Review of worked Examples in NCHRP Project 12-116A #112

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The US FHWA has proposed revised Specifications for Design of Piles for Downdrag related to piled foundation design (NCHRP Project 12-116A "Design of Pile for Downdrag" with Appendices C-J, Report 398), Coffman et al. 2024). The NCHRP report is available at link:

http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=4919

The NCHRP report is a proposal for the forthcoming 10th Edition of the AASHTO Specs pertaining to piled foundations. The following quotes and reanalyzes eight worked examples in Appendix C of the NCHRP report, addressing octagonal concrete piles (Examples 1 - 4), H-piles (Examples 5 and 8), and closed-toe pipe piles (Examples 6 and 7).

Example 1

Example 1 presents a case taken from Briaud and Tucker (1997), comprising a 41 m long, 420 mm (16.5 inch) face-to-face diameter octagonal concrete pile driven in clay (cross section area was 0.145 m^2). The sustained load was 1,225 kN. The pile E-modulus was 24 GPa. The soil profile consisted of two clay layers, an upper 23 m thick and a lower 19 m. The soil below the pile toe level was not mentioned, but appears to be non-compressible. Both clay layers had a bulk density of 1,970 kg/m^3. The groundwater table was at the ground surface and the pore pressure was hydrostatically distributed. The only mention of soil compressibility was in terms of a soil E-modulus indicated to be 21.5 MPa at the full depth. The Poisson's Ratio was 0.3. The undrained shear strength of the clay was indicated as 13 kPa at the ground surface, 31 kPa at the layer boundary, and 128 kPa at the pile toe level.

After the pile installed and loaded, the site became subjected to long-term settlement of the clay layers stated to amount to 320 and 17 mm at the ground surface and the layer boundary, respectively, and assumed to be linearly distributed within the layers. Whether the cause of the settlement was due to placing an embankment or general fill on the ground, to lowering of the groundwater table, or to general subsidence was not mentioned, nor was any information on initial and final stress distribution provided.

The original example (Briaud and Tucker 1997) reported a force distribution calculated from shear resistance taken as equal to the undrained shear strength times an alpha-coefficient. The so-calculated accumulated "ultimate" shaft resistance was indicated to be 1,697 kN. The toe resistance was indicated as a 528 kN toe "bearing resistance" and, this plus the shaft resistance ("ultimate") makes a 2,800 kN total. Briaud and Tucker (1997) presented a data table containing the results, quoted as Table C2 in the NHCRP report. Table C2 listed a 24 mm compression of the pile due to the axial force in the pile from the 2,225 kN sustained load to the 528 kN pile-toe force. (For comparison, the calculated compression of the 2,225-kN load on a free-standing column is 26 mm). I plotted the data in Figure 1-1, showing the resulting two intersecting force distribution curves determined per the Unified Method (Fellenius 1984; 1988). The right graph shows the plot of the pile movement, starting at 90 mm and reducing with depth due to the pile compression to a 67 mm toe movement. The latter would be the toe penetration associated with the 528 kN pile toe force. The right graph also shows the soil settlement (320 mm) at the pile head (ground surface).

The depths of the two intersections of the ultimate resistance and settlement curves differ by 2.0 m (Equilibrium Plane). This difference is likely due to the fact that the original example used ultimate resistances instead of movement-dependent response. Adjustment of either the pile-toe force or pile-toe penetration, or of both, would have made the planes to be at an equal depth.

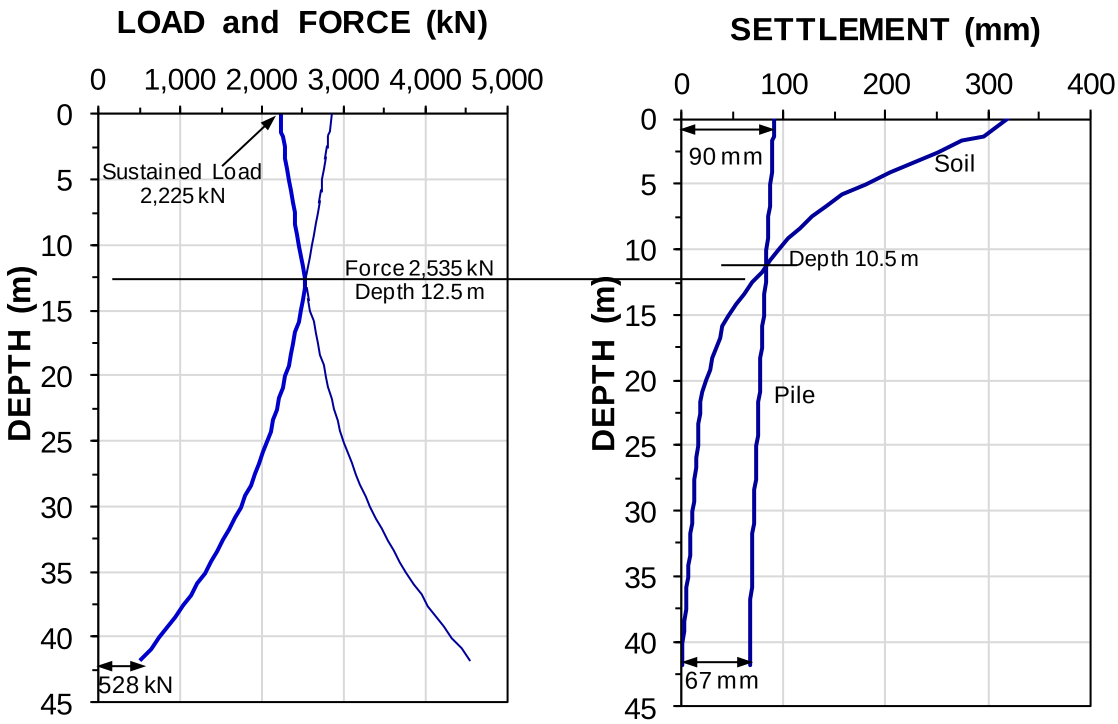

Fig. 1-1 Force and settlement distributions (plotted from Table C2)

The authors of the NCHRP Report 1112 edited the example to have the soil settlement be caused by a 6 m high embankment of soil with a density of 1,950 kg/m^3 with a given geometry placed on the ground. (Why the analyzed pile would be placed in the middle of that embankment was not indicated). The NCHRP indicates a soil settlement of 97 mm at the pile head. As to the pile ultimate shaft resistance and axial force distribution, the NCHRP Report assigned an ultimate shaft resistance of 2,653 kN, larger than before and the toe 'bearing resistance' was stated to be 168 kN. However, in the table (C6) showing the calculation results, it was stated as 290 kN. The total pile "capacity" was 2,821 kN, almost the same as in the first calculation.

The pile toe movement that was associated with the 168-kN pile-toe force was now stated to be 41 mm. The value appears to have been taken from a simulated pile-head load-movement curve for an assumed "rigid pile", calculated from the undrained strength values, employing the method by Fleming (1992). The movement for such a pile is, of course, only caused by toe penetration and, as shown in Figure 1-2, the pile-head curve becomes horizontal very early because of the assumed ultimate toe resistance (plastic) had been reached. The authors then applied the Davisson Offset limit construction to determine the pile-head movement for a "capacity" equal to the "Offset Limit" and for a non-rigid pile, as shown, and found it to be 41 mm. This was then applied as toe movement and used in plotting the right-side graph pile settlement curve. (I admit that I am rather confused by the approach. The simulated loading test is further commented on below).

The NCHRP report combined the 41-mm toe movement with the axial shortening in a renewed analysis, which resulted in a 12.1 m depth to the Settlement Equilibrium Plane as opposed to the 13.4 m depth to the Force Equilibrium Plane, but because the difference was smaller than 1.5 m, the NCHRP Report stated that it was acceptable. The two planes must be at equal depth. Their common name is Equilibrium Plane (also called Neutral Plane) The results are shown in Figure 1-3 (copied from the NCHRP report; the term "top" means "pile head"). The curves are based on assuming plastic soil response, i.e., once the shaft and toe resistances have obtained the "ultimate values", the further response is plastic, which is not often the case for shaft response and rare for the toe response.

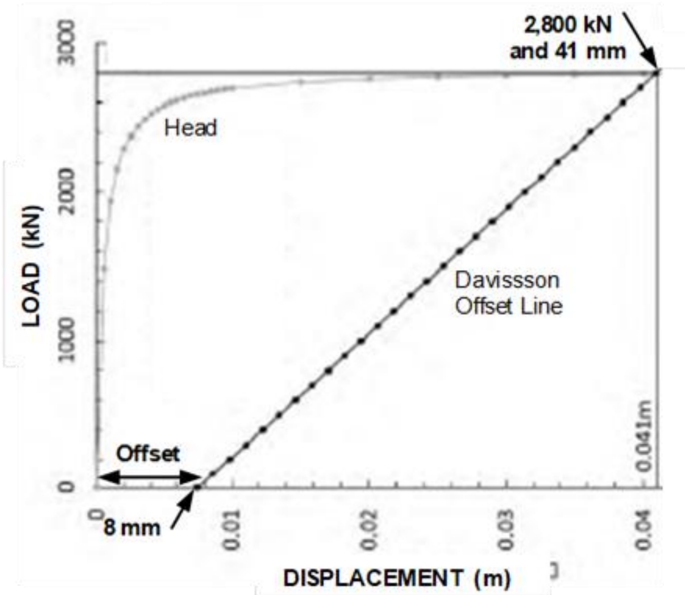

Fig. 1-2 Pile-head load-movement and Davisson Offset Line (Figure C11 in NCHRP Report 1112, 2024)

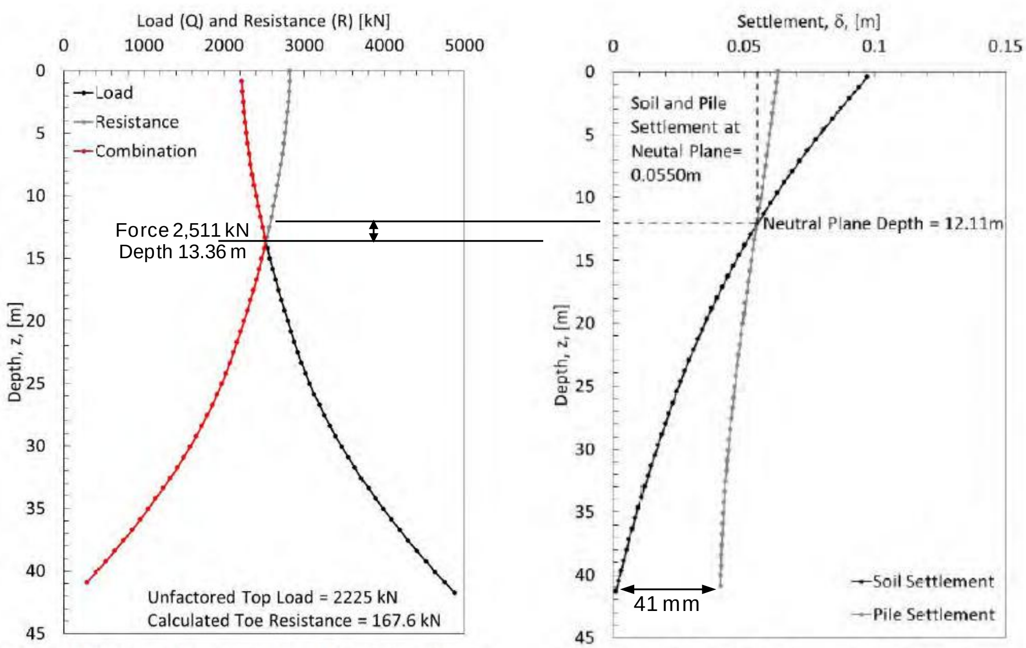

Fig. 1-3 Force and settlement distributions (Figures C7 and C12 in NCHRP Report 1112, 2024)

Example 1 disregards the fact that both shaft and toe resistances depend of the relative movement between the pile element and the soil, as expressed in equations functions called "t-z function" for the shaft resistance and "q-z function" for the toe resistance (Fellenius 2018; 2024). The functions depend on the soil type, soil conditions, pile construction, and much more and the resistance increases for increasing movement. Usually, the shaft resistance increase is drastic for the first few millimetres and, then continues to increase at a smaller rate, only sometimes becoming plastic and, in very soft clay, maybe, even softening. In contrast, the increase of the toe resistance is gentler and it does not show a plastic response. This means that there is no such thing as an ultimate toe resistance, but for some very rare and special condition. In short, the shaft and toe resistances are governed by movement and the original Unified Method has long since been updated to incorporate the influence of movement.

