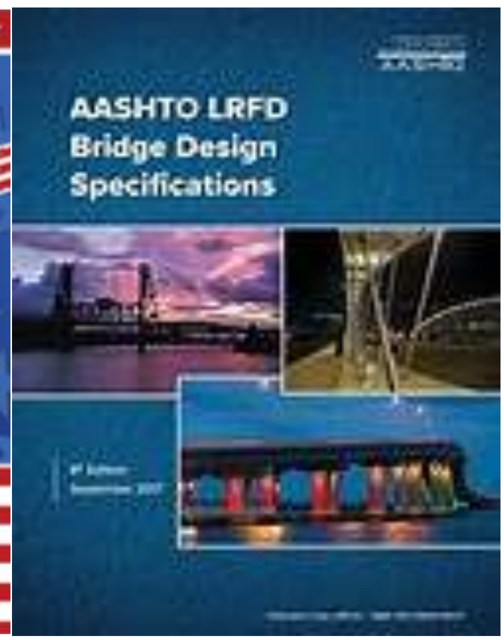

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DISCUSSION

Discussion of "Comparison of Canadian Highway Bridge Design Code and AASHTO LRFD Bridge Design Specifications regarding pile design subject to negative skin friction"[1]

Bengt H. Fellenius

The authors are complimented on a well-written and to the point article exposing the fallacy in the AASHTO LRFD Bridge Design Specifications (hereafter referred to as AASHTO Specs) (Bartz and Blatz 2020). The article is important for the practice in view of the fact that the AASHTO Specs are sometimes used in Canada — despite having no jurisdiction — as opposed to the Canadian Highway Bridge Design Code (hereafter referred to as the Canadian Code), which is more correct and, in contrast, has jurisdiction. The article is additionally important as the AASHTO Specs are also applied in many other countries — along with the 2017 EuroCode that is similarly erroneous in regard to the treatment of the issues associated with piled foundations in subsiding ground.

The authors employ a simple example involving a single H-pile (330 × 110 driven through a 10 m high fill over a clay layer with 1800 kg/m^3 density deposited on a "dense granular till"). The clay layer is either, 5, 10, or 15 m thick and the fill consists of the same material as the clay and it has the same density. The fill will result in settlement and, thus, a drag force will develop in the pile. The authors imposed that the eventual force equilibrium (neutral plane, N.P.) be located at the interface between the clay and the granular till.

The 1800 kg/m^3 density of the clay correlates to an about 40% natural water content, which is representative for a soft and compressible clay. I doubt very much that it would be practically possible to place a 10 m fill on such soil, even more so were it to consist of the same soft clay. Moreover, the authors assume that there is no reduction with depth of vertical stress imposed by the fill, which means that the fill must have at least a width equal to about twice the depth of the pile, i.e., about 60 m. The granular till is indicated to have a 2000 kg/m^3 density, which corresponds to an about 25% natural water content, a water content more representative for a loose to compact sand — a density of at least 2200 kg/m^3 (about 15% water content) would be more representative for a dense granular till.

I realize that the parameters of the example are not important to the comparison between the two codes, which after all is the authors' main objective with the article. However, in a paper addressing geotechnical engineering design, I think it is essential that examples are realistic. For example, the soil settlement (magnitude is essentially irrelevant to the comparison) could have been be instigated by a small lowering of the groundwater table and by setting the upper 10 m layer (the fill) to be a part of an existing soil profile of natural clay. These more realistic conditions would have given the same stress distribution as that for the authors' calculations.

