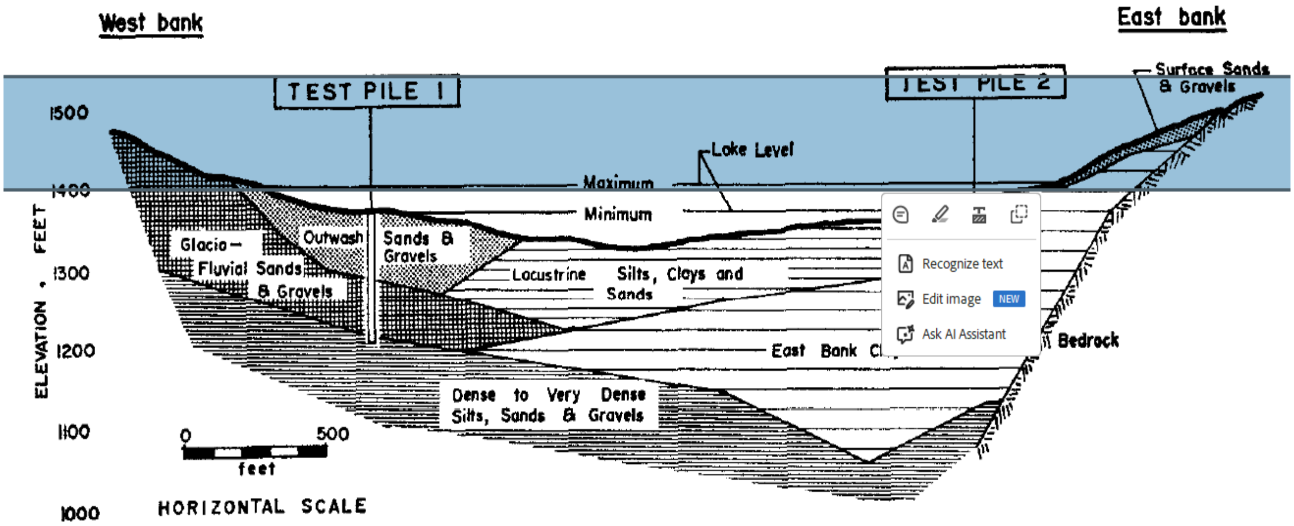

Fig. 1. Subsoil stratigraphy at test site

Fellenius, B.H., 2025. Revisiting Meyerhof et al. (1981) and McGammon and Golder (1970)—Analysis of tests on piles in want of instrumentation. Journal of the Deep Foundation Institute, 19(1) 10 p.

Case Study

Revisiting Meyerhof et al. (1981) and McGammon and Golder (1970) - Analysis of tests on piles in want of instrumentation

Bengt H. Fellenius[1]*

Abstract: Two old case history papers comprising static loading test on single piles are reanalyzed. Both address separating shaft and toe resistances from the capacity determined from the pile-head load-movement curve. The first paper, Meyerhof *et al.* (1981), involved a precast concrete pile driven into a clay till where the toe response was estimated by theoretical analysis. The second paper, McGammon and Golder (1970), involved pipe piles in sand and clay, where the toe response was determined as the difference of resistance between a test on a open-toe pipe pile for which all toe resistance had been removed by drilling out the pile core and a following test on the pile after restoring the toe resistance by grouting the pile and redriving it a short distance. The test data are digitized and presented in graphics with, for the 1981 paper, a discussion on the relevance of the theoretical analysis. For the 1970 paper, because the capacities were not obtained at the same movements relative to the soil, the suitability of comparing the two tests is questioned. The today available software resources of analysis enabled additional insights of the old case histories to be extracted, revitalizing their value to the profession.

Keywords: driven piles, load-displacement performance, experimental testing, case history revisit, toe and shaft resistance

Introduction

The load from a supported structure applied to a piled foundation is sustained by shaft and toe bearing. Therefore, a foundation designer needs to determine both responses, be it a theoretical prediction or as based on an actual test. A static test, however, is usually carried out only by applying load to the pile head, establishing a "capacity" by some definition or other, with the capacity distributed as part shaft resistance and part toe resistance. The two interact and uncertainty of one will lead to uncertainty of the other, as demonstrated by the first of the here reanalyzed two case histories, comprising precast concrete piles. The second case history comprises open-toe steel tube piles and shaft and toe responses were separated by performing, first, a static loading test after having removed all toe resistance from the test pile by cleaning out the core and, then, after restoring the toe bearing (concreting the pile) and driving the pile a short additional distance, performing a second test. The difference between the two tests would represent the actual toe bearing of the two tests. The fallacy of both approaches lies in the fact the "capacity" of the purely shaft-bearing pile does not occur at the same movement as the "capacity" of the pile bearing by combination of shaft and toe responses.

[1] Consulting Engineer, 2475 Rothesay Ave, Sidney, BC, Canada, V8L 2B9
* Corresponding author, email: bengt@fellenius.net

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Head-down test on a driven precast pile

Meyerhof *et al.* (1981) reported results from a head-down, static loading test performed in about 1970 on a 305-mm diameter, 13 m long, hexagonal, precast concrete pile (H800 with 800 cm^2 area) driven to support the foundations of a bridge in Eastern Canada. Figure 1 shows the soil profile encompassing distribution of water contents, Atterberg limits, undrained shear strength, and STP N-indices in the firm clay till at the site with the strength increasing with depth. The water content, about 10 %, corresponds to a 2,250 kg/m^3 total density. The till was stated to be preconsolidated with an OCR of about 10, which means that the preconsolidation margin was about 1,000 kPa at about 7 m depth. Consolidated-undrained triaxial tests indicated a 29° internal friction angle and a 16 kPa cohesion intercept. The earth stress coefficient at rest, K_0, was stated to be 1.5 before pile driving. The depth to the groundwater table was not mentioned, but my recollection of the site is that it would have been at about 2 m below the ground surface. (I was at the time working for the pile driving contractor and I remember visiting the site at the time of the loading tests; about 1968). The paper also included test results on a second pile (H420 with 420 cm^2 area). Because the H420 results are in all aspects very similar to those on the H800 pile, these records are not included here.

