Design Procedure and Considerations for Piers in Expansive Soils

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Discussion "Design procedure and considerations for piers in expansive soils" by John D. Nelson, Erik G. Thompson, Robert W. Schaut, Kuo-ChiehChao, Daniel D. Overton, and Jesse S. Dunham-Friel. ASCE Journal of Geotechnical and Environ-mental Engineering, 138(8) 945-956.

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The authors present an interesting compilation of published methods for calculating soil and pier (pile) heave in expansive soils and propose a numerical method. They mention that the axial forces in a pier "must be in equilibrium", that is, there is always a depth in the pile where upward direction forces are in equilibrium with the downward direction forces. However, they do not mention that the heave movement must also have equilibrium between the heave of the pier and the heave of the ground near the pier and that the heave equilibrium depth is the same as the force equilibrium depth. In fact, the principles of the displacement for piles in expansive soil are the same as the principles of design for piles in settling soil. In order to make that clear, I need first to present a summary of the design method for piles in non-swelling soil.

Figure 1 shows the principles of the design of single piles and small pile groups in non-swelling soils, called the unified analysis, presented by Fellenius (1984; 1988; 2012). As the illustration has general validity, the figures show no numbers on the axes. The rather simplified soil profile is assumed to consist of two layers and the upper layer is settling after the load, Qd, has been placed on the piles. The settling soil produces negative skin friction along the pile accumulating to a drag force at the neutral plane. The load from the structure (Qd) and the drag force are in equilibrium with the positive shaft resistance and pile toe resistance. As shown in Fig. 1A, the negative skin friction and the positive shaft resistance are assumed equal, that is, the unit shaft shear is independent of the direction of relative movement between the pile and the soil. The transition of shaft shear from negative to positive direction does not occur suddenly, but within a tra's nsition zone—short or long depending on the magnitude of the relative movement between the pile and the soil. (For reasons of emphasizing the principles, Figs. 1A and 1B depict a small zone, despite Fig. 1C showing the relative settlement to be small above and below the transition zone, which in reality would infer a long transition zone). The contribution of the pile toe load to the depth of the force equilibrium—the neutral plane—depends on the downdrag-induced pile toe penetration. Determining the position of the neutral plane therefore includes matching the pile toe load to the pile toe penetration, that is, applies a q-z relation to the analysis.

All and any analysis of forces and movements must show the concurrence of force and movement equilibrium neutral planes. The magnitude of the settlement at the neutral plane determines the magnitude of settlement at the pile head—the settlement of the structure supported on the pile(s). Whether or not an analysis accurately predicts or agrees with measured movements is a function of the input of shaft shear and toe response to the calculation of the force distribution and of the input to the calculation of the settlement distribution. These details are critical to the analyses, but do not belong to the subject discussion.

The unified design principles of analysis apply equally well to piers in expansive soil as is illustrated in Fig. 2, which main difference to Fig. 1 is that the upper layer is assumed swelling, as opposed to settling. The swelling introduces positive skin friction along the pile and the requirement for force equilibrium means that the positive skin friction extends into the non-swelling soil. Fig. 2A assumes that the positive skin friction induced by swelling would be equal to the negative skin friction induced by settling. However, in contrast to the foregoing case, this is an oversimplification in swelling soil. When soil swells, the horizontal component of the overburden stress increases considerably. Several studies, e.g. Johnson and Stroman (1984), show that the horizontal stress can become three or more times larger than the vertical overburden stress. However, as the purpose of the figure is to illustrate the principles of developing the neutral plane, neglecting this aspect makes no difference. (The figure could have been drawn to show a smaller negative skin fiction unit shear, keeping the positive unit skin friction as is, which would have made no difference to the location of the neutral plane, or the opposite, which would have lifted the location of the force equilibrium, but made no difference to the illustrated principles. Moreover, because of the larger unit shaft shear in the active zone, the practice is to anchor the pier much deeper into the non-swelling soil than indicated in Fig. 2).

The distribution of the axial load shown in Fig. 2B assumes that the pile toe does not engage the soil sufficiently to require a toe load to be considered. However, there could be a small increase of the toe load in a particular case.