The authors used effective stress analysis in the clay with a beta-coefficient of 0.25, but used stress-independent shear response in the till with a limit shear stress of 81 kPa (the 81 kPa limit was apparently chosen because the American Petroleum Institute (2007) recommends this for a "dense sand-silt", a soil quite different from a "dense granular till"). However, while it is sometimes practical to assume a stress-independent shaft resistance in clay, this is not correct for sand. The authors' applying stress-independent shear force limit in the till, that is, applying the same total shaft resistance (500 kN) to the 5 m thick granular till at each of the 15, 20 or 25 m depths, resulted in pile-toe resistance values of 600, 1000, and 1500 kN values, respectively. An effective stress analysis (with the authors' recommended correlation coefficient, β, of 0.37) returns shaft resistance values of about 600, 700, and 800 kN in the till (shaft area assumed equal to the square circumference of the H-pile) and toe resistances of about 500, 800, and 1200 kN over the square cross-sectional area. The differences are of little relevance for the authors' main conclusion about the costly consequence of the AASHTO Specs treating the drag force as a load. Neither are they of any substantial consequence for an ultimate limit state (ULS) analysis, that is, applying resistance factors to an ultimate resistance; the "capacity".

However, the response of a pile to an applied load, the load transfer, is not just in terms of shaft shear and toe stress. An integral part of the response is the movement of pile elements relative to the soil and the penetration of the pile toe. Both the conventional "capacity" approach and the ULS analysis approach pay little attention to movement, leaving movement and settlement to the serviceability limit states (SLSs). While the SLS does consider the settlement of the soil due to fill or a foundation, the current edition of the Canadian Code offers no specifics in regard to the load-transfer movement of a piled foundation and settlement induced by general subsidence (downdrag).

As mentioned by the authors, the settlement of a pile affected by general subsidence is the value that develops at the force equilibrium (neutral plane) plus the compression of the pile between the neutral plane and the pile head due to the axial force in the pile from drag force and sustained load. However, it is not enough to calculate the shaft and toe resistance values that will ensure that the force equilibrium lies at a certain depth — imposed in this case at the interface between the clay and the granular till. The values must be coupled to the movement, specifically the pile-toe movement — we can usually assume that the shaft resistance is essentially elastic–plastic, that is, it develops according to a

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Fig. D1. The shaft resistance function; t-z.

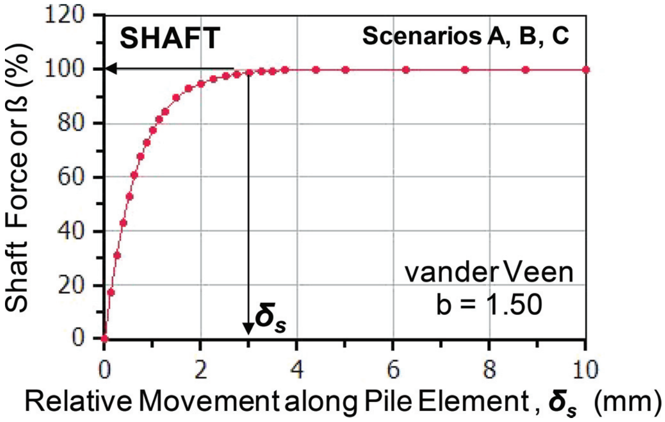
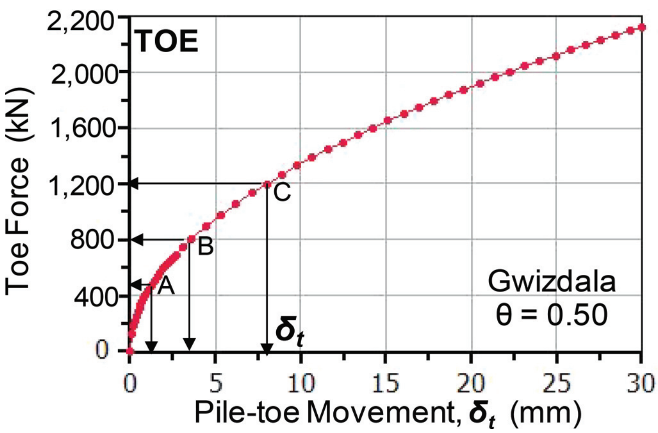


Fig. D2. The toe resistance function; q-z.