The site investigation included two compression screw-plate (160-mm width) tests at depths of 3 and 6 m. Figure 2 shows the stress (kPa) versus movement (% of plate diameter). The 6 m depth test curve is fitted to a Gwizdala q-z function (Gwizdala 1995, Fellenius 2025), which resulted in an exponent, θ, equal to 0.600. Equation 1 expresses the general q-z function. The close fit to the test curve labeled

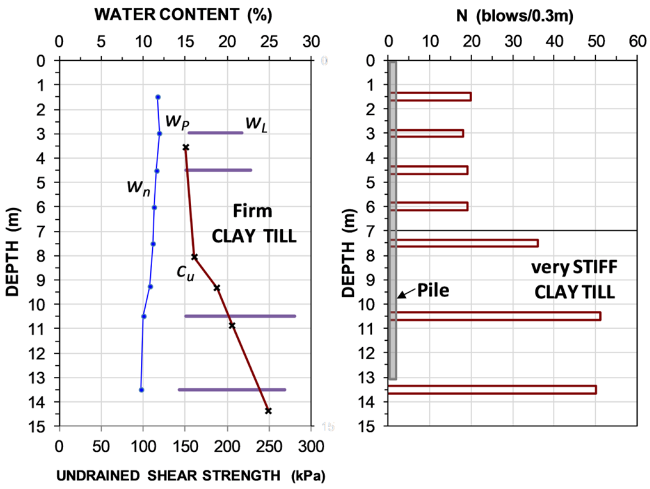

Figure 1. Soil profile

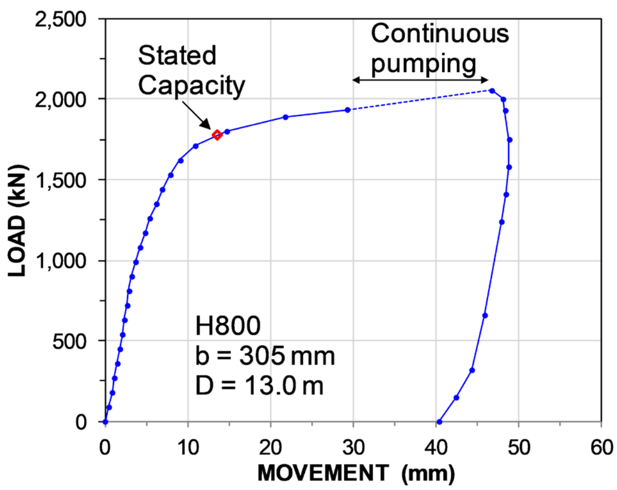

Figure 3. Load-movement from static loading test with data points digitized from Meyerhof *et al.* (1981)

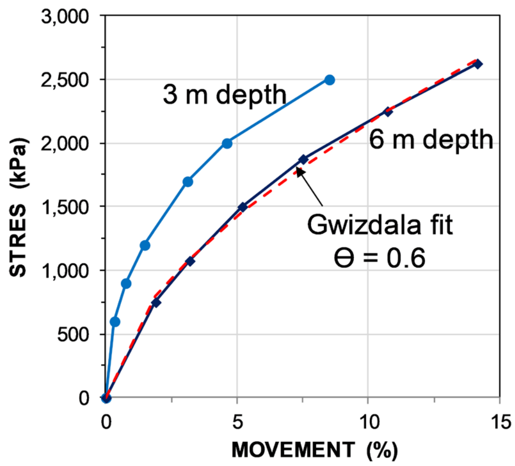

Figure 2. Stress-movement graphs from the screw-plate tests

"Gwizdala Fit" was obtained by entering the coordinates, r_1/δ_1 and r_2/δ_2, of two points on the curve, and the equation gave the exponent, θ, of 0.60.

$$r_1 = r_2(\delta_1/\delta_2)^\theta \tag{1}$$

where r_1 and r_2 = unit forces at Points 1 and 2, respectively
δ_1 and δ_2 = movements at Point 1 and 2, respectively
θ = Gwizdala function coefficient

Figure 3 shows the pile-head load-movement curves from the static loading test. Meyerhof *et al.* (1981) interpreted the capacity to be 1,780 kN (plotted in the graph at 13.4 mm pile-head movement), but gave no indication of how it was defined. However, it happens to be very close to the Davisson Offset Limit (Davisson 1972).

To determine an ultimate toe resistance, Meyerhof *et al.* (1981) referred to the screw-plate tests and concluded that the ultimate pile-toe toe resistance of the test pile, total, R_t, and unit toe resistance, r_t, were 165 kN and 2,000 kPa, respectively. Dividing the latter by a factor of 9 (they referenced Tomlinson 1957), gave a an undrained shear strength of 220 kPa at the pile toe level that they stated agreed well with the undrained shear strength construed from the screw-plate records (c.f., Figure 1). There was no mentioning of the fact that the screw-plate test showed no ultimate value and that it would be unlikely that the pile toe would either.

A force-movement response of a pile toe is almost always in the form of a Gwizdala q-z function much like the screw-plate test and like ordinary vertically and centrically loaded footings. If accepting the 165 kN value as a target force for the toe response, and applying the same Gwizdala function coefficient (θ = 0.60) as that found for the screw-plate test, the target toe force would have been reached at a 6.5 mm toe movement. Assuming a pile E-modulus equal to 35 GPa, this movement agrees well with that of estimated pile compression subtracted from the measured pile head movement for an applied load equal to the stated capacity.

To the ultimate shaft resistance, Meyerhof *et al.* (1981) applied two approaches. A total shaft resistance of 1,060 kN was obtained by applying shear strength data determined from the soil profile and a value of 1,615 kN was obtained by applying an effective stress approach ($r_s = \beta\, p'$ and $\beta = K_s \tan\delta$; Burland 1973). The authors mentioned that that the latter method gave more reliable results. The values correspond to an average β-coefficient of 1.2. Considering that the soil was softer above 7 m depth than below, the same resistance is obtained by applying beta-coefficients of 0.9 above 7 m depth and 1.3 below. Figure 4A shows the resulting force distribution for an applied load equal to the stated capacity. The mentioned β-coefficients are larger than usually found, which could presumably be due to the till being considerably preconsolidated. Figure 4B shows the shaft response as distribution of unit shaft shear values, r_s and c_u.