The following presents a UniPile6 (UnisoftGS 2024) analysis of the Example 1 case with the movement response incorporated. In order to stay close to the assumption of plastic shaft resistance indirectly assumed in the NCHRP analysis, the t-z function (the same function was used for the full 41-m pile length) was assumed to follow a hyperbolic response illustrated in Figure 1-4. The shaft resistance was not chosen to be stress-independent, but was set to be a beta-coefficient, that is, analysis was by effective stress. The function coefficient was 0.90, meaning that the shear resistance at the 5-mm target movement was 90 % of the resistance at infinitely large movement—the target value is indicated as "100 %" in the figure. The beta-coefficients achieving the fit to the indicated ultimate shaft resistance distribution ranged from ß = 0.13 through 0.21. The target ß-coefficients, therefore, were input as 90% of the coefficients found for the NCHRP "ultimate" distribution.

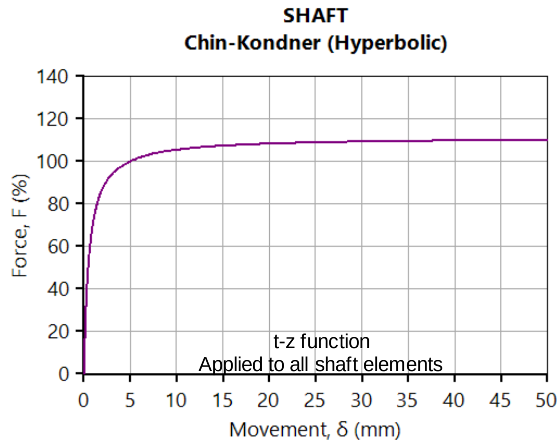

Fig. 1-4 The applied t-z function

The q-z function was chosen as a Gwizdala function with a function coefficient of 0.7 as shown in Figure 1-5. The toe-force value of 168 kN and associated 41-mm pile-head movement was accepted and according to the chosen q-z function, at a 5-mm target toe movement, the toe force will be 100 % of the target, input as 35 kN. (The force-movement relation shown in Figure 1-1 of the NCHRP report does not fit this function).

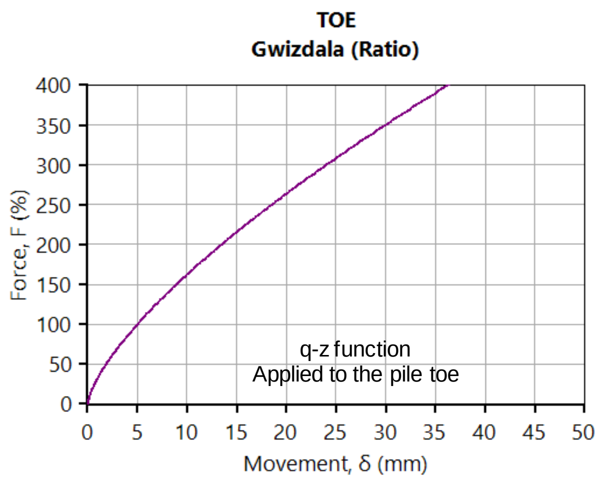

Fig. 1-5 The applied q-z function

The placing of the embankment added stress to the soil and this affected the shear resistance in the effective tress analysis as shown in Figure 1-6. Because the t-z function governs an almost plastic response with increasing movement, little difference appears between the initial and final distributions of unit shaft resistance, r_s. (The $r_{s,\ final}$ listed in Table C6 of the report was fitted to the ultimate resistance of the report). In a soil exhibiting a different shaft response, whether strain-hardening or strain-softening, the difference between the initial and final distribution could be considerably larger.

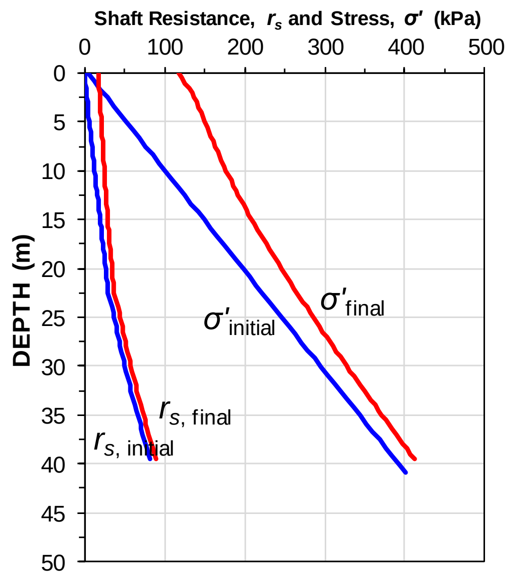

Fig. 1-6 Distributions of initial and final effective stress, σ', and initial and final shaft resistance, r_s

Similarly, to distribution of the beta-coefficient chosen to fit the force distribution, the distribution of soil compressibility was chosen to result in a soil settlement distribution that fitted the one stated due to the embankment. Figure 1-7 shows the results of analysis using UniPile6 with input of the soil parameters fitted to the example. The pile compression has been adjusted for the pile compression due to the sustained load before the soil settled. As the soil input is the same and the unit shaft resistance (t-z) function is almost plastic, the NCHRP example and the UniPile6 output are quite similar, but for the toe movement. The latter could be obtained by a small adjustment to the t-z/q-z function coefficients, which, however, would merely be for cosmetic reason.

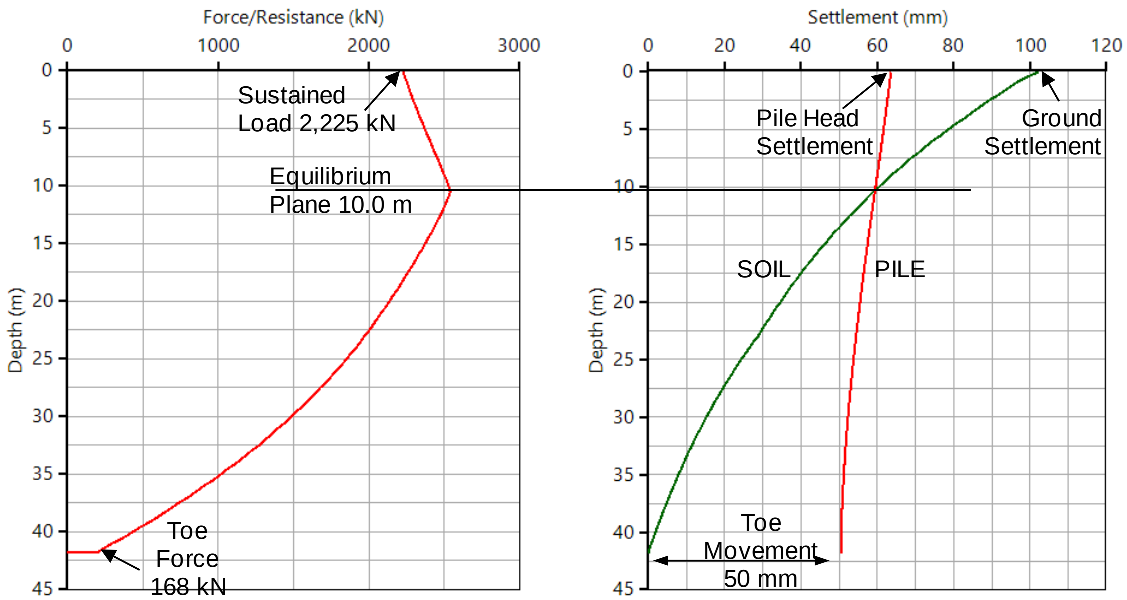

Fig. 1-7 Example 1 Unified Method results calculated using UniPile6

The UniPile6 software includes the option of applying the input to output a static loading test simulation. Figure 1-8 shows the results in red curves together with the clearly unrealistic load-movement curves in blue (as extracted from the report). The load-movement curve in the NCHRP report assumes a rigid response (excludes the pile compression) and applies a stiffer shape of the assumed hyperbolic shaft resistance response and, also, a hyperbolic response of the toe resistance as opposed to the strain-hardening response typical for toe resistance shown in Figure 1-5.

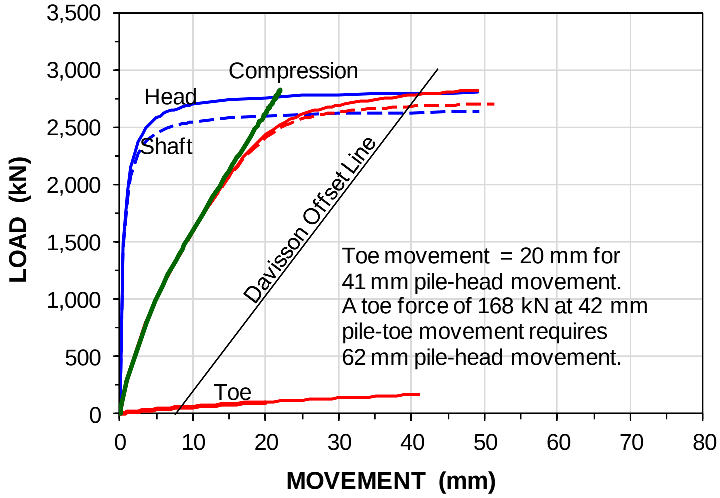

Fig. 1-8 The results of a simulated static loading test as shown in Appendix C of the report (Fig. C11–blue curves) and the results calculated using UniPile6 (red curves)

Example 2

The NCHRP Example 2 applies the same soil profile input and embankment load as Example 1. The NCHRP Example 1 was a "hand calculation". Example 2 uses a dedicated software, Pile AXL by Innovative Geotechnics 2023) for the analysis and arrives at the same result as shown in Figure 1-3. Figures D15 and D16 in the NCHRP Report are identical to Figures C7 and C12.

Example 3

The NCHRP Example 3 uses the same soil profile as used for Examples 1 and 2 and special software for the analysis, "Shansep" (Ladd and Foott 1974). The results (Figure 3-1) are similar to the previous examples. The difference between the equilibrium depths is slightly larger, though. As for Examples 1 and 2, the toe force was 168 kN and the toe movement for that force was 41 mm. Still the force and settlement curves intersections are obtained at different depths, 14.2 and 11.2 m, respectively, which is a significant results error.

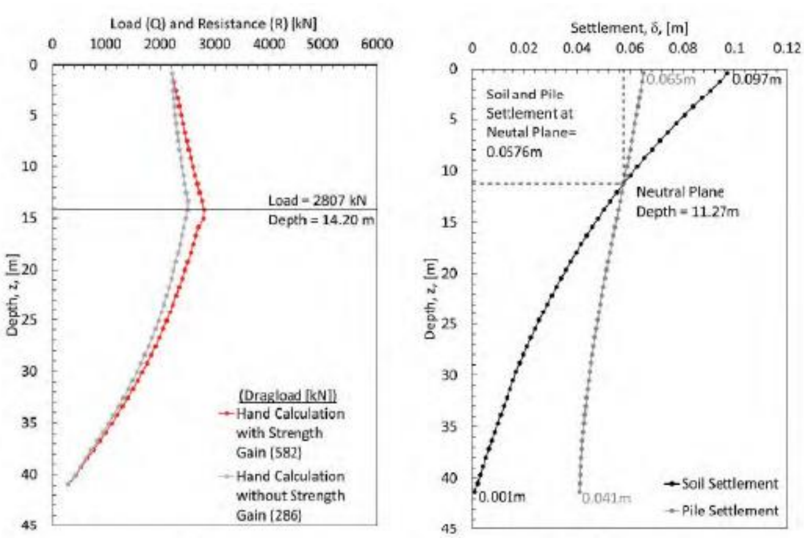

Fig. 3-1 Example 3 Force and settlement distributions (Fig. E6 in NCHRP Report 1112, 2024) with and without including soil strength gain

Example 4

The NCHRP Example 4 also applies the soil profile used for Examples 1 - 3, but replaces the stress increase due to the embankment load with that due to lowering the groundwater table to 6.0 m depth, assuming that the pore pressure will also stay hydrostatically distributed. Example 4 uses the software TZPile (Ensoft 2021). Figure 4-1 shows the resulting force and settlement curves. (The TZPile imports settlement calculated outside the software). In contrast to the results of the previous three analyses, the force and settlement curves intersections are at the same depth. As for Examples 1 - 3, the toe force was 168 kN and the toe movement for that force was 44 mm.