The important aspect of Fig. 2B is the calculation of the maximum load in the pile, which is a tension load. As a concrete section is much more sensitive to axial tension than to axial compression, potential tension damage to a pile due to swelling is more often a governing part of a design than the drag load is for piles in settling soil. For larger diameter piers, this is even more decisive because of the potential for the thermal fissures developing in the cooling stage of the concrete hydration (Sinnreich 2012, Fellenius and Tan 2012). The rebars placed in the pier may be the only pier component able to resist the tension.

In considering the potential for tension resistance reduction due to the smaller strength of concrete as opposed to in compression, a designer should recognize that a given upward unit shaft shear will result in a larger tension force in a small diameter pile as opposed to a large diameter pile. The relation is linear to the ratio of the diameters.

Fig. 2 shows that combining the distribution of force and the distribution of heave in the analysis is equally important when addressing piles in expansive soil as when addressing piles in settling soil. There are several important qualitative differences, however. For the design of piles in settling soil, the maximum axial load in the pile—the applied dead load plus the drag load—is rarely of concern for other than very long piles. The pile axial compression strength is normally quite satisfactory. For design of piers in expansive soil, however, the structural strength is critical.

The authors' presentation of the measurements of the pier at the Colorado University test site does not include the distribution of heave with depth. However, the observed heave values for the pier head and for the ground "adjacent" to the test pile still provide information in support of the use of the unified design analysis, as demonstrated by two diagrams in Fig. 3. The diagram to the left is a