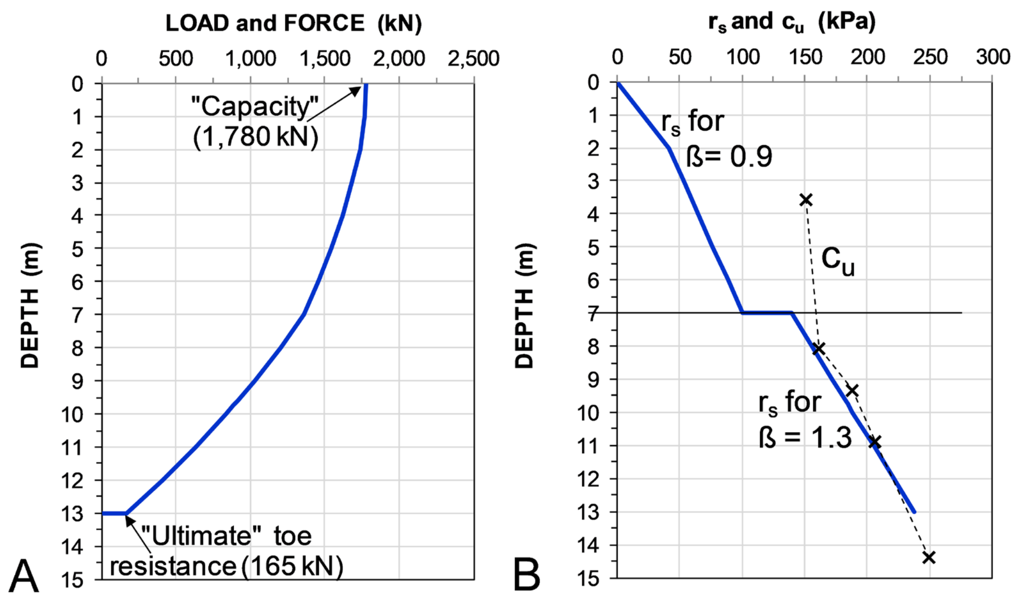

Figure 4. Force distribution for the load equal to the stated capacity and its correlated distribution of unit shaft resistance with stated undrained shear strength

The 1981 paper was written before the advent of the personal computer and software solutions. Now available software, here UniPile (www,UnisoftGS.com), allows simulation of a the measured pile-head movement and assumed pile-toe resistance at the stated pile-toe force and a pile-toe movement correlated to the measured pile-head movement and assumed shaft response. Figure 5 shows the results of such simulation. In matching the simulation to the actual load-movement curve, keeping the q-z function constant, controlled the t-z input for the shaft resistance simulation. The match indicated in the figure was obtained for input of a Zhang strain-softening t-z function with a 0.016 function coefficient and 12 mm target movement for the pile elements (Fellenius 2023).

The largest unknown in the forgoing analysis is the value of the target toe resistance. I have difficulty in accepting the 165-kN toe resistance, expecting it to be much larger. As mentioned, I actually worked for the piling contractor at the time of the project and, while I was not involved in the test, I saw the site and remember that the piles drove relatively easy until the last few feet where the blow-count picked up considerably. I would therefore expect that the toe resistance would be much larger than that calculated as undrained shear strength multiplied by 9. Moreover, the beta-coefficients, as back-calculated from the target resistance (taken as equal to the stated ultimate shaft resistance), appear unusually large. Beta-coefficients of 0.3 and 0.5 in the two layers, respectively, would be more usual. This simulation results in a 500-kN shaft resistance and, thus, a 1,200-kN balance from the stated capacity being a toe resistance commensurate with the load-movement at the stated target ("capacity"). To fit the simulated pile-head load-movement curve to the measured, the t-z functions for the shaft elements are set to a Zhang t-z strain-softening function with a 0.015 function coefficient and 6-mm target

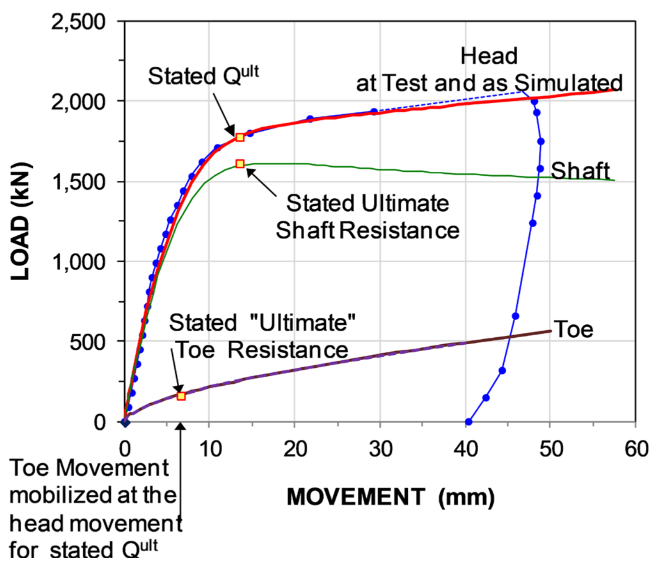

Figure 5. Measured and simulated pile-head load-movements and stated ultimate resistances and simulated pile-shaft and pile-toe load-movements

movement for the pile elements and the toe response was set to a Gwizdala q-z function with a 0.20 function coefficient and a 6 mm target movement. The 0.20 toe function coefficient is very low and indicates that the pile was affected by locked-in axial force (residual force) at the start of the test. Figure 6 shows the load-movement curves resulting from this new set of assumptions. The fit between the measured curve and the simulated is as good as that shown in the preceding figure.

The second simulation of the test cannot be stated to be more true than the first. Were I to bet on one or the other, knowing Meyerhof's considerable experience from projects

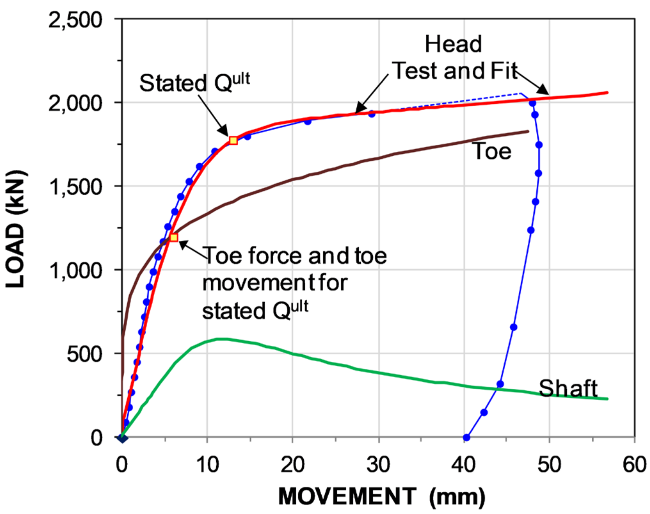

Figure 6. Measured and simulated pile-head load-movements and stated ultimate resistances and new simulated pile-shaft and pile-toe load-movements

in the area, my money would be on the 1981 analysis. However, were the case to involve more than pride and a friendly bet, I would recommend a test that included instrumentation to measure the pile toe force (the axial force near the pile toe). Simply put, an important static loading test must encompass more observation than just the load-movement of the pile head. Today, instrumentation for measuring strain and, thus, force distribution, is routine and must not be omitted, or the value of the test results would be limited to that of a simple proof test.