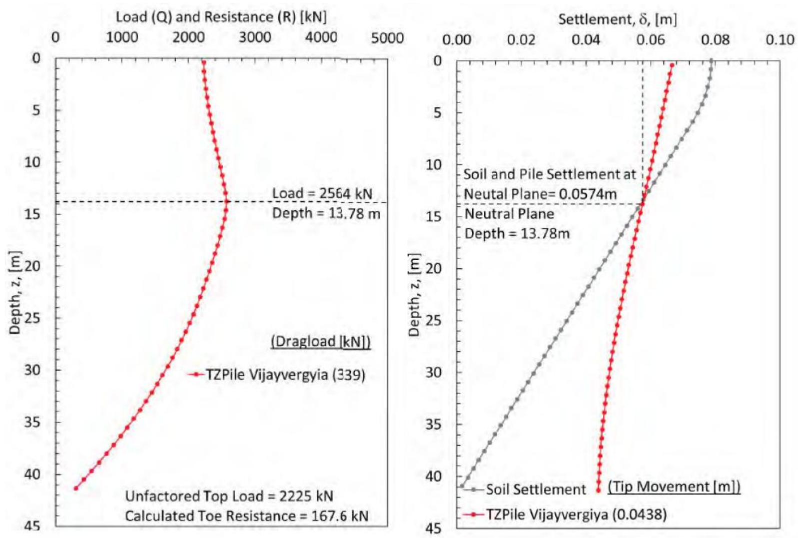

Fig. 4-1 Force and settlement distributions (Fig. F11 in NCHRP Report 1112, 2024)

A UniPile6 analysis for the same soil input as used for Example 1 was used for Example 4, as before, the settlement was calculated inside the UniPile6 employing the soil input. The pore pressure applied lowering of the groundwater table instead of placing an embankment. Figure 4-2 shows the results per the Unified Method calculated using UniPile6. The analysis applied the t-z/q-z functions that were used for Example 1 (Figures 1-4 and 1-5). Note, both shaft and toe resistances are, therefore, movement dependent. The pile compression included adjustment for the compression due to the sustained load prior to the lowering of the groundwater table. The TZPile and UniPile6 delivered practically identical results per the Unified Method. Note, the T-ZPile does not include calculating the soil settlement, but has to import values determined by other means.

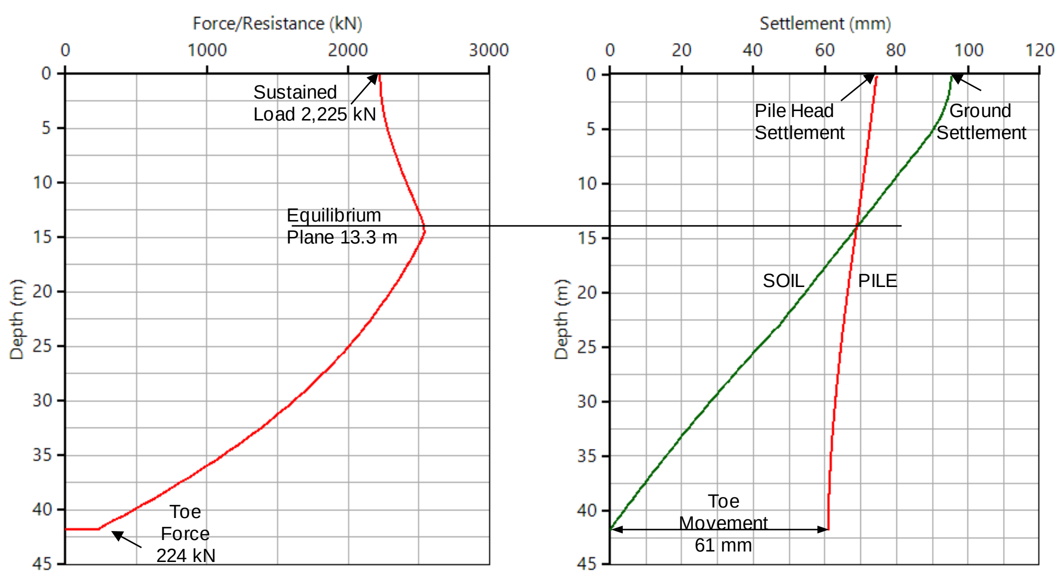

Fig. 4-2 Example 4 Unified Method results calculated using UniPile6

Figure 4-3 shows the static loading-test shown in the report (blue curve) as simulated by the software TZPile together with the results obtained by UniPile6. The dashed red curve shows the results for assuming that plastic-responding shaft and toe resistances after a 5 mm initial movement to confirm to the assumptions made for the NCHRP examples. The difference is primarily due to that the NCHRP report applies a hyperbolic toe response and an early appearing ultimate toe resistance. The UniPile simulation applies the more realistic strain-hardening pile-toe response (Figure 1-5). The area between the two pile-head load-movement curves is approximately the area difference between an early (small movement) appearing ultimate toe resistance and that of a more gradually developing toe resistance.

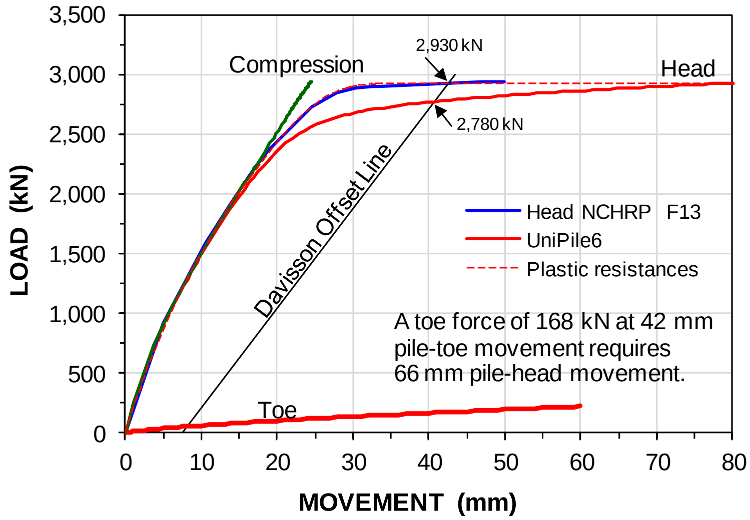

Fig. 4-3 The results of a simulated static loading test on the octagonal pile as shown in the NCHRP report (Fig. F13–blue curves) and the results calculated using UniPile6 (red curves)

Example 5

The NCHRP Example 5 presents a case addressing seismic liquefaction analysis employing CPT records for determining ultimate pile response and ground settlement. The first 25 pages of the CPT appendix are incidental to the issue of downdrag and drag force. The example pile, then introduced, is a 360HP152 (14HP102) driven to 26.7 m depth through a soil comprising 2.6 m of clay over sand. The CPTU sounding made at the site (Blytheville, AR) are shown in Figure 5-1 with a soil profile after Robertson (1990).

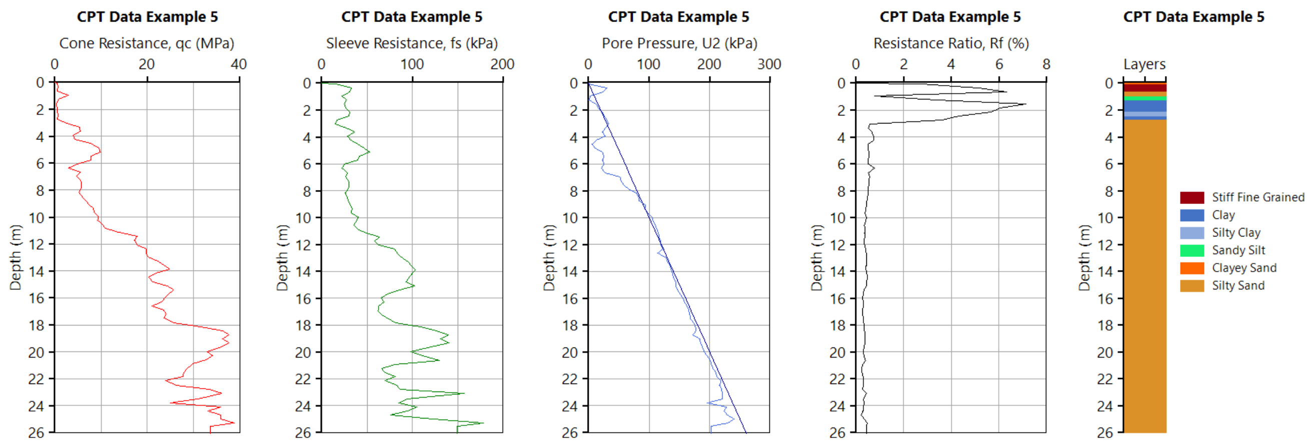

Fig. 5-1 CPTU-diagram and soil profile classification produced by UniPile6 from Table GO.

The sand below the clay is liquefiable and in the event of an earthquake, the upper about 11 m of the soil profile will compress. The NCHRP report used the CPT sounding records to analyze the soil for liquefaction risk and to calculate the volume loss (compression) and settlement from the accumulated compression due to liquefaction. Figure 5-2 shows the distribution of settlement and the q_c-diagram. The red-dotted curve show calculated settlement from the NCHRP report (Fig. M2). The black solid line is average settlement distribution, and the red solid curve shows the settlement calculated by UniPile6 for soil compressibility distribution fitted to the settlement distribution given in the report as calculated due to the seismic liquefaction.

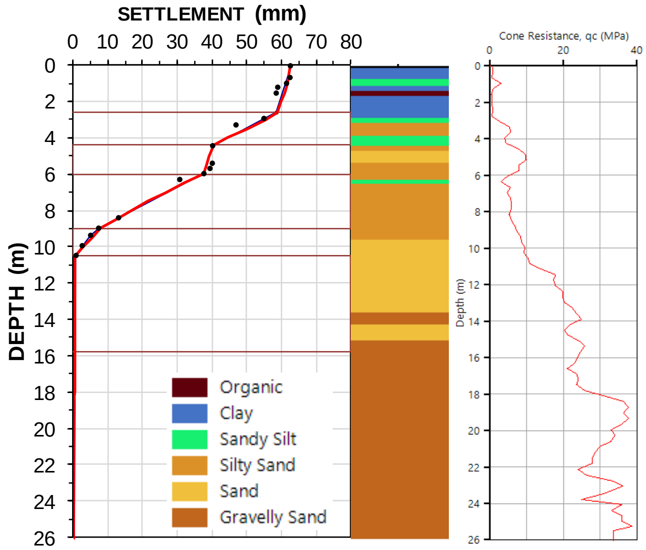

Fig. 5-2 Calculated settlement distribution from Fig. M2 and UniPile6

The NCHRP report used the CPT records to calculate also the pile capacity and resistance distribution according to the LCPC 1982 method employing the ALCOTT software. Figure 5-3 shows the force distributions presented in the NCHRP report (solid line) and calculated using UniPile6 (dashed line) applying the LCPC 1982 method. The two calculations results agree well. The CPTU-determined resistance represents ultimate conditions and indicate that the ultimate resistance of the pile was 3,340 kN, ample for the 950 kN sustained load. Note, all CPT-related methods assume that the results pertain to ultimate resistance along the pile and the common assumption is that the soil response is plastic also at the pile toe—after a small initial movement. The figures shows the distribution curves plotted both downward from the applied load and upward from the calculated toe resistance. However, this does not mean to suggest that the CPT-calculation delivers the curves per the Unified Method. That requires adjustment of the force to movement.