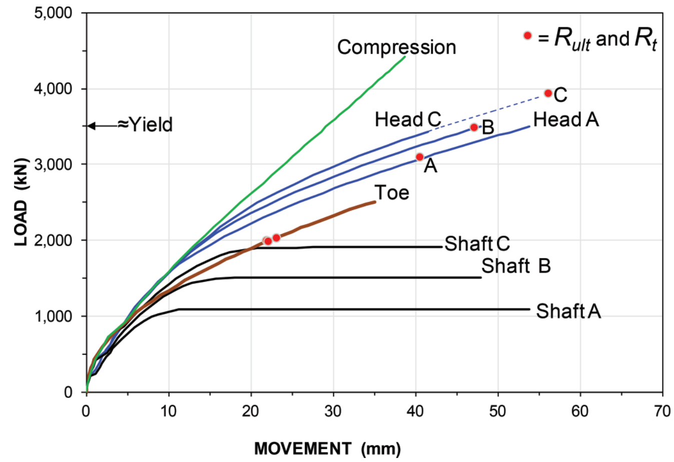


vander Veen function (Fellenius 2018, 2020) as illustrated in Fig. D1. The movement denoted δ_s is that necessary for developing 100% shaft shear, i.e., plastic response. To simplify matters for the analysis of the authors' example, I assume that the same t-z function applies to both the clay and the till. A vander Veen function coefficient, *b*, set to 1.50 indicates that 100% shaft shear (0.25 representing beta-coefficients of 0.25 and 0.37 in the clay and the till, respectively) will have developed at a 3 mm movement, δ_s, between the pile element and the soil.

If we assume that a realistic toe movement, δ_t, for mobilizing the 500 kN toe resistance, r_t, for scenario A would be 2 mm and that the pile-toe load–movement relation would be according to a q-z function expressed in the Gwizdala function (Fellenius 2018, 2020) with a function coefficient, θ, equal to 0.50; then, as shown in Fig. D2, the 800 and 1200 kN toe resistances for scenarios B and C, respectively, will develop at toe movements of 3.6 and 8.0 mm, respectively.

The authors assumed that the ultimate resistance of the clay layer was represented by beta-coefficients of 0.25 in the clay and 0.37 in the till layers, commensurably with a 2000 kN unit ultimate toe resistance. Figure D3 shows the static loading test simulation of the load–movements that would be found for the three scenarios using the mentioned t-z and q-z functions. The so-determined pile capacities, R_{ult}, and 2000 kN toe resistance, R_t, are marked out on the curves. The chosen q-z function has resulted in that the 2000 kN toe resistance value was, indeed, commensurable. The calculations were performed using the UniPile5 software (Goudreault and Fellenius 2014).

Fig. D3. Results of simulated static loading tests on the three piles.



Obviously, none of the three piles has reached "failure", though the ultimate resistances indicated by the authors are close to values that would be determined using the Davisson offset limit, a common definition for ultimate resistance. I believe that regardless of which definition of pile "capacity" that would be applied as derived from the load–movement measurements, the same conclusion would be drawn in regard to comparing the Canadian Code and the AASHTO Specs.

The ULS analysis only shows that a foundation supported on a single pile would satisfy the ULS requirements of the Canadian Code for all three scenarios. As mentioned, a design analysis needs also to determine what the foundation settlement might be and to assess if the structure can accept that settlement. The assessment includes estimating the pile compression above the N.P. due to the sustained load plus the drag force.

Moreover, the analysis must also consider the length of the movement necessary to fully mobilize the shaft resistance. By and large, the shaft resistance response can be considered plastic upon "full mobilization" as indicated in Fig. D1 and both the length of the transfer zone from fully mobilized negative skin friction above the N.P. and the length for fully mobilized positive shaft resistance below the N.P. are usually short. However, in the subject example, the N.P. is imposed to be at the boundary between the clay layer and the till layer. The compressibility of the clay is much larger than that of the till. Considering the pile compression contribution to the relative movement of about 0.5 mm/m, the subsidence of the clay immediately above the N.P. will provide the 3 mm distance required for fully mobilized shaft resistance in the clay, within a length of about 0.6 m above the N.P. For the transfer length in the till, however, the 3 mm necessary movement is all pile compression as the till is assumed to not compress. The compression of the pile between the N.P. and the pile toe will be about 3 mm, which means that the length from zero shaft resistance at the N.P. to full mobilization of the shaft resistance is just about equal to the 5 m embedment length.