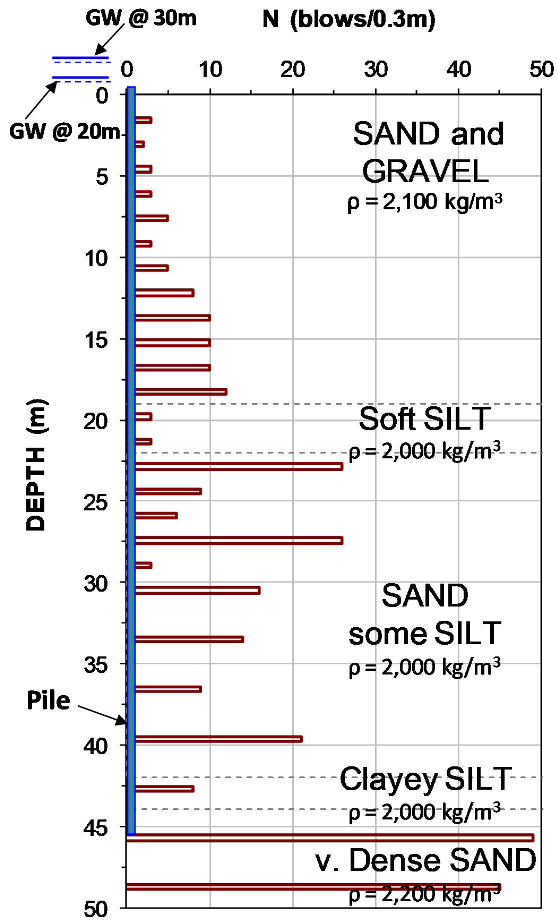

Figure 7. Soil profile and SPT-indices at west side, Tests 1A and 1B

Shaft and toe resistances in head-down tests on 24-inch driven pipe piles

McGammon and Golder (1970) reported two series of head-down static loading tests at the Lower Arrow Lake in British Columbia on 619 mm diameter driven pipe piles. One pile each was driven on the sides of a river in a glaciated valley with the soil profile on one side, west side, comprising silty sand profile and the other one, east side, a clay profile. The tests combined testing piles with only shaft resistance and with both shaft and toe resistances. The purpose of the test was to establish pile capacity for reference to planned actual bridge foundation piles for approach piers and, also, to serve as scale model to 1,524-mm diameter piles considered for the main bridge piers.

Test 1, pipe pile driven in silt and sand

An open-toe pipe pile was driven in four 12 m long segments, through the soil profile summarized in Figure 7. The soil densities indicated in the figure are estimated values. Standpipe measurements of the groundwater table showed that the pore pressure distribution was artesian with heights rising to 1.2 and 2.7 m above the ground surface, as measured at 20 and 30 m depths, respectively. The paper mentions that that a newly completed downstream dam will raise the maximum water level by about 15 m. That will increase the artesian height for the bridge foundations, if not by 15 m, so by an appreciable amount.

Test pile 1 was driven to 45.4 m depth. After each 12 m length of driving, all soil inside the pile was removed and replaced with water. At end of driving, the cleaning was extended to 0.3 m beyond the pile toe into the very dense sand at the pile toe level. A static head-down loading test, Test 1A, was carried out nine days later. Thereafter, a 10 m long concrete plug was tremied into the pile, and, 15 days later, the pile was driven to 46.0 m depth (0.3 m deeper; the penetration resistance was 25 blows/25 mm). A week later, a second static loading test, Test 1B, was carried out.

Figure 8 shows the test schedule and measured load-movement response for Test 1A. The loading started with three increments of 445 kN (50 ton). The first two were held for 30 minutes, the third for one hour. The 4th increment was half size (222 kN added to 1,557 kN total load) also held for one hour. The paper includes no reason for this illogical loading schedule. When attempting a fifth load increment, continued jacking was required and large movements occurred; the pile plunged and the test was then terminated after a few minutes of continued pumping. It is not possible to discern whether the post-peak response was plastic or strain-softening.

The maximum load for Test 1A, 1,557 kN (175 tons)—the ultimate resistance—was used as target load for effec-

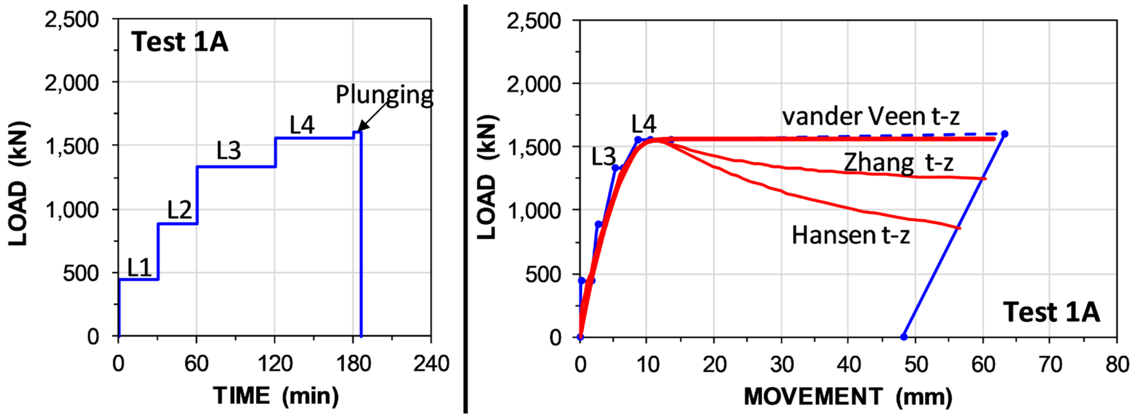

Figure 8. Load-time schedule and measured load-movement curves for Tests 1A

tive-stress back-analysis, which gave a beta-coefficient of 0.07 down to 42 m depth in the sand and gravel layer and, then, 0.11 and 0.16 in the clayey silt and dense sand, respectively. These numbers are unusually small considering the fact that the associated movement was close to 10 mm and the pile had reached a plunging state. They correspond to average unit shaft shear values of about 15 kPa, and 46 and 62 kPa, respectively, and 22 kPa on average to 22 m depth. The average shaft resistance below 22 m depth was 33 kPa. Possibly the actual pore pressures were larger than mentioned in the paper and, therefore, the effective stress smaller than assumed here. A more probable reason is that the shaft resistance never recovered from the disturbance of the re-driving by the process indicated by Saye *et al*. (2020). McGammon and Golder's (1970) concluding recommendation was that the design of the west side bridge foundations should apply 19 kPa as ultimate unit shaft resistance.