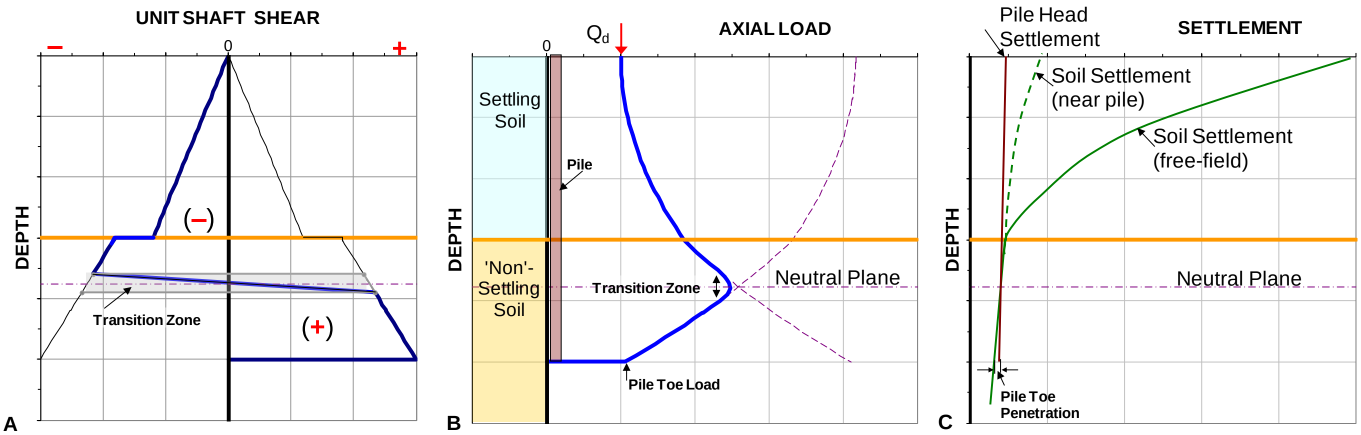

Fig. 1 Distributions with depth of unit shaft shear, load, and settlement

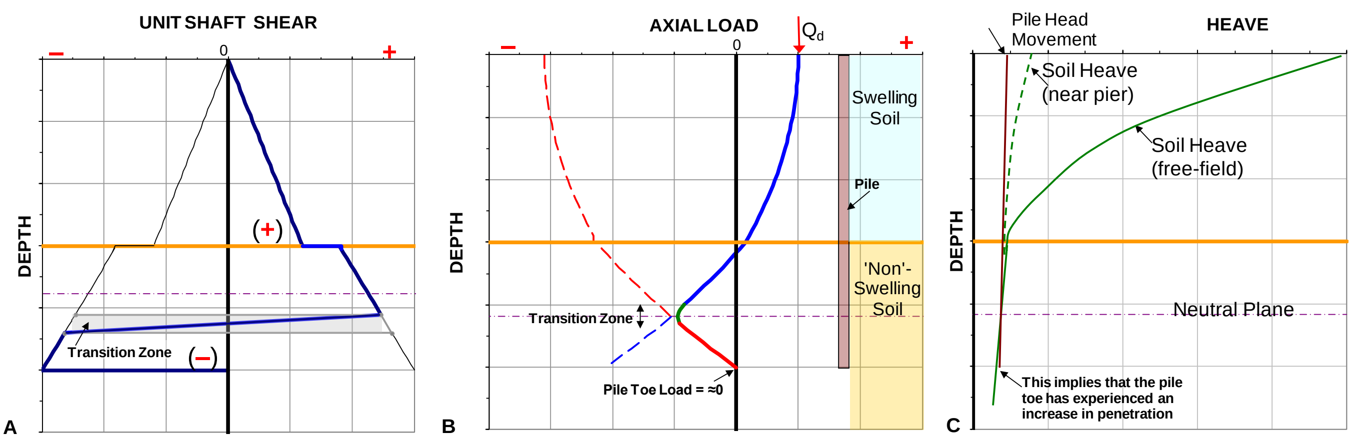

Fig. 2 Distributions with depth of unit shaft shear, load, and heave

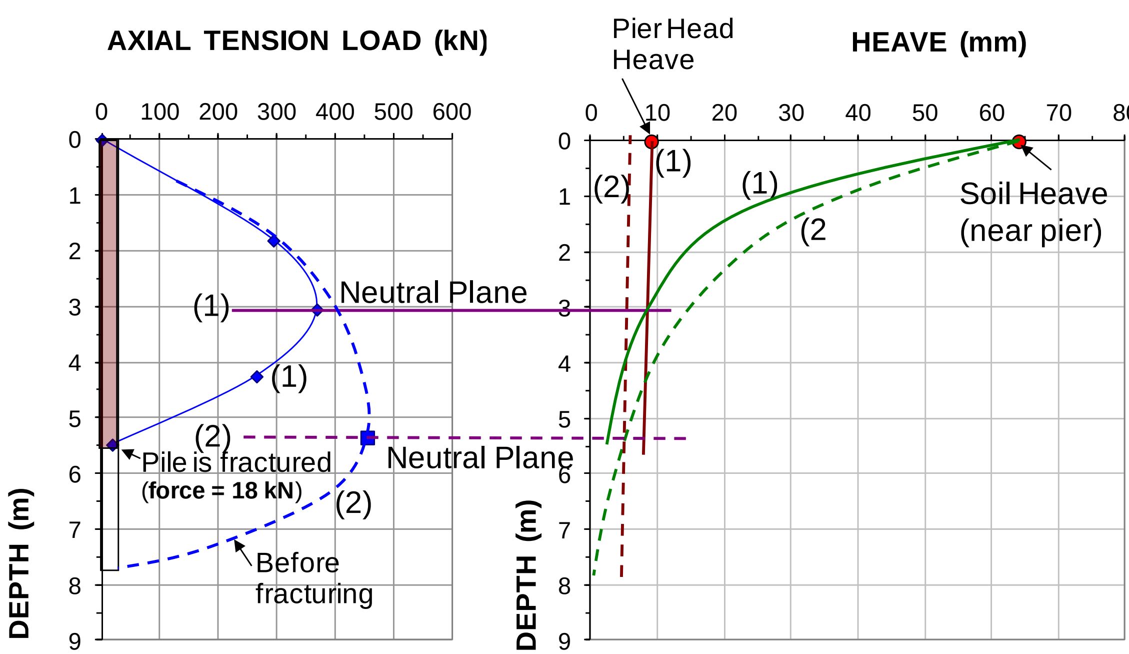

Fig. 3 Replot of authors' Figure 8 and deduced heave distribution (typo in abscissa label "kPa" is corrected to "kN")

replot of the authors' figure No. 8 that presented the loads (1) measured in the pile after a cracked (fractured) zone had developed at about 5.5 m depth and (2) before the cracks developed. The neutral planes are assumed located where the maximum loads were measured. The diagram to the right shows the two measured values of heave at the ground near the test pier and of the test pier head. The requirement that the settlement equilibrium must be located at the same depth as the force equilibrium results in the shown (1) distribution of soil heave, approximately deduced. Before the pile fractured, the load distribution could have been similar to the alternative marked (2) with the soil and pier head heave distributions marked (2).