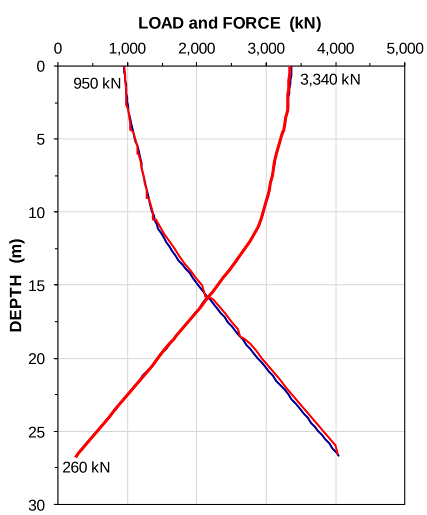

Fig. 5-3 Distribution for ultimate resistance calculated per the LCPC 1982 method

The NCHRP report simulated a static loading test on the Example 5 pile by applying the Fleming (1992) method for a rigid pile. Figure 5-4 shows the simulated pile-head load-movement curve. As was the case for Example 4 (Figure 4-3), the figure includes the UniPile6 simulation of the case (red curves), again applying the t-z/q-z functions of Figures 1-4 and 1-5. The report obviously applies an ultimate pile resistance larger than that determined for the case from the CPT records and the report simulation (blue curve) does not seem relevant to the example.

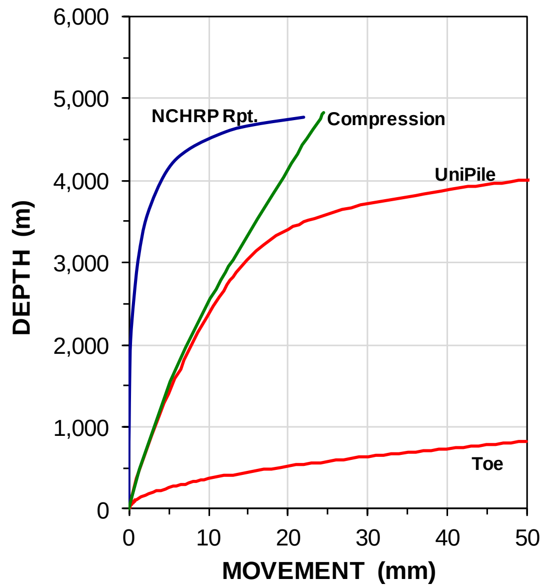

Fig. 5-4 Load-movement curves from simulated head-down static loading tests

Figure 5-5 shows the NCHRP report's compilation of the force and settlement distributions. The force distribution is plotted from the "ultimate" resistance determined by the LCPC 1982 method. The settlement distribution is that obtained from assessment of the liquefaction effect using the CPT-records. (0.238 inch = 6.0 mm, 32.3 ft = 9.8 m, and 39.4 tons = 350 kN).

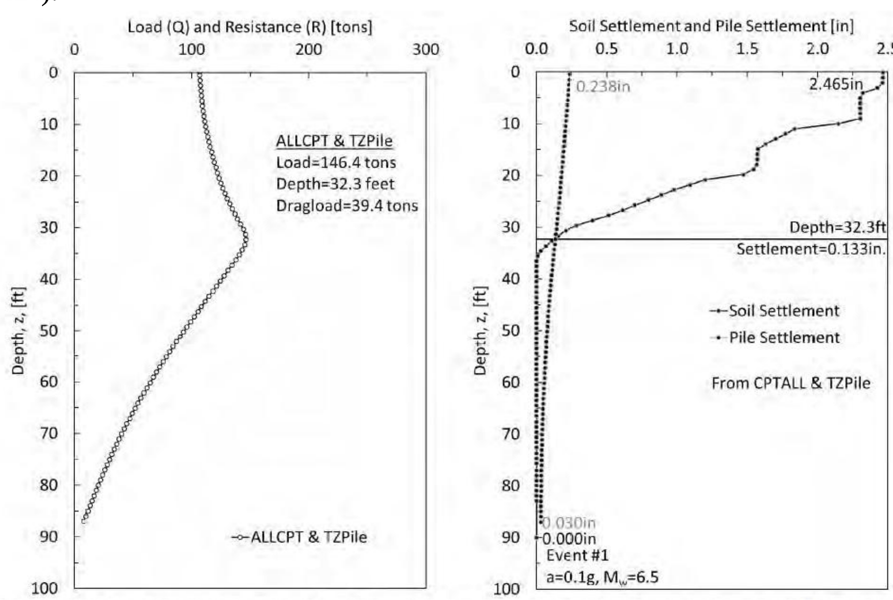

Fig. 5-5 Force and settlement distributions per the NCHRP report Figures G15 and G26

Figure 5-6 shows the results of the UniPile6 analysis per the Unified Method for Example 5. The 10.0-m depth to the Equilibrium Plane and the magnitude of the drag force are intentionally directed to be similar to those in the NCHRP report. The LCPC 1982 method was applied to the calculations.

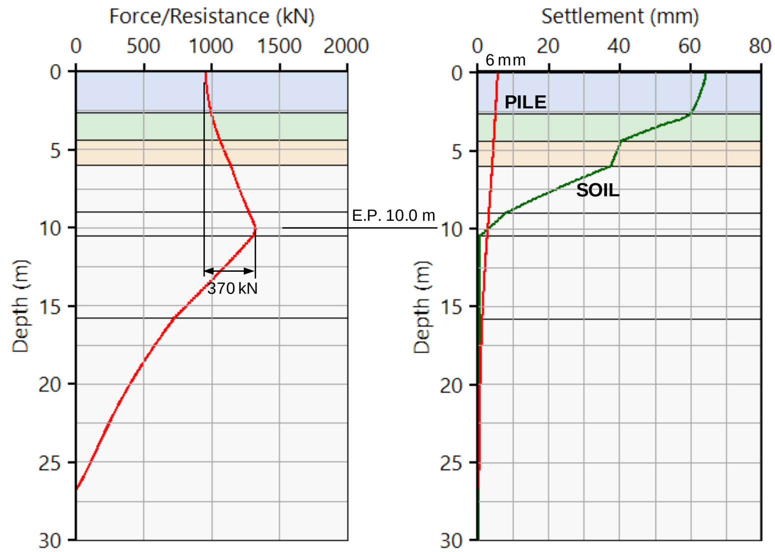

Fig. 5-6 Force and settlement distributions per the Unified Method (UniPile6)

The NCHRP report does not state why the LCPC 1982 was used instead of its update, the LCPC 2012, or some of the other many methods available. Figure 5-7 shows the ultimate force distributions for six CPT-methods available to the profession incorporated as options in UniPile6. (For details on the methods, see Fellenius 2024). The curves differ significantly from each other. It is obvious that in a real case, before relying on one or the other, they choice of method will require corroboration to the particular site and pile.

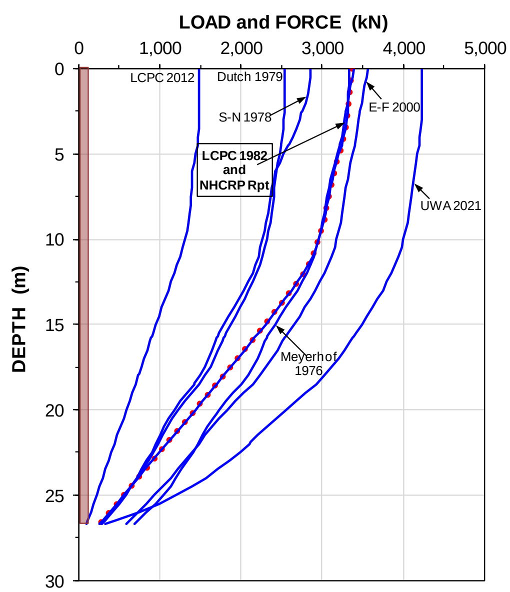

Fig. 5-7 Compilation of ultimate resistance calculated by six CPT-methods applied to Example 5

All CPT-methods are based on statistical correlations to actual static loading tests collected in a data base applying one definition of or other to determine the "capacity" of the test pile. Many of the tests in the particular data base have poor separation of shaft and toe resistances. The one definite fact about the CPT results and a pile is that where the CPT shows more resistance at a site than at another site, the pile response at the former will be stiffer than at the latter. However, before the calculation of pile-force distribution calculated by a CPT analysis is relied on, it needs to be "calibrated" to the particular conditions (soil and pile) at the site.

Pile force distribution is proportional to effective stress and the most representative analysis result is obtained using effective stress, so-called beta-analysis (as opposed to a stress-independent analysis). The conversion of the force distribution determined from a CPT-method by correlation to the effective stress distribution provides the beta-distribution shown in Figure 5-8 of the CPT-results shown in preceding figure. A designer can assess the beta-distributions for a specific pile, site, and CPT-method and choose the input to an effective stress calculation in assessing performance of a pile, also combine the stress independent method (α-method) with the effective-stress method (ß-method) as desired for the chosen soil layers.

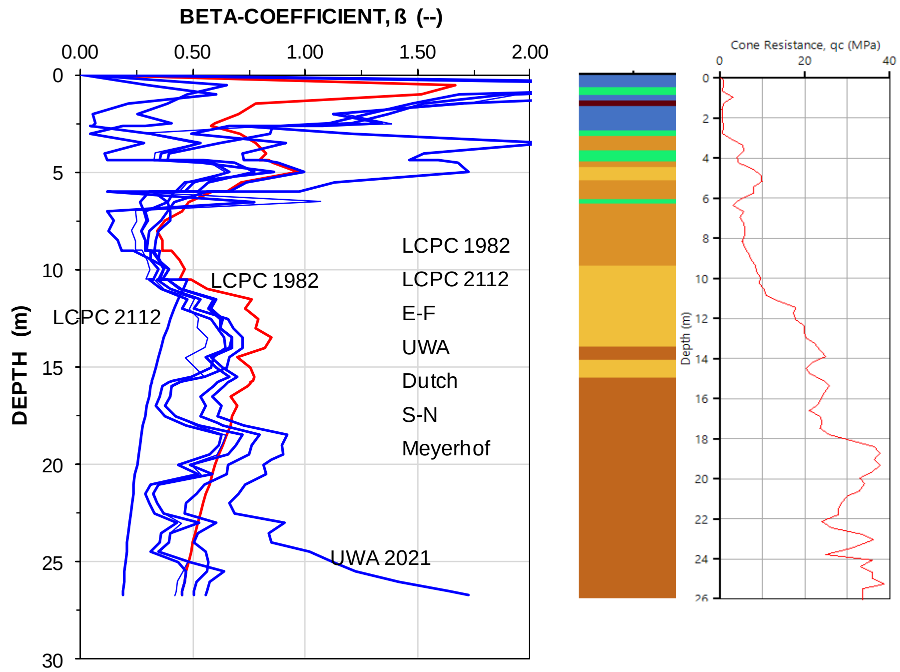

Fig. 5-8 Distributions of beta-coefficients calculated by six CPT-methods applied to Example 5

Example 6

The NCHRP Example 6 presents an analysis of a pipe pile in the same soil profile and CPTU-sounding as that for the previous examples. The pile is a 457 mm (18-inch) concrete-filled, pipe-pile with a 12.5 mm wall installed to 21.6 m depth. Two scenarios are included: Event #1, which is the same seismic liquefaction event as in Example 5 and Event #2 for which the earthquake was stronger. As before, a UniPile6 calculation was performed for the same input data (CPT records and using LCPC1982 for determining the ultimate resistances and with the same distribution of liquefaction-induced settlement). Figure 6-1 shows the distribution of the axial force (ultimate resistance) according to the LCPC1982 for the pipe pile.

The settlement distributions for the two liquefaction events as given in the NCHRP report are combined in Figure 6-2.