The Test 1A simulated load-movement curves were produced by simulating force-movement response of the pile elements to the pile-head movement before and after the peak applied load—assuming the same pile element shear-movement relation (t-z function) for all elements. For target response, the mentioned back-calculated *β*-coefficients were fitted to the maximum applied load (the fit could just as well been to unit shaft resistance values). A vander Veen elastic-plastic t-z function gave the fit to a potential plastic resistance as post-peak response. Because the post-peak response was not measured, it is not possible to determine whether or not the shaft response was plastic or strain-softening. For example, both a Zhang or a Hansen strain-softening response (shown in Figure 9) also fit the measured before-peak pile-head response. Details on the functions are in Fellenius (2025).

Test 1B measured pile-head load-movement was simulated by means of assuming the pile element shear-movement relation, t-z function, and a q-z function for the toe response. The same t-z vander Veen function was used as the one used for the back-calculation of Test 1A. The lengthening and the stiffening of the pile due to the

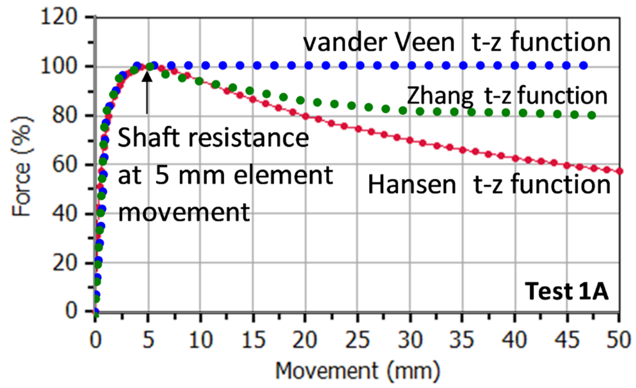

Figure 9. Pile element t-z relations for matching back-calculated axial maximum force

10 m concrete plug and the additional 0.3 m of driving was incorporated (the latter added 80 kN of shaft resistance). Figure 10 shows the final fit to the test records, as achieved with a vander Veen t-z and Gwizdala q-z functions. The Gwizdala function coefficient was 0.80, representing an almost linear relation. The fit includes adjusting the t-z function movement for reaching the peak shaft resistance from 5 mm to 15 mm, or the simulated movement for the first four loading steps would have been too stiff. This difference in t-z responses might be due to the fact that Test 1B is a reloading of the pile. A back-calculation of the Test 1B, records, considering the mentioned 80 kN increase, gave a 2,800 kN toe resistance at the maximum applied test load, which correlates to a unit toe resistance of 9.6 MPa developed at a 38-mm pile toe-movement. This stress is also the stress that McGammon and Golder (1970) recommended that the design of the west side bridge foundations should take as the ultimate unit toe resistance.

It was not possible to achieve a good fit assuming a strain-softening t-z function for Test 1B. Already a slight strain-softening would have required the q-z function coefficient to be equal to unity, which is the realistic limit of the function coefficient to use. Figure 11 shows the t-z and q-z functions that gave the fit.

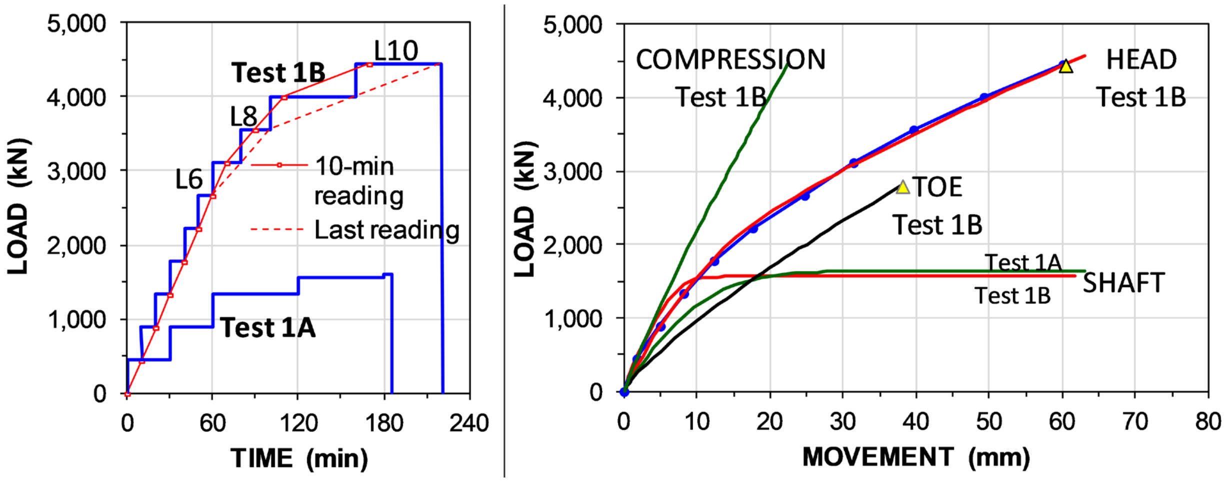

Figure 10. Actual and simulated (vander Veen t-z) loading-test results

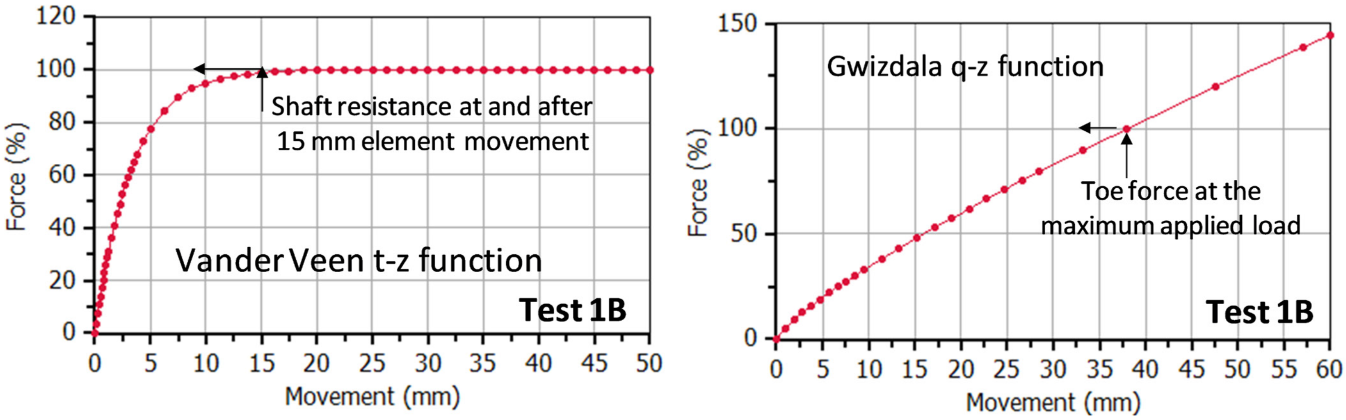

Figure 11. Pile element t-z and q-z relations for matching back-calculated maximum force