The "quick and dirty" practical engineering design approach for piles in settling soil is to make sure that the neutral plane is located down in non-settling soil. The similarly practical approach for piers in expansive soil is the same: make sure that the neutral plane is located in the non-expansive soil. For both design approaches, the difficult part of the design is then limited to determining the load distributions to ensure that the neutral plane is indeed in the non-settling or non-swelling soil, respectively.

For calculating the maximum unit shaft shear along the pile, the authors recommend using the cohesion intercept plus the radial stress against the pile surface times the friction at the soil/pile interface (authors' equation No. 6). This expression is essentially the same as that for design of piles in non-swelling soil, commonly written as $r_s = c' + \beta\sigma'$. Of course, the proportionality coefficient, ß, is very much controled by the earth stress coefficient, K, which as indicated above, could be much larger than unity. Values back-calculated from results of static loading tests before swelling occurred or subsided have little validity.

The maximum unit shear—the "ultimate" resistance—is only one point on the shear-displacement curve, however. For, in particular, an analysis of pile soil interaction in an expansive soil, the shear resistance due to the pile-soil relative movement is important. However, in back-calculating the soil parameters from a static loading test or choosing them for a design case, the shape of the curve for shear stress versus movement must be included in the analysis. Such curves are referred to as t-z functions (q-z for the pile-toe stress-response). A "slip and soil failure" response is not adequate. Moreover, while the pier compression and elongation can be omitted for short and stubby piers, it cannot be omitted from the analysis of long slender piers or piles. Four common shapes of the unit shaft shear curves are shown in Fig. 4. They are called "Ratio", "Hyperbolic", "Exponential", and "80%" and the mathematical relations are available in Fellenius (2012). The curves are normalized to the ultimate shear value and assume that it occurs at the same movement, but for the "Exponential" method, which assumes that the ultimate value occurs at infinite movement. Usually the movement at the ultimate value is small; 4 mm is used for the curves in Figure4. The four functions can be used to model the shear responses from strain-softening to elasto-plastic, to strain-hardening, and to considerable strain-hardening.

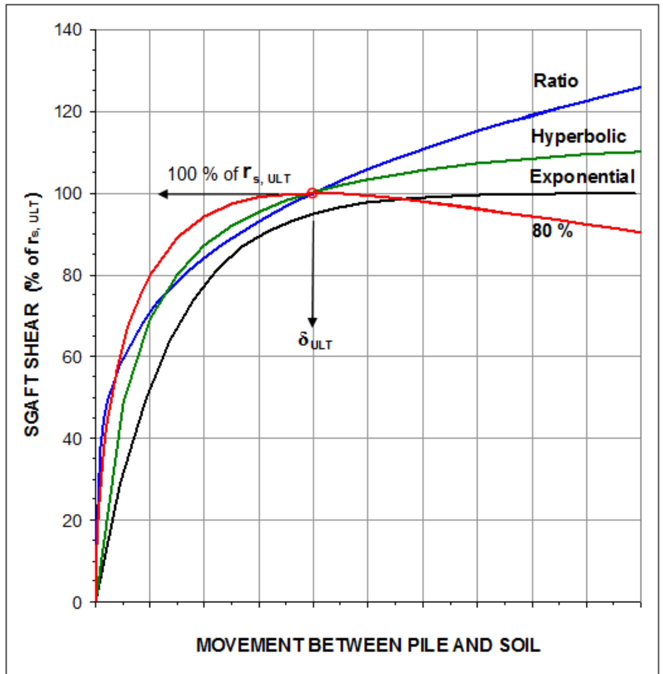

Fig. 4 Unit shaft shear versus movement for four types of t-z functions

The principles of the unified method, the analysis using the t-z and q-z functions can be and has been combined with settlement and/or heave analysis in software ranging from the simple to the sophisticated. The key aspect is that the software must be able to work out the common depth to the neutral plane for both the force and settlement equilibrium.

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