Figure D4 shows the results of the long-term distributions of axial force for scenarios A through C with the lengths of the transfer zone equal to 0.6 m above and 5 m below the N.P., respectively. The thick solid lines in the left diagram represent the analysis for the gradual mobilization of the shaft resistance in the granular till from β = 0 through β = 0.37. The thin lines between the N.P. and the pile toe are those resulting from the authors' assumed toe resistance values and total shaft resistance in the till of about 500 kN.

The right diagram shows the compression of the pile for the three scenarios considering the movement necessary for mobilizing

Fig. D4. Correlation between distributions of force and settlement per the unified method.]

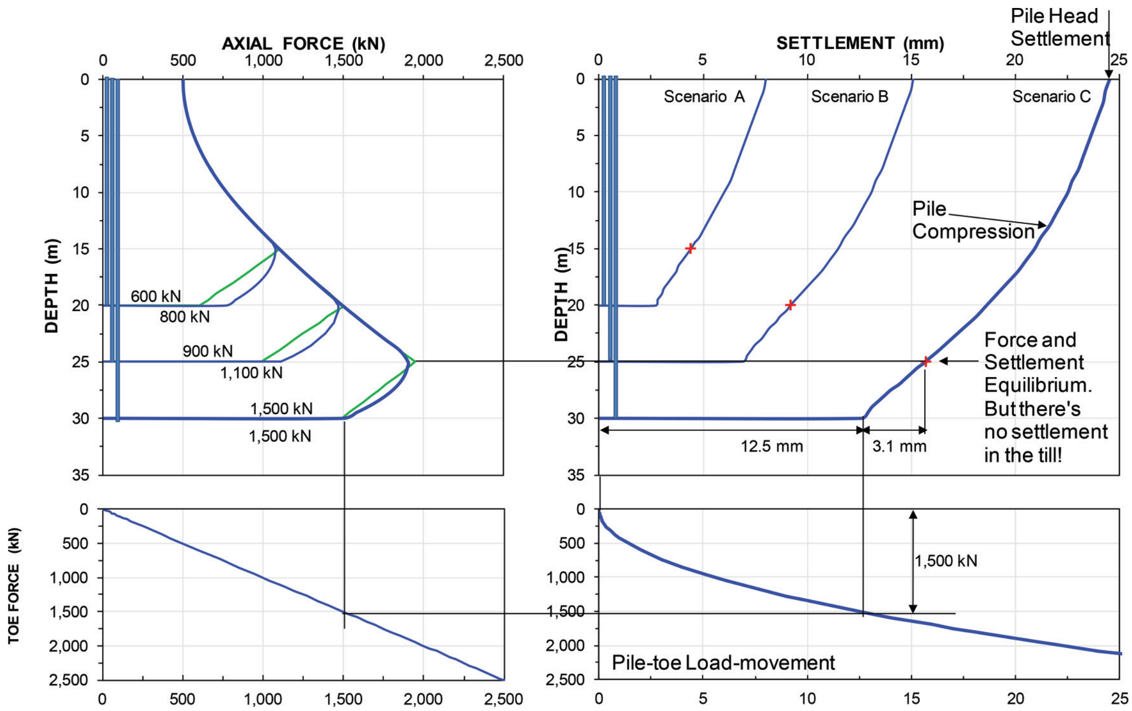
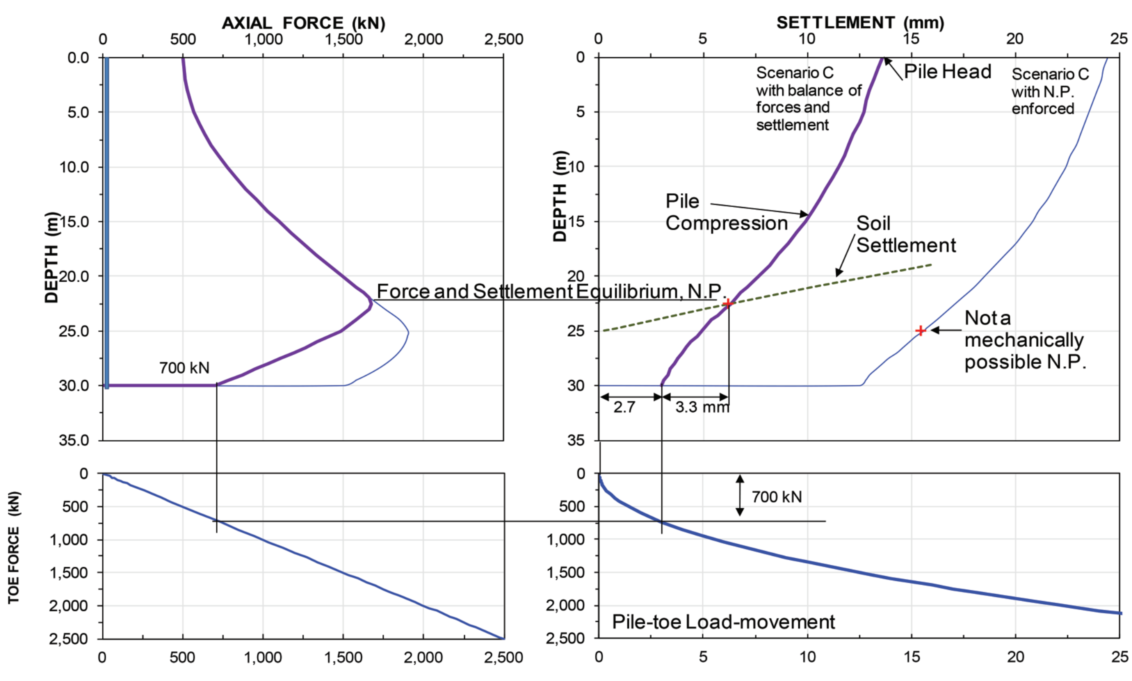