Test 2, Pipe pile driven in clay

The same type of pile was tested across the river in the clay profile, as summarized in Figure 12. The soil profile comprises a stiff, possibly slightly overconsolidated lacustrine clay. It is interesting to see the that two sides of the river exhibited different soil profiles. Standpipe measurements of groundwater table showed that the pore pressure distribution was artesian with heights seasonally rising to several meters above the ground surface. However, as the dates of the static tests were not stated, the pore pressure distribution at the time of the tests is not known. The pore pressure measurements reported in the paper to analyze the measurements assume a groundwater table at the ground surface, and a distribution represented by artesian height of 1 m above ground at 6 m depth and artesian height of 3 m above at 40 m depth.

Five tests, 2A through 2E, were carried out. Tests 2A and 2B had the soil removed from the inside of the pile and about 0.5 m beyond the pile toe. Test 2A was carried out about 26 hours after the end of driving (continuous) to 30.5 m depth and Test 2B was carried out nine days later.

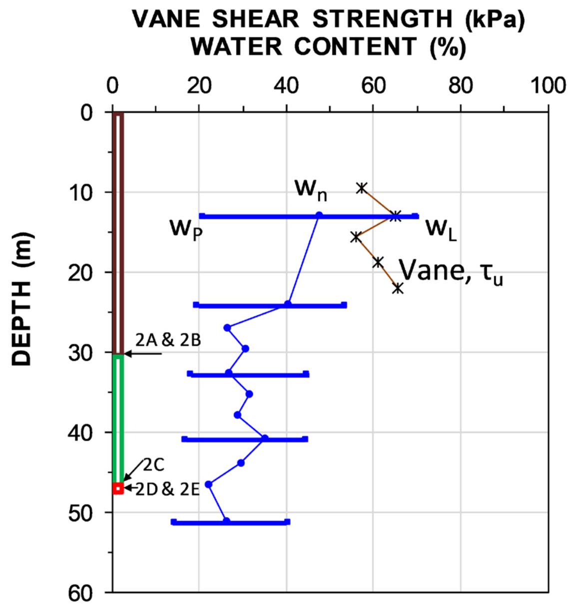

Figure 12. Soil profile at east side, Tests 2

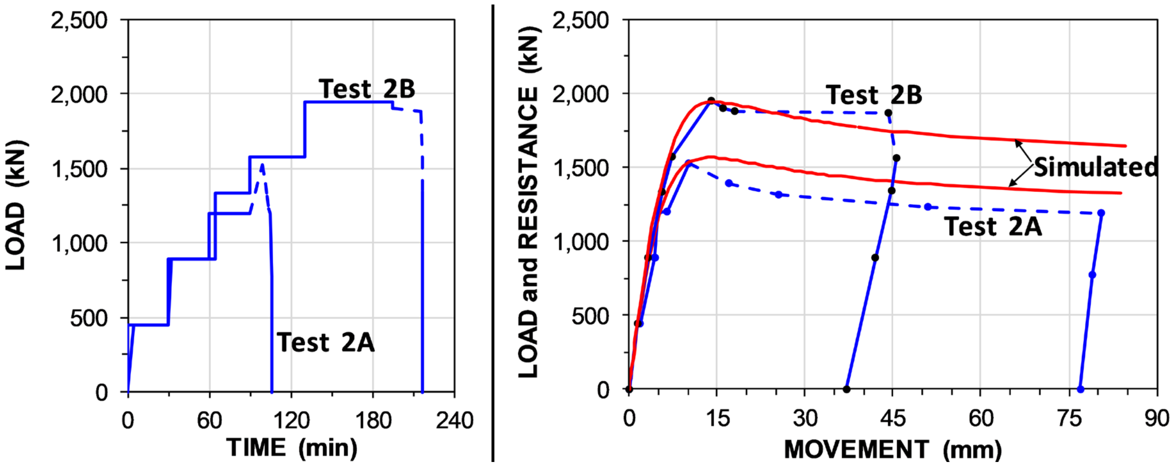

Figure 13. Load-time and load-movement curves for Tests 2A and 2B

The pile was then driven to 46.6 m depth and cleaned out as before. Thirty hours later, a static test, Test 2C, was carried out. Thereafter, a 10 m long concrete plug was cast in the pile and the pile was driven to 48.1 m depth, a further 1.5 m, to ensure toe bearing for Test 2D, carried out 3 days later. Finally, a repeat test, Test 2E, was carried out 170 days after Test 2D.

Tests 2A and 2B. Set-up for open-toe pipe pile

Figure 13 shows the test schedules and load-movement curves measured for Tests 2A and 2B. The latter showed that the nine additional days of set-up had resulted in a 25 % larger resistance. The back-analysis fit for the peak load, assuming the same beta-coefficient along the full length of the pile gave β = 0.25 at Test 2A and 0.31 at Test 2B, corresponding to unit shaft resistances increasing linearly from about 5 kPa at 5 m depth to values of about 60 kPa at the pile toe level. (The soil profile indicates no reason for not having the same beta-coefficient and t-z function for all elements).

For both tests, the testing procedure after the peak load included a few measurements after locking the jack and letting the load reduce until it stabilized, then again raising the load and letting it reduce to new "stable" level. This procedure is helpful in establishing, qualitatively, whether or not the shaft response is strain-softening, as it appears to have been for Tests 2A and 2B. "Qualitatively", because this procedure does not establish the magnitude of the softening shear resistance. The simulation was aimed to fit the short-time, large-movement, measurements using a Zhang strain-softening t-z function as shown in Figure 14. The same function was used for both tests.

Test 2B is really a second phase reloading of the pile, Therefore, it is best reported together and in sequence with Test 2A, as shown in Figure 15. There is no sign of Test 2B having been particularly affected by the prior test event, Test 2A.