The force and settlement distribution curves given the NCHRP report for Event #1 are shown in Figure 6-3. The data are from Figures H18 and H19 as produced by TZPile from the ALLCPT resistance distribution

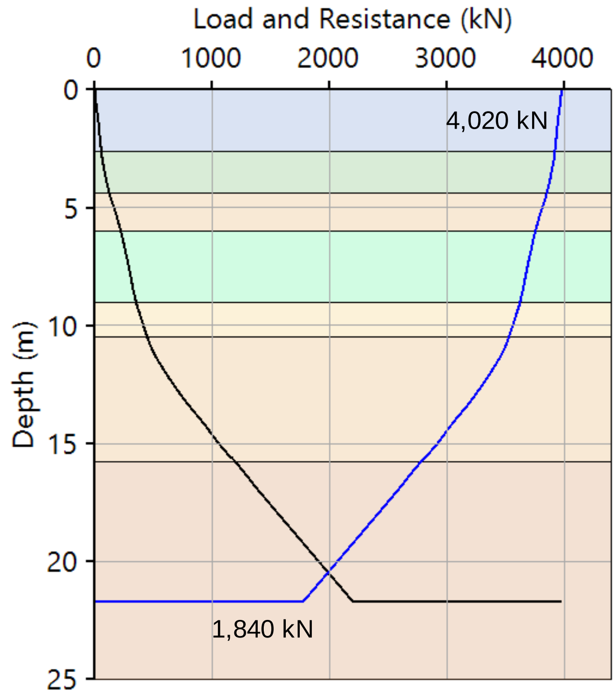

Fig. 6-1 Distributions of ultimate resistance according to the LCPC1982 method calculated in UniPile6

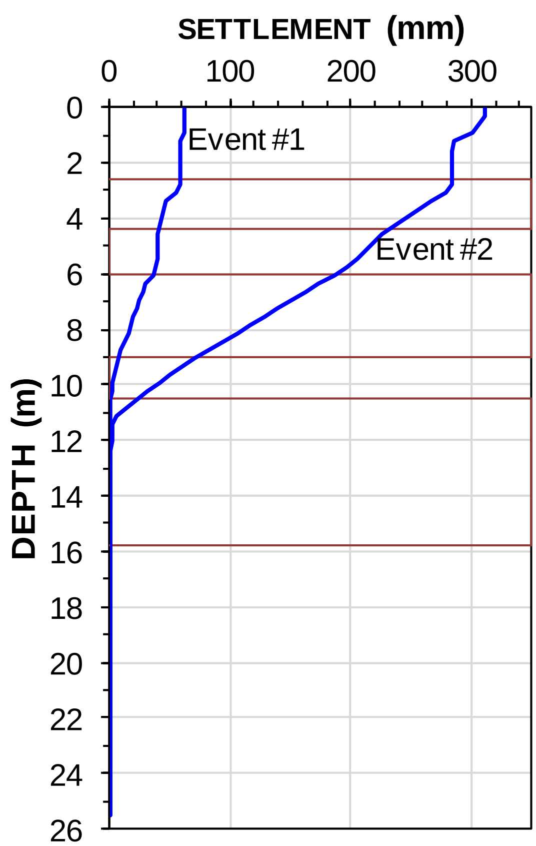

Fig. 6-2 Distributions of calculated liquefaction settlement

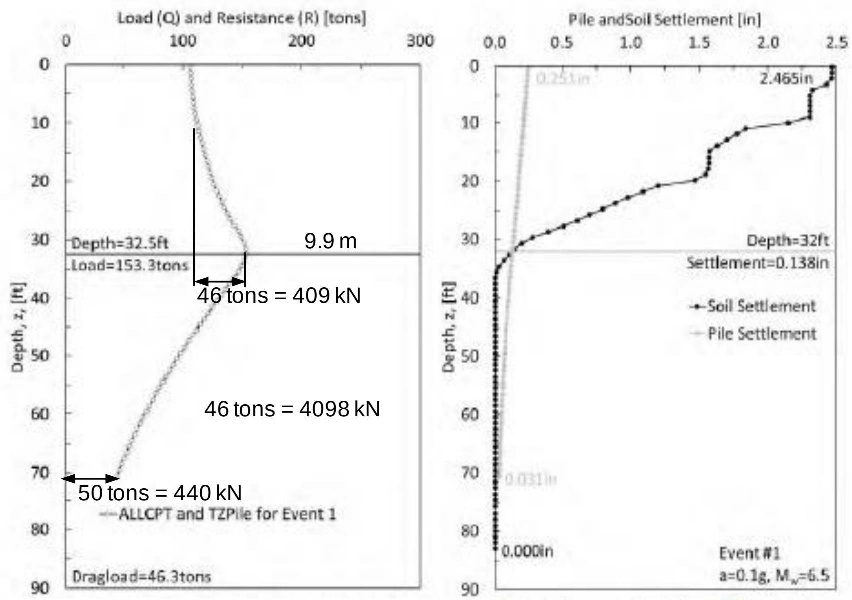

Fig. 6-3 Event #1 Force and settlement distributions per the NCHRP report Figures H18 and H24

However, it is not likely that a sufficient magnitude of the maximum axial force at the 9.9 m depth would reach the pile toe and the pile-toe movement, therefore, would be smaller than indicated. The 440 kN toe force in Figure 6-2 (report figure, H18) is not realistic and it is not compatible with the 0.031-inch toe movement (report figure H24).

Figure 6-4 shows the results of a Unified Method calculation for the pile employing UniPile6. Note the more realistic toe force; compatible with the minimal toe movement.

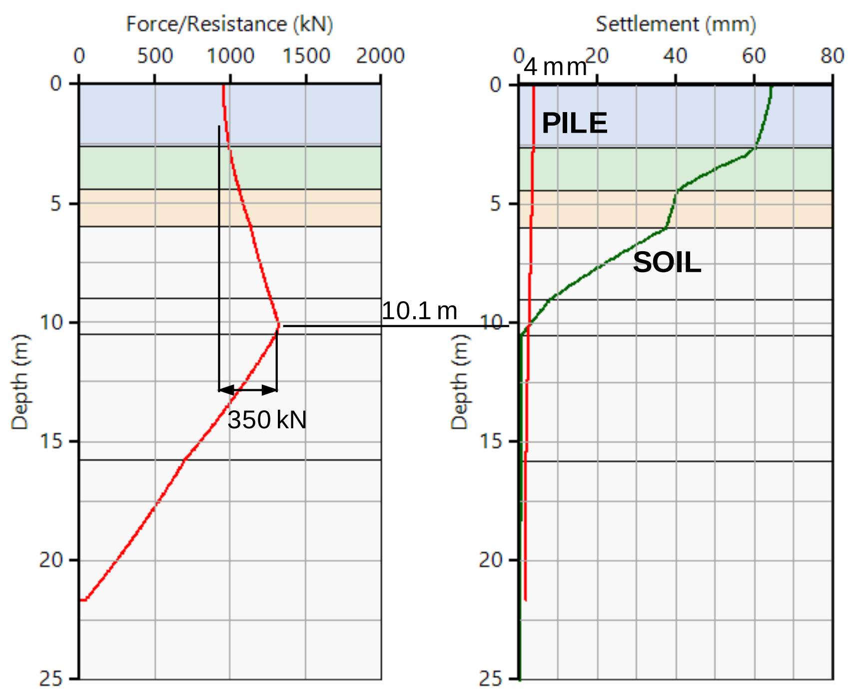

Fig. 6-4 Event #1 Force and settlement distributions per the Unified Method (UniPile6)

Figures 6-5 and 6-6 show the similar compilations for Event #2 as given in the NCHRP report and by UniPile6, respectively. Again, the report TZPile calculation shows a significant toe force that is not compatible with the minimal toe movement, while the toe response calculated by UniPile6 is realistic. Example 6 can be said to show a case for where neither downdrag nor drag force would be of concern for the pile foundation.

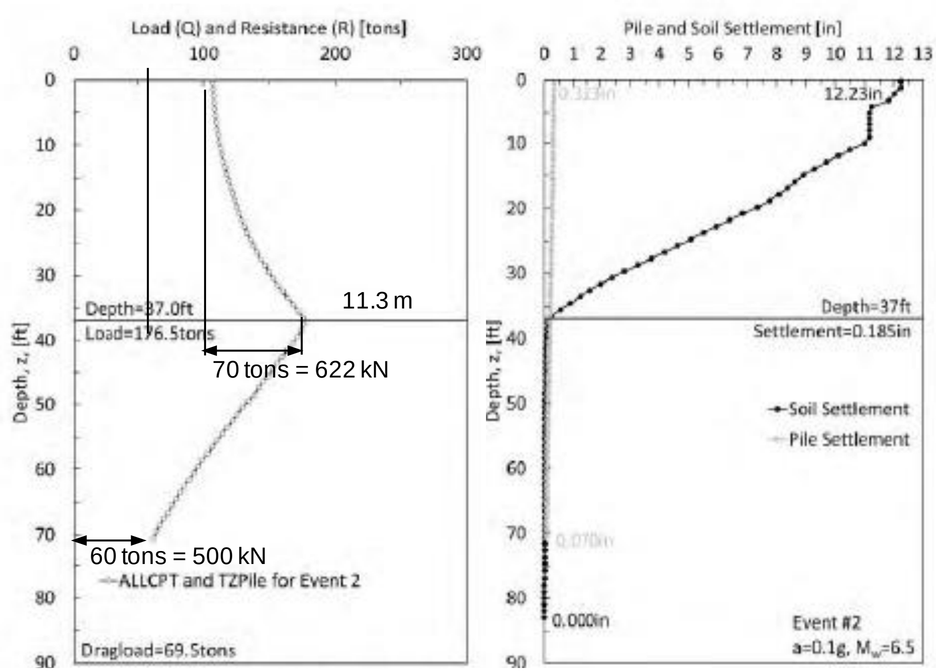

Fig. 6-5 Event #2 Force and settlement distributions per the NCHRP report Figures H19 and H25

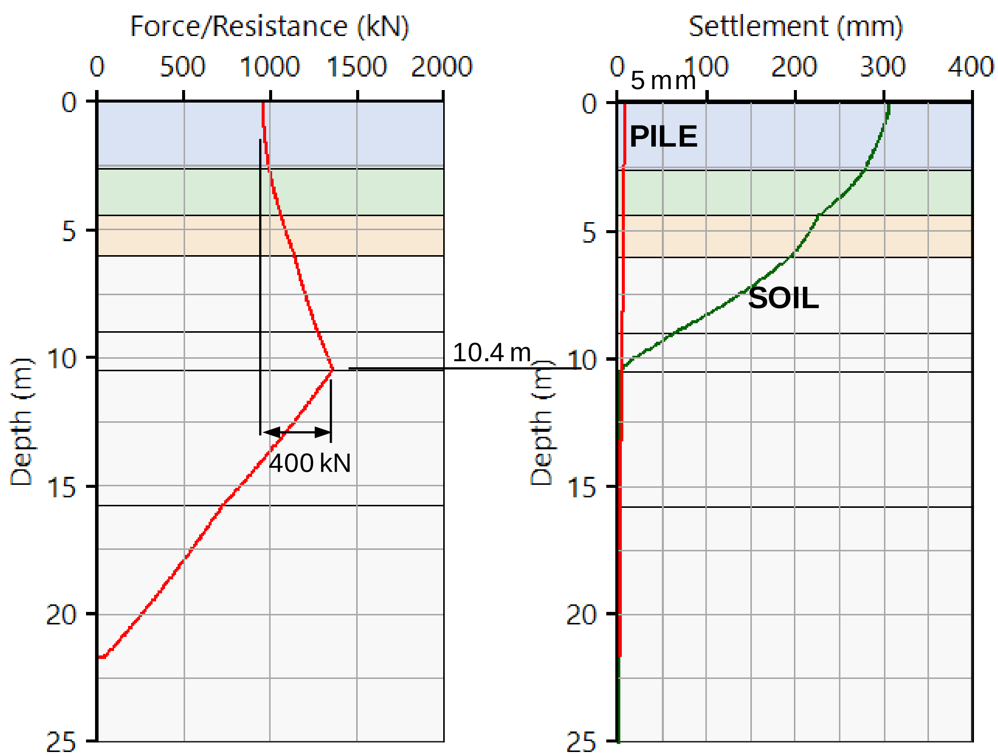

Fig. 6-6 Event #2 Force and settlement distributions per the Unified Method (UniPile6)

The NCHRP report Appendix H also presents the results of a simulated static head-down loading test. UniPile6 was used to simulate the test applying the same t-z/q-z function reported earlier (Figures 1-3 and 1-4). The results are compared in Figure 6-7. The NCHRP report does not state what functions the report analysis might have used. However, they are obviously much stiffer than those used by UniPile6. If the latter analysis would be repeated for a new choice of t-z/q-z functions, a better agreement would of course be obtained. However, the report curve would always be stiffer than any repeated analysis, because the line stated to be the 'Davisson Offset Line' is too steep in report analysis and appears to have input an incorrect pile axial stiffness. The slope should be that of the elastic line. For example, a 21.6 m long, free-standing concrete-filled, 457-mm pipe pile subjected to a 4,000-kN load would compress about 10 mm and about 8 mm in a static loading test on an embedded pile.