Fig. D5. Correlation of force and settlement that satisfies the pile movement vs. soil settlement.]



the shaft resistance and the pile compression due to the axial force. The soil settlement distribution is not shown other than at the N.P. (imposed at the clay–till boundary), where by definition the pile and the soil settle equally, the "settlement equilibrium". Adding the pile compression above the N.P. to the settlement at the N.P. represents the long-term settlement of the single pile foundation. The lower right diagram shows the pile-toe force–movement and the lower left shows construction to correlate the force and settlement diagram according to the unified method (Fellenius 2016, 2020).

While it is acceptable to construe an unrealistic example to illustrate a point, the example must still be mechanically possible. The imposed condition that there is no settlement at the boundary between the clay and the till is not commensurable with the sum of pile-toe movement plus the pile compression between the pile toe and the N.P. The analysis must either assume that the compression of the till is at least equal to the sum of toe movement and pile compression up to the N.P., or the N.P. must be moved up in the clay to where the condition of pile movement being equal to soil settlement is satisfied. The latter is achieved by reducing the pile-toe load until a match between the toe movement and compression and the soil settlement is found. Figure D5 shows the results for an assumed soil settlement of 6.0 mm at 22.5 m depth, 2.5 m above the clay–till boundary. If larger or smaller progress of soil settlement were assumed, the depth to the N.P. and the pile head settlement would change. This shows the importance of determining the settlement distribution around a piled foundation.

The authors showed that the ultimate pile resistances increased with increasing pile length, improving the ULS condition for each longer pile. However, the settlement diagrams show that the foundation settlement increases with the pile length. Even at the longest length, the values are still small. However, if the soil below the pile-toe levels would be compressible and develop settlement for the site conditions, the foundation settlement could well show to be excessive despite the very large ratio between the sustained load and ultimate resistance. The often-heard statement that "if capacity is OK, then, settlement will be OK, too" is not correct. An SLS analysis is always necessary and it is imperative that it includes proper relations between mobilized force and mobilized movement.

The authors' example pertains to a single pile and small (narrow) pile groups. Similar issues arise for the design of wide pile groups (at least four piles wide), where the perimeter piles respond in ways similar to the single pile, but the response of the interior piles is quite different (Fellenius 2019).

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