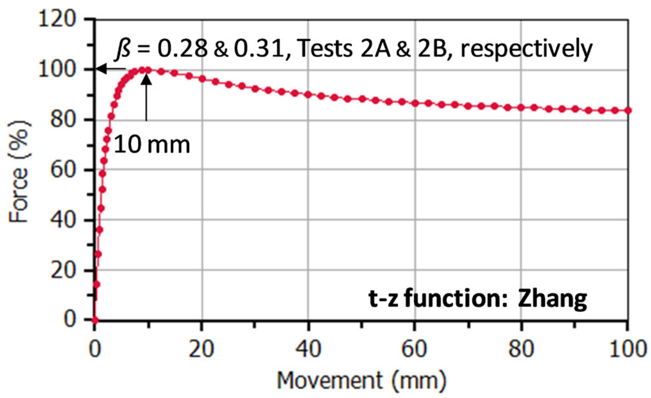

Figure 14. Zhang t-z function applied to both Tests 2A and 2B

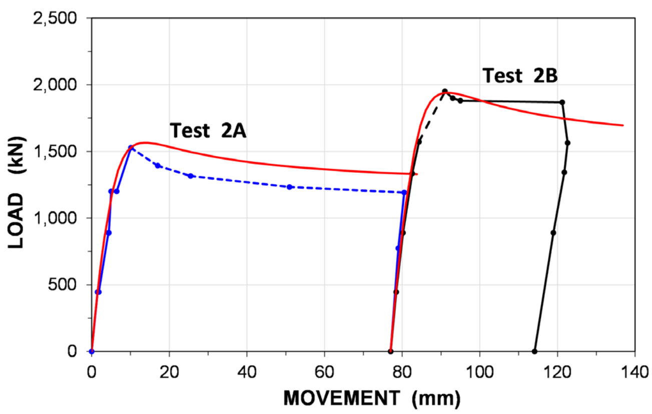

Figure 15. Tests 2A and 2B load-movements as 1st and 2nd phases of the same test

Tests 2C and 2D. Shaft resistance separated from toe resistance

Figure 16 shows the test schedules and load-movement curves measured for Tests 2C open-toe pile, tested 3 days after driving, and Test 2D, closed-toe pile after adding the

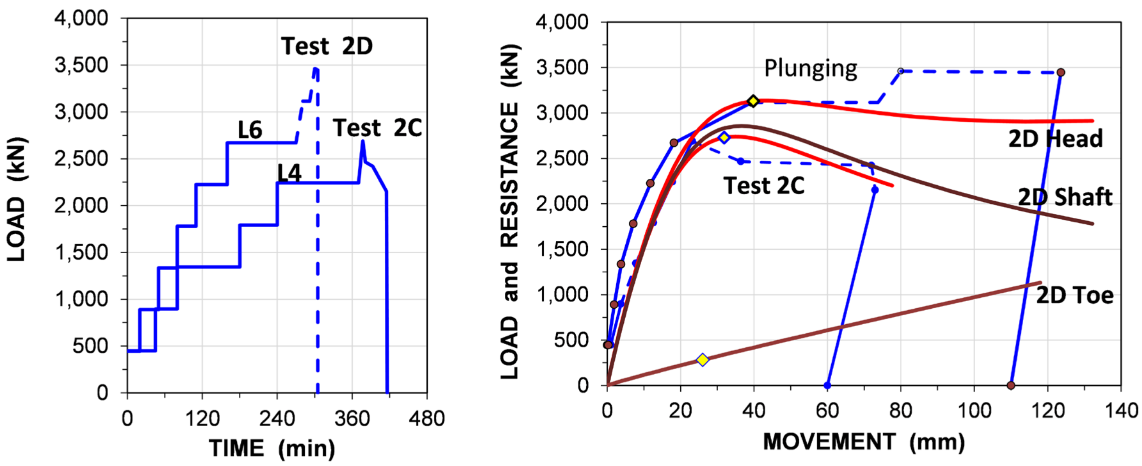

Figure 16. Load-time and load-movement curves for Tests 2C and 2D

concrete plug and driving the pile 1.5 m deeper and then waiting nine additional days. The load records at very end of the tests are disregarded as unrealistic due to the pile plunging under continued pumping to increase the load. Tests 2C and 2D are similar to Tests 1A and 1B, the tests on the shorter pile in silty sand. The figure also includes the simulated (fitted) shaft resistances curves (for both 2C and 2D) and the toe resistance for Test 2D. Because of the assumption of same shaft response, the difference in pile-head load-movement is due to the 2D toe response and a small increase of shaft resistance because the pile had been lengthened by 1.5 m.

The simulation of Test 2C (open-toe pile) indicated that the beta-coefficient that produced the fit to the three-day test to the maximum applied load was 0.18, as opposed to the values for fit to the Tests 2A and 2B: for the one-day test 0.25 and for the ten-day test 0.31, respectively. The β = 0.18 correlates to a Test 2C average unit shaft resistance of 31 kPa. The Test 2D toe resistance at the maximum applied load was 260 kN corresponding to 900 kPa toe stress.

Test 2C was carried out after the 30.5 m long test pile had been driven without displacing soil. the driving causes the shaft resistance to breakdown and the recovery (set-up) occurring without the benefit of consolidation from large pore pressures induced by displacing soil. It appears that the soil did not recover its original strength after the break-down of shaft resistance. A repeat simulation, not shown here, with input of β = 0.10 along the upper 30.5 m and β = 0.25 along the lower 10 m length, gave the same good fit to the recorded pile-head load-movement curve. The back-analysis thus suggests that the upper length having been re-driven without displacing the soil would not recover the earlier on set-up strength, as also suggested by Saye *et al*. (2020). Unfortunately, the absence of instrumentation for determining the force distribution does not allow an assured analysis of this indication.

Figure 17 shows that the Test 2C shaft input is slightly different to that developed for Test 2A inasmuch that fitting to the peak load required 25 mm movement as opposed to the 10 mm movement developed for Test 2A (c.f., Figure 13)

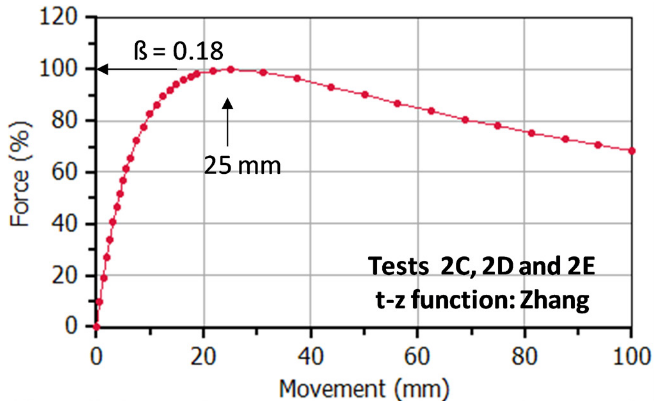

Figure 17. The Zhang t-z function assigned to the pile elements for Tests 2C, 2D, and 2E

and the softening magnitude was almost twice as large as for Test 2A.