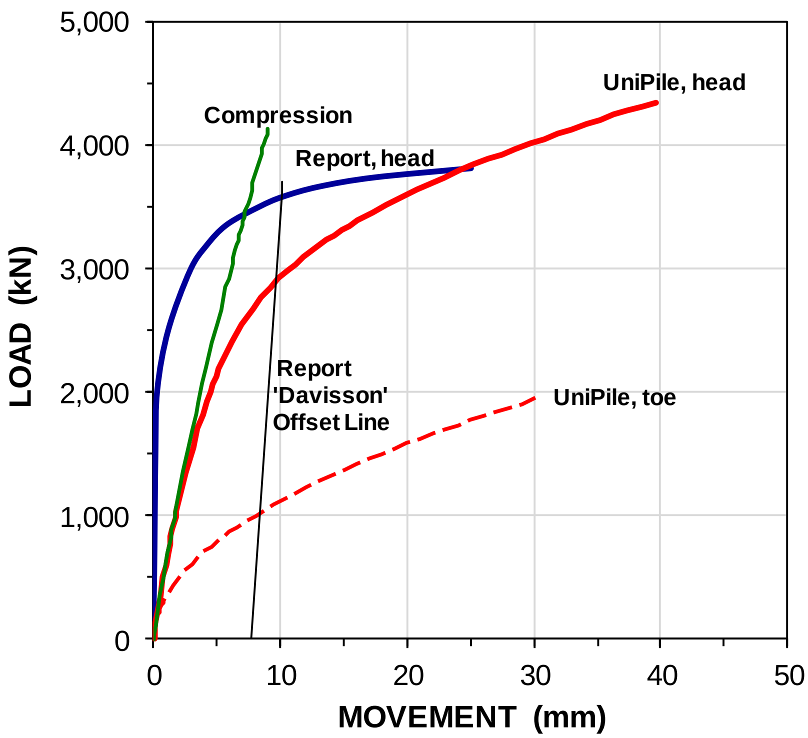

Fig. 6-7 Event #1 Load-movement curves from UniPile6 simulated head-down static loading test

Example 7

The NCHRP Example No 7 comprises two piles, which both are of the same type and size as Example 6 pile, but longer, 124 ft and 136 ft. The issue is also liquefaction, but now the liquefaction is deep-seated, mostly occurring below 124 ft depth. The report states that the results are presented in "Imperial units", i.e., customary US units, and I have conformed to this, but for my not using both tons and kips, preferring to apply a single force unit.

Figure 7-1 shows the settlement distribution reported by NCHRP. The UniPile6 software was used to simulate the same settlement by fitting soil layer compressibilities to obtain the report values and the figure includes also the UniPile6 settlement distribution output.

The report gives the ultimate resistance distribution of the 124-ft pile, here shown in Figure 7-2. The similar data for the 136-ft pile are not included in the NCHRP report, but it appears that the same shaft and toe resistances were assumed also for the longer pile. The red curves are from UniPile6 output for both piles.

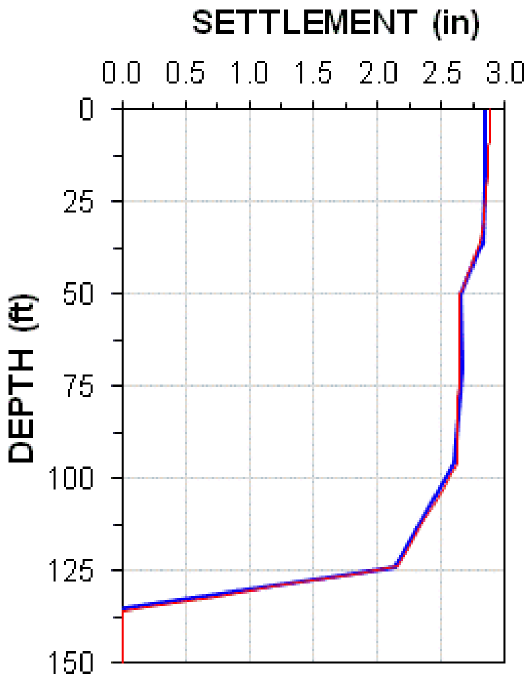

Fig. 7-1 Settlement distribution after liquefaction

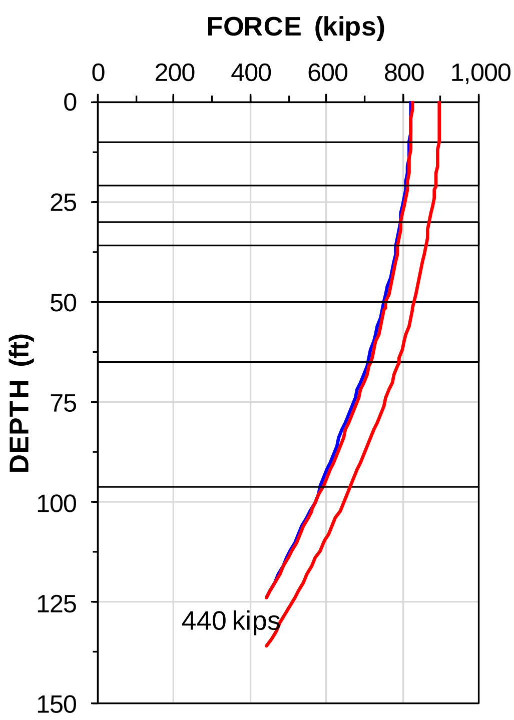

Fig. 7-2 Distribution of ultimate resistance for the 124-ft and the 136-ft piles
per the NCHRP report Table I8 and Figure I13

Figure 7.3 shows the results of a simulated static loading test on the 136-ft pile. The 746-kip ultimate resistance (by the Davisson Offset Limit method) is approximately the same as indicated in Figure 7-2.

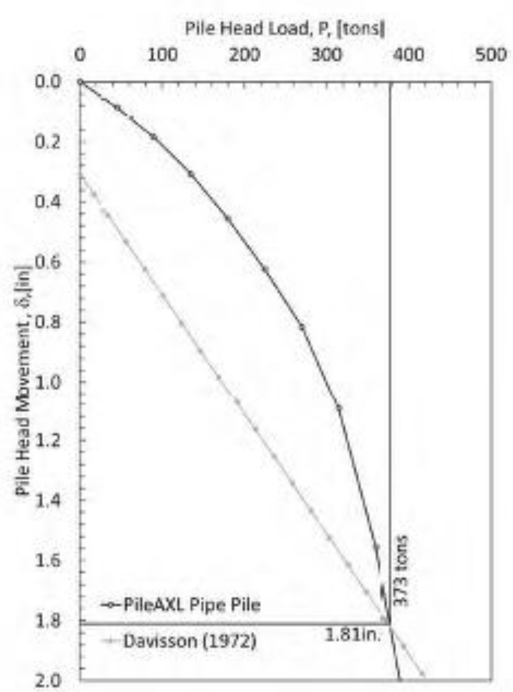

Fig. 7.3 Load-movement from a simulated static loading test on the 136-ft pile per the NCHRP report Table I10 and Figure I18

As for the previous examples, the NCHRP report used two different software to calculate the effect on axial pile force and pile settlement. Only the TZPile software gave reasonable results and only the results for the 136-ft pile are in the report. Figure 7-4 shows the distributions of force and settlement for the 136-ft pile. The toe resistance is indicated as 440 kips occurring for a 50-mm toe movement. The shaft resistance is somewhat smaller than the stated ultimate shaft resistance.

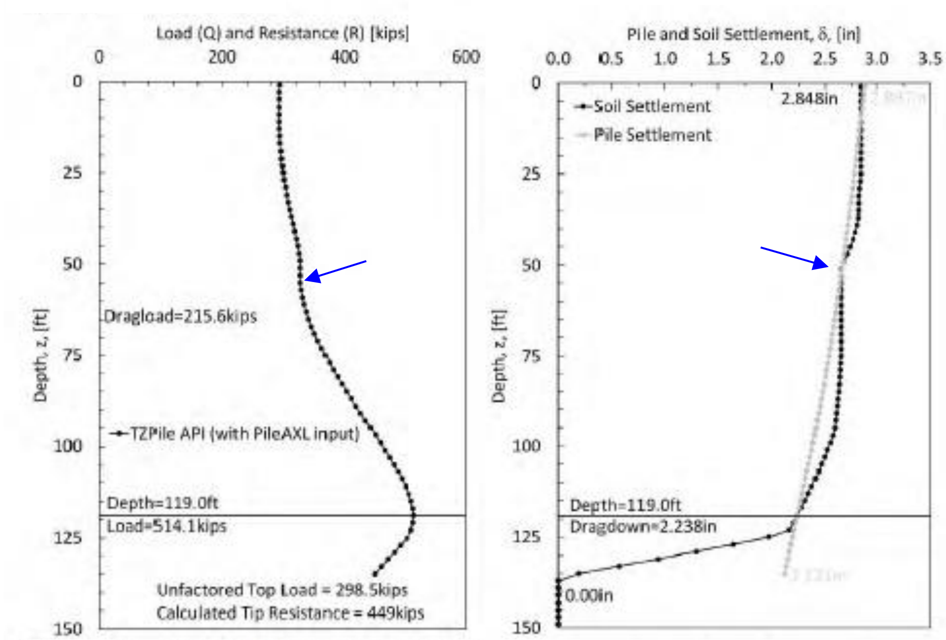

Fig. 7-4 Distribution of force and settlement for the 136-ft pile per the NCHRP report Figure I16 and I17

The two blue arrows added to the figure indicate the depth where the relative movement between the pile and the soil is small, essentially zero, and, therefore, the axial force is not changing at that depth. This is correct, but the analysis is very sensitive to small movements. Adjusting the pile compression to reflect the fact that the pile was subjected to some compression due the applied sustained load before the seismic event would be useful, as it would show smaller deformation that that estimated from assuming a zero condition of load and deformation.

The results of UniPile6 calculation of the same conditions as applied in the report for the two piles—as best as I can figure this out—are shown in Figure 7-5 for the 136-ft pile. The calculation includes adjusting for pile compression existing before the seismic event. Figure 7-6 shows the distributions when no such adjustment is made.

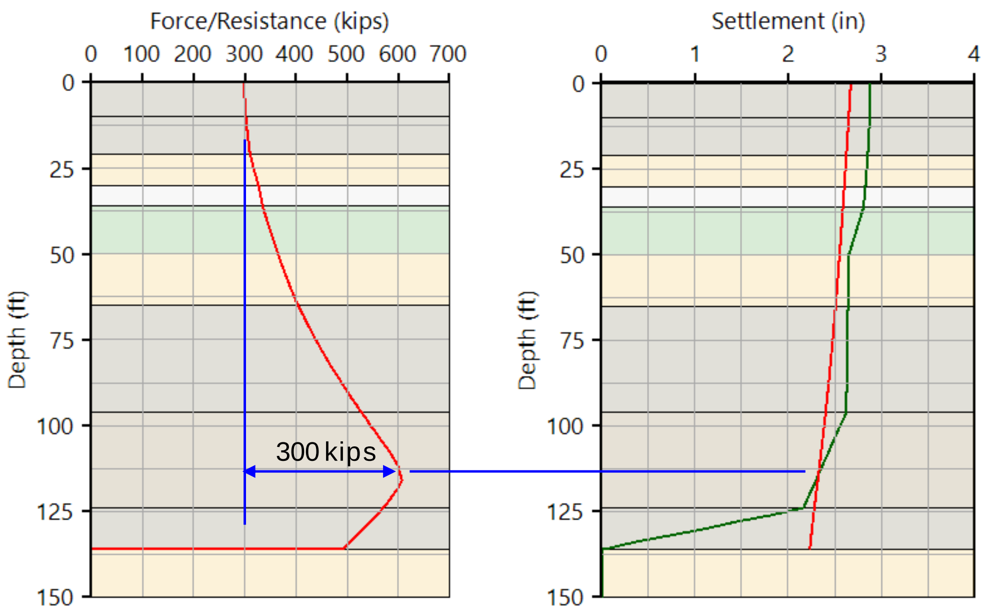

Fig. 7-5 136-ft pile Force and settlement distributions per the Unified Method (UniPile6)

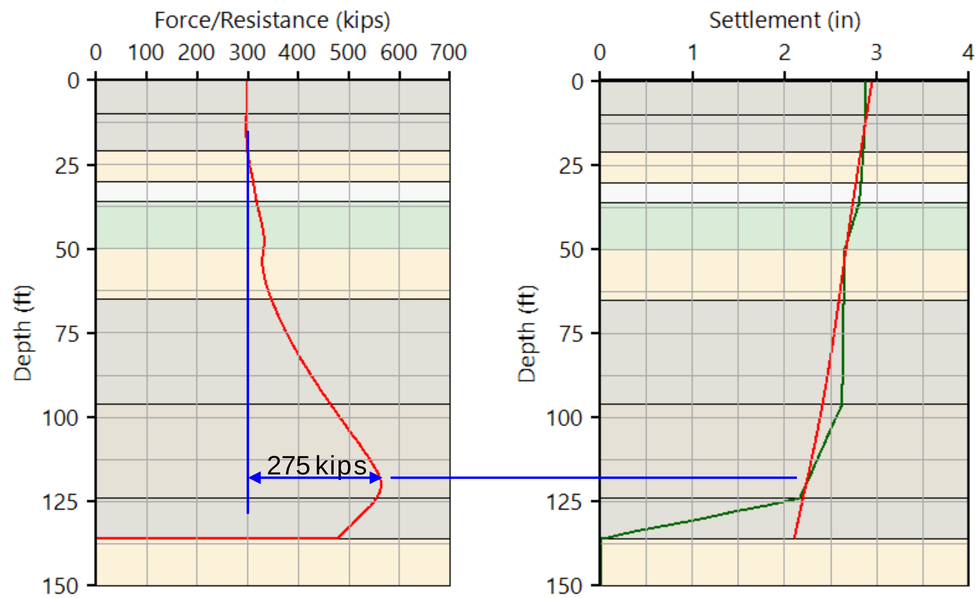

Fig. 7-6 136-ft pile Force and settlement distributions per the Unified Method (UniPile6) with pile compression not adjusted for pile compression before the seismic event

Example 8

The NCHRP Example 8, makes use of a case history reported by Budge and Dasenbrock (2016) comprising an abutment foundation for a single span railroad overpass bridge supported on two single rows of 15.8 m long 310HP73 (12HP53) driven through a 14.5 m thick deposit of compressible clay to bearing in weathered bedrock. The groundwater table was at the ground surface and the pore pressure, I assume, was hydrostatically distributed. No information is included on the magnitude of the sustained load intended for the bridge piles. Figure 8-1 shows the soil profile with values of undrained shear strength indicating firm consistency.

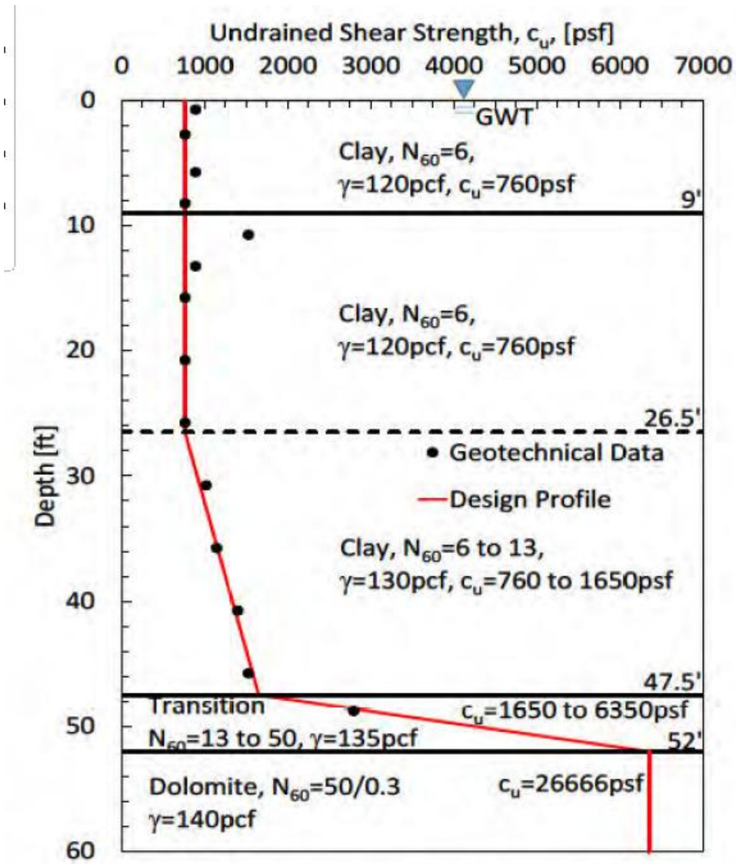

Fig. 8-1 Soil profile (Fig. J11 in the NCHRP report)

As mentioned by Budge and Dasenbrock, two CPT soundings, CPT-1 ands CPT-2, at the site were pushed into very dense till above the bedrock. One was pushed at each abutment, i.e., about 15 m apart. Figure 8-2 shows the CPT-diagrams plotted from the records. The CPT-2 shows presence of a dense, slightly dilatant, sand layer between depths of about 11.5 and 13 m that did not show up in CPT-1. The cone stress (q_t) indicates very soft consistency. (Neither the report nor the paper included the CPT records. The diagrams in the figure are plotted from records courtesy of Drs. A.S. Budge and D.D. Dasenbrock).

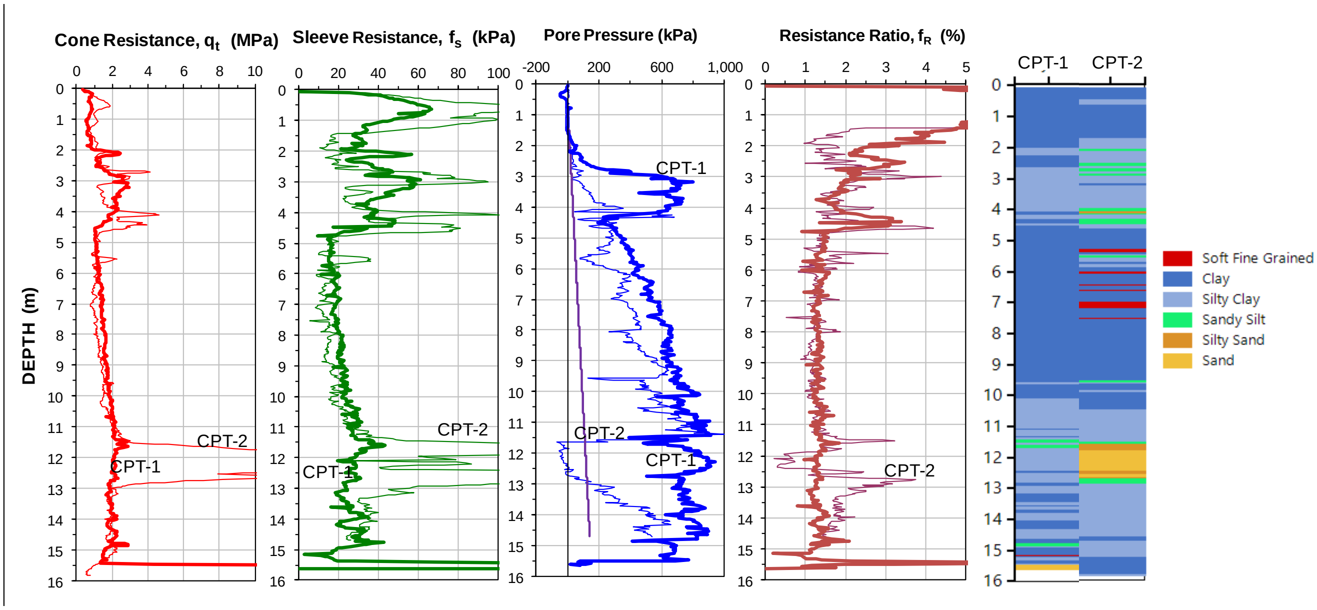

Fig. 8-2 Plot of CPT records from the CPTs pushed at the abutments

Twelve consolidation tests were performed, but the results were not included in the NCHRP report and the paper only presented a graph of one stress-strain test on a sample from 8.6 m depth with no interpretation of compressibility. Analysis of the data after digitizing the graph showed a compressibility of 60, expressed as a Janbu modulus number, which implies a low compressibility.

The overpass required placing an 8.8 m high embankment. The piles were shielded with a corrugated pipe in the embankment, but would be affected by downdrag and drag force in the clay. Figure 8-3 shows a cross section of the embankment at the abutment. One of the 11 foundation piles was instrumented and the pile movement and force distribution, as well as the soil settlement distribution were monitored during the placing of the embankment and a time thereafter (the actual duration was not stated).

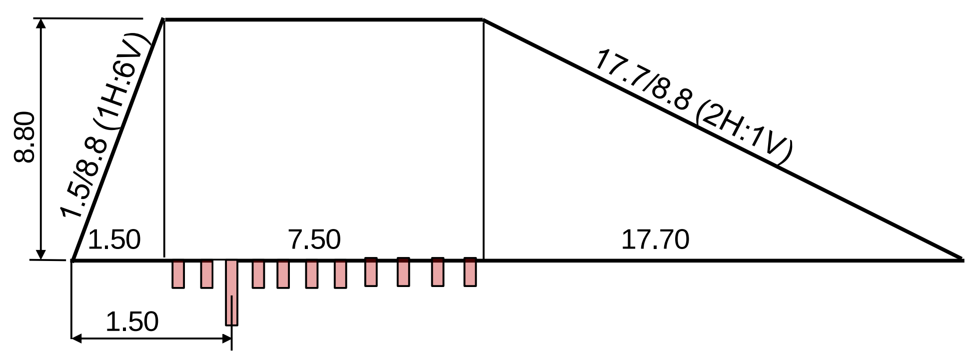

Fig. 8-3 Cross section of embankment at pile abutment next to the pile row.

The report includes a table (Table J2) with values of calculated settlement distribution. I expect that the values have been correlated to the observations. Figure 8-4 shows the plot of the indicated soil settlement distribution, determined using UniPile6 file with input of the soil profile and embankment. By trial and error calculations, the settlement distribution calculated by UniPile6 was matched to that of the table, resulting in a distribution of soil compressibility. (The fitted Janbu modulus number at 8.6 m was 50, quite close the number 60 extracted from the oedometer graph).

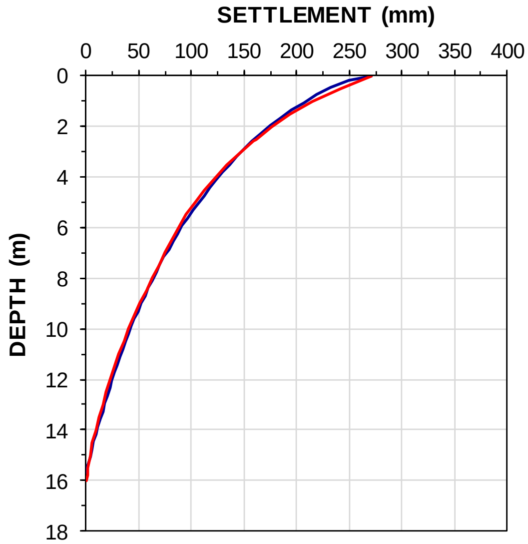

Fig. 8-4 Calculated distributions of soil settlement

The NCHRP report included a table (Table J3) showing the distribution of pile "ultimate" resistance, likely determined from the distribution of undrained shear strength. Similarly to the process of the settlement distribution, the UniPile6 file was fitted to the table data to determine the distribution of effective stress parameters for shaft and toe resistances represented by the table. Figure 8-5 shows the resistance curves of the table together with those obtained by UniPile after the fitting.