In a sense, Test 2C is a repeat of Test 2A, but for the pile being 40.6 m long as opposed to 30.5 m. However, while Test 2A was a virgin test, Test 2C was not because, as mentioned, down to 30.5 m depth, the pile was advanced without displacing any soil. Neither is Test 2D, because it is a test performed after the lower 10 m length of the pile had been concreted and the pile driven 1.5 m deeper to build toe resistance.

Test 2E. Long-term set-up

The test pile was then left without action for 170 days, until the fifth test, Test 2E was performed. Figure 18 combines the test schedules and load-movement curves measured for Tests 2D and 2E and includes the simulated pile-head and pile-toe movements. The load movements beyond load increments L6 and L4, respectively, are short on load-holding time and excess on load. Therefore, the load-movement measurements beyond L6 and L4, respectively, are uncertain, which is why they are shown dashed. Both shaft resistance simulations use the same input of beta-coefficient and Zhang t-z shaft-element response (c.f., Figure 16). This

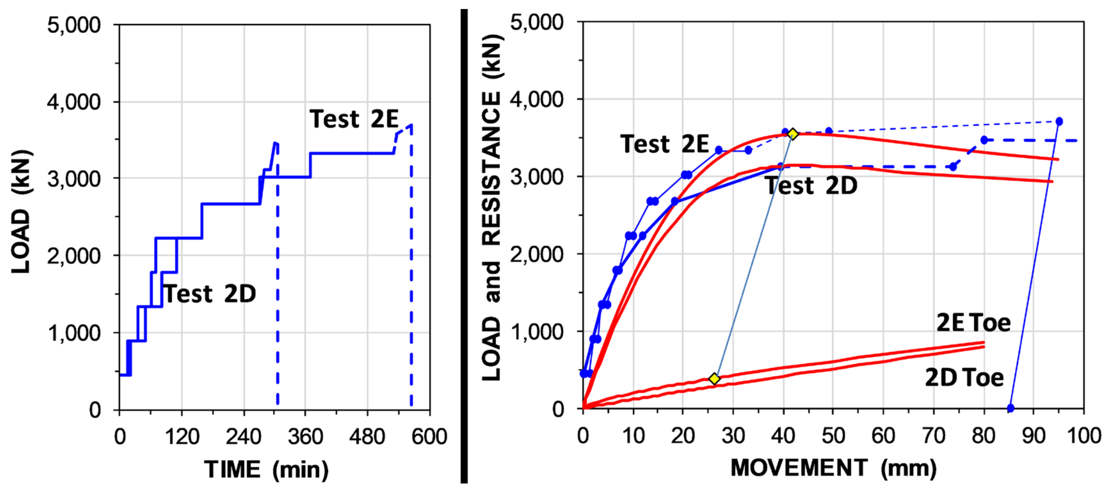

Figure 18. Load-time and load-movement curves for Tests 2D and 2E

disregards the possibility that, along the upper 30 m length (the "2C length") of the pile, the shaft resistance of Test 2D would have been affected by the pile having been driven from 30 m depth to 46 m depth after a ten-day set-up time before the test.

The 2E pile-head load-movement curve shows that the six month set-up time resulted in an increase of the maximum test load. The question is if the increase is due to increase of shaft resistance or to toe resistance. Assuming that the toe resistance has not changed, the increase corresponds to a $\beta = 0.20$ as opposed to $\beta = 0.18$ for Test 2D and average unit shaft resistances for the two tests of 33 and 36 kPa, respectively. To attempt to separate the beta-coefficients or shear resistances along the upper length not driven anew and the lower length, is not meaningful as that lower length was a mere 1.5 m. The 2E simulation required input of a slightly stiffer toe resistance than assumed for the 2D simulation (the target values were assumed to appear at a smaller movement, which is why the 2E toe-curve plots above the 2D toe-curve. This is commensurate with Test 2E toe being reloaded.

McGammon and Golder (1970) recommended that the design of the west side bridge foundations should take 19 kPa as the ultimate unit shaft resistance and an ultimate unit toe stress of 1.0 MPa.

Comments and Conclusions

The test reported by Meyerhof *et al*. (1981) was from a project that involved no subsidence concerns. Therefore, there was little need for knowing more than the total capacity for the relatively short pile, 13 m, only. However, the authors used the results for analysis of the shaft and toe response of the test pile based entirely on theoretical assessment and the process can be questioned due to the lack of confirming instrumentation results. Some corroboration to driving records would have been desirable.

The comprehensive series of tests reported by McGammon and Golder (1970) on the two relatively long pipe piles satisfied the objective of providing the capacity reference required for the design of the bridge foundations. However, the different loading schedules and variations complicate advanced back-analysis.

Because the strain-softening shaft response, the comparison between the open-toe pile to the closed-toe pile (Tests 2C and 2D), by simply subtracting the maximum load of the former from that of the latter does not provide commensurate values. Figures 16 and 18 show that the maximum shaft resistance of the open-toe pile does not mobilize at the same movement as for the closed-toe pile. Moreover, it is questionable that the shaft resistance would be equal for on open-toe and closed-toe piles. Of course, in the late 1960s, when the tests were performed, there was no simple way of instrumenting a test pile to measure the force distribution and toe response other than telltale instrumentation, which does not provide sufficient accuracy for details such as those mentioned,

McGammon and Golder (1970) analyzed the test results in terms of ultimate resistance and stress-independent shear resistance applied this to the design of the bridge foundations. The values of ultimate resistance recommended by the authors are rather conservative. It was not stated whether this was in consideration of the effect of the forthcoming increase of artesian head for the bridge foundations. No reference was made to the fact that the tests showed that there is actually no specific ultimate toe resistance. The 1.5-m diameter bridge piles would not likely show the same unit toe resistance at a specific pile head-movement interpreted as the pile capacity by a similar approach as that applied to the 600-mm test pile. In contrast, it is likely that the smaller diameter approach piles would show a similar relation between unit toe stress and toe-movement as the test pile, which, then, would serve as base for the design.

Simulation of the pile- head load-movement curve can be achieved with a range of assumptions in regard to what input to use for the t-z and q-z functions. Provided that a t-z function is seemingly reasonable, after a trial-and-error process, a fit to the pile-head curve can always be obtained by some q-z function and vice versa. However, for a representative and true back-analysis it is necessary that the test determines the axial force in the pile at or near the pile toe level, enabling separation of shaft and toe resistances, and this requires instrumenting the test pile.

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