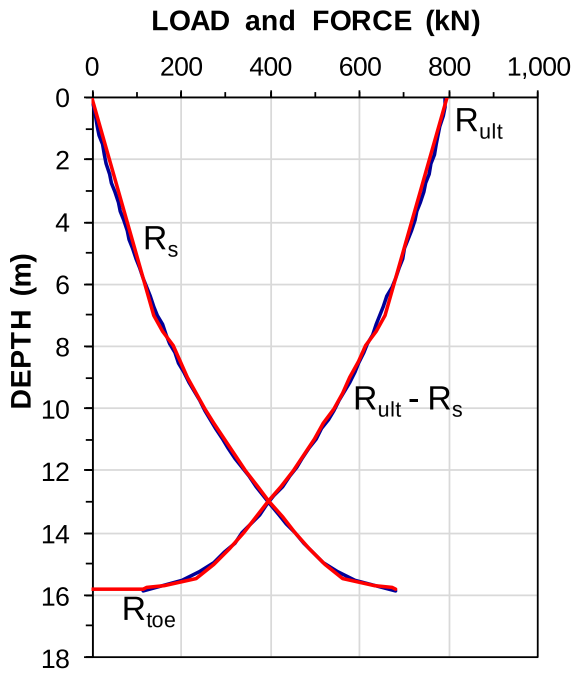

Fig. 8-5 Distributions of axial force for ultimate conditions

Figure 8-6 shows the CPT-calculated resistances using various methods applying the CPT-2 records. The figure includes the resistance distribution applied to the analysis as shown in Figure 8-5. It would seem that the analysis of the pile relied on the LCPC 1982 method. I would assume that the method had been confirmed as reasonably correlating with results of back-analysis of static loading tests for similar piles in similar geology.

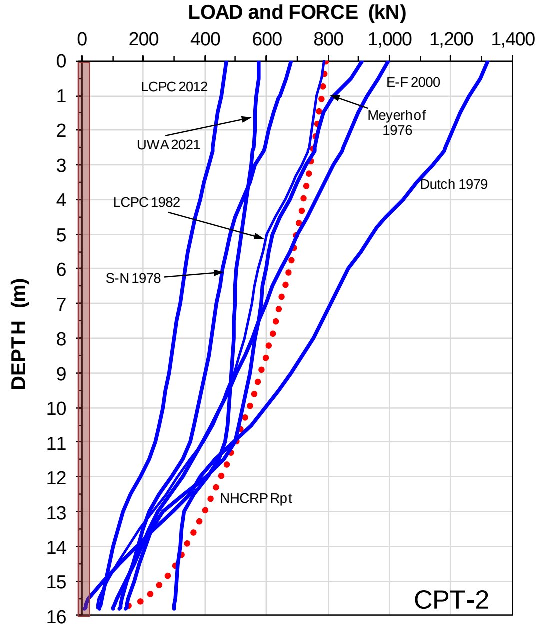

Fig. 8-6 Resistance distributions calculated by various CPT-methods

Figure J14 in the report shows a simulated load-movement graph of a head-down static loading test. The graph is plotted in Figure 8-7 together with a UniPile simulation using the resistances of Table J3. The blue curve is the pile-head movement of the NCHRP report and red and green curves are from the UniPile6 simulation. The UniPile6 simulation has been fitted to the report curve by means of trial-and-error calculations using t-z and q-c functions with the ultimate resistances of Table J3 taken as target values. The fitting is as good as reasonably possible considering that the same t-z function was used for the full length of the pile and the J14 figure does not indicate a toe response. The toe response of Table J3 was accepted for use in the UniPile6 analysis although it is rather small considering that the pile toe is ostensibly seated in bedrock.

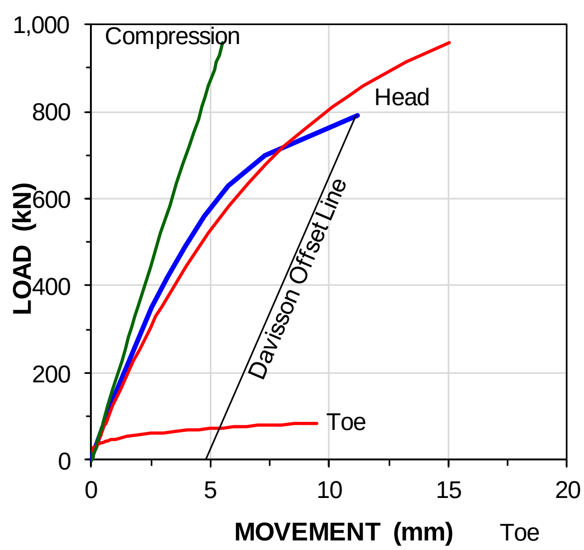

Fig. 8-7 Simulated static loading test load-movement curve

I wonder why Example 8 records were included amongst the design examples. The results of the monitoring data, some of which were included in the Budge and Dasenbrock paper were not used in the example. The example does not include a sustained load of the piles. Of course, the piles were indeed used to support a bridge and the example could be used to assess their suitability to support the bridge abutments. I assume the sustained load would be about 600 kN. The data extracted from Example 8 and input to UniPile6 can be used to assess the piled foundation. Figure 8-8 shows the long-term distributions of force and settlement for the software input. The input is that obtained for the fitting to the ultimate resistances and compressions given in the report as quoted above calculated for the long-term conditions and assuming that the sustained load is 600 kN/pile.

The maximum axial pile force (sustained load plus drag force) is about 1,500 kN, which well within acceptable limit of the pile structural strength. It would seem that the piled foundations would settle about 80 mm, which likely would be unacceptable. However, the pile toe is bearing on bedrock and the indicated pile-toe force is far smaller than bearing on bedrock would imply. After changing the input to represent a more reasonable stiffer pile-toe condition and repeating the calculation, Figure 8-9 results, showing an about 25 mm long-term settlement, a much more acceptable value. No mention is given in the report of the paper as to observations of the functioning of the bridge. I assume that the performance has been satisfactory and that no excess settlements have been observed.

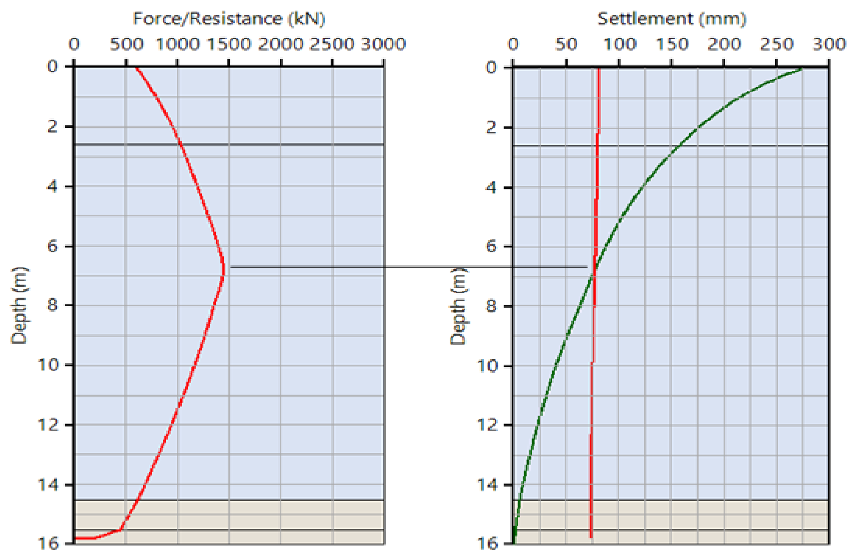

Fig. 8-8 Distributions of force and settlement for long-term conditions based on the fitted parameters

Conclusions

The NCHRP proposal and worked examples show convincingly that a drag force is an issue for the structural strength of a pile and not for the geotechnical bearing response of a piled foundation. The important issue is the downdrag, as it will affect the settlement of the foundation, indeed its serviceability. The downdrag can be analyzed by applying shaft- and toe-resistances utilizing so-called t-z/q-z relations specific to the case and combining them with the settlement of the soil developing as a result of pile loads and other sources changing the effective stress distribution. The main design and analysis issue is settlement. Settlement due to the load added to the piled foundation and settlement due to downdrag over the long-term.

The worked examples show the futility of applying methods that rely on ultimate resistance approach and demonstrate the reliability of integrating force-movement relations with the analysis.

The NCHRP report and worked examples is limited to the analysis for single piles only and this review has not addressed issues of pile groups. However, the UniPile software is equally suitable for analyzing pile groups be they narrow or wide, comprising a few piles or many, corner piles or center piles. Equally suitable for addressing interaction between piles and between piles and adjacent foundations, and back-calculating results from static loading tests, including bidirectional tests, routine tests or tests on instrumented piles. Such back-calculations will calibrate the pile-soil interaction in order to establish the input parameters to use in the design effort.

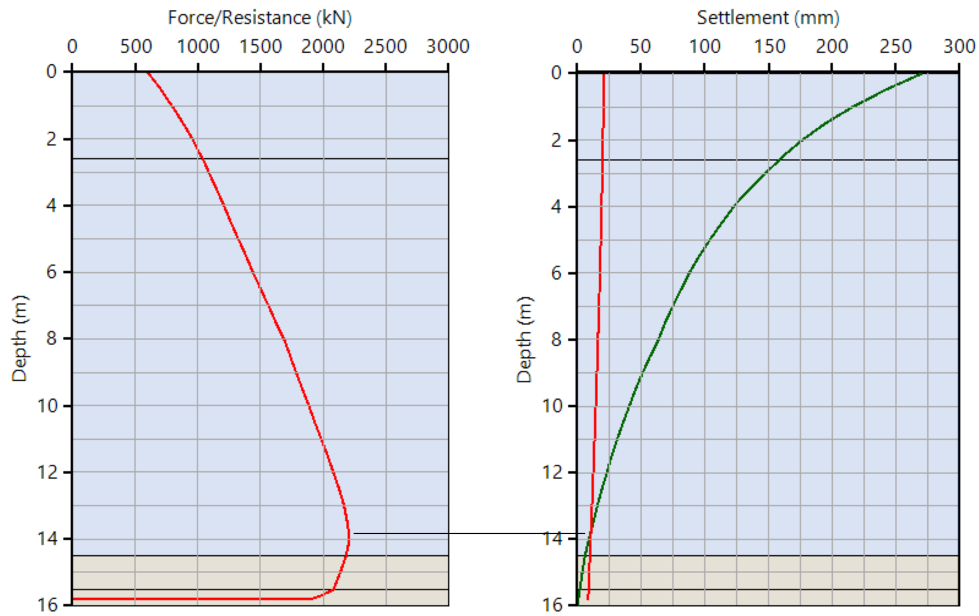

Fig. 8-9 Distributions of force and settlement for more realistic long-term conditions of pile-toe response

References

Briaud, J.L., and Tucker, L. (1997). NCHRP Report 393: Design and construction guidelines for downdrag on uncoated and bitumen coated piles. TRB, National Research Council, Washington, D C. 282 p.

Budge, A.S. Dasenbrock, D.D. (2016). The downdrag response of a driven pile bridge foundation in Minnesota. Proc. of GeoVancouver 2016, CGS, Vancouver, BC, October 2-5, 8p.

Coffman, R.A., Blanchard, J.D., Stuedlein, A.W., Jana, and A., Budge, A.S., 2024a. Design of piles for downdrag. NCHRP Research Report 1112, TRB, Washington, 353 p.

Coffman, R.A., Stuedlein, and A., Budge, A.S., 2024b. Pile design for downdrag–Examples and supporting material, NCHRP Web-Only Document 398 TRB, Washington, 285 p.

Ensoft 2021. TZPILE., Austin, Texas

Fellenius, B.H., 1984. Negative skin friction and settlement of piles. Proceedings of the Second International Seminar, Pile Foundations, Nanyang Technological Institute, Singapore, November 28 - 30, 12 p.

Fellenius, B.H., 1988. Unified design of piles and pile groups. Transportation Research Board, Washington, TRB Record 1169, pp. 75-82.

Fellenius, B.H., 2026. Basics of foundation design—a textbook. Electronic Edition, www.Fellenius.net, 572 p.

Fellenius, B.H., 2018. Development of axial pile load transfer curves based on instrumented load tests. Discussion, ASCE J. of Geotech. and Geoenviron. Engng., 44(2) 4 p.

Fleming, W.G.K., (1992). A new method for single pile settlement prediction and analysis. Geotechnique, 42(3) 411-425.

Innovative Geotechnics (2023). PileAXL2.5, Willetton, Western Australia.

Ladd, C.C., and R. Foott. 1974. New design procedure for stability of soft clays. ASCE J. Geotech. Geoenviron. Eng. 100(GT7) 763–786.

UnisoftGS 2024. UniPile6, Ottawa, Ontario, Canada. www.unisoftGS.com.