NEUTRAL PLANE SOLUTION FOR LIQUEFACTION-INDUCED DOWN-DRAG ON VERTICAL PILES

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ABSTRACT

Down-drag loads on pile foundations can be an important design consideration when earthquake-induced liquefaction is expected to cause ground settlements. A modified neutral plane solution for liquefaction-induced down-drag on vertical piles is described that accounts for the variation in excess pore pressures and ground settlements over time as a liquefied layer reconsolidates, the dependence of sand compressibility on excess pore pressure ratio, and the dependence of shaft skin friction on the excess pore pressure ratio. A worked example illustrates the role of various parameters on peak pile loads and settlements. The modified solution predicts substantially smaller pile settlements than obtained from a traditional neutral plane solution for end-of-consolidation conditions. Recommendations for design practice are presented.

INTRODUCTION

Down-drag loads on pile foundations are an important design consideration when earthquake-induced liquefaction is expected to cause ground settlements. There is a need for direct measurements, from case histories or physical modeling studies, of down-drag loads on piles in liquefied soils. Pending such data, the response of pile foundations to down-drag loads is generally analyzed using methods developed for other situations.

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The neutral plane solution (Fellenius 1972) has been used to estimated down-drag loads and settlement of vertical piles due to consolidation of clays. Although the down-drag load or skin friction in the consolidating clay will increase over time as the effective stresses increase (pore pressures decrease) during consolidation, for most situations the down-drag loads are treated as constant, typically equal to the values expected at the end of primary consolidation (e.g., Wong and Teh 1995). The neutral plane solution can be used to derive the rule of thumb that pile settlements will be small if the sum of the superstructure's service load and the down-drag load is less than or equal to the sum of the resisting load capacities from the underlying (essentially non-settling) layers.

This paper describes a neutral plane solution for down-drag loads on vertical piles in liquefied soil deposits (Boulanger et al. 2003). This solution relates shaft friction in a reconsolidating liquefied layer to the variation in excess pore pressures (Δu) over time. The sand compressibility (m_v), which depends on excess pore pressure ratio ($r_u = u/\sigma_{vo}'$), is used to calculate ground settlements and pile head settlements as the liquefied layer reconsolidates. Questions that are explored include whether it is necessary to include down-drag loads from the liquefied layers since the shaft friction will slowly increase toward its fully-drained capacity as the liquefied sand reconsolidates, and how degradation of skin friction with relative slip displacement or gap formation affects the down-drag loads and pile settlement. These and other aspects of liquefaction-induced down-drag loads are discussed, and recommendations for practice summarized.

NEUTRAL PLANE SOLUTION FOR LIQUEFACTION

The modifications to the neutral plane solution by Fellenius (1972) are first described and then illustrated through its application to an example problem. Calculating settlements during reconsolidation for a liquefied layer requires (1) a description of excess pore pressure isochrones over time, and (2) a relation between m_v and r_u. The dissipation of Δu over time has been well described by physical model studies and numerical analyses (e.g., NRC 1985). Observed patterns (isochrones) from such prior studies are directly used herein, as will be described with the example problem.

Shaft friction within liquefied sand was modeled as being proportional to the effective stress in the sand, as:

$$f_s = \sigma_{vo}' K_o \tan(\delta)(1 - r_u) \tag{1}$$

where σ_{vo}' is the vertical effective consolidation stress, K_o is the coefficient of lateral earth pressure at rest, and δ is the interface friction angle. The values of K_o and δ undoubtedly change during the course of liquefaction and reconsolidation, but in the absence of data describing their changes, these values were kept as constants in this study. In addition, variations in these parameters over time are likely to have a small effect on skin friction compared to that of the r_u.

The relation by Seed et al. (1975) was used to model the variation of m_v with r_u. This relation approximates the lab test results by Lee and Albaisa (1974) as follows.

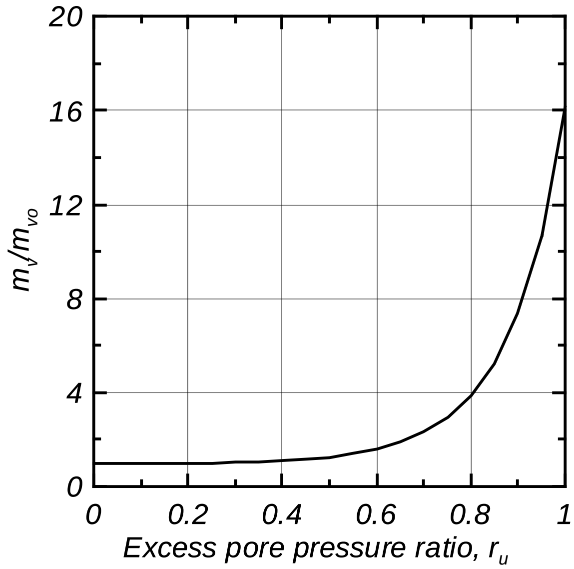

Figure 1. Relation between sand compressibility and excess pore pressure ratio.

$$\frac{m_v}{m_{vo}} = \frac{exp(y)}{1 + y + y^2/2} \tag{2}$$

$$y = 5(1.5 - D_R)r_u^b \tag{3}$$

$$b = 3(4^{-D_R}) \tag{4}$$

where m_{vo} = sand compressibility with zero excess pore pressure, and D_R = relative density. The calculated variation of m_v/m_{vo} with r_u is shown in Figure 1 for D_R=30%.

Large relative displacements (S_{rel}) can develop between a pile shaft and a nonliquefied "crust" that settles as the underlying liquefied layer reconsolidates. The shaft friction between the pile and "crust" soils may experience some post-peak degradation at large S_{rel}, depending on the nature of the soil. This possibility was modeled by expressing the ratio of $f_s/(f_s)_{peak}$ (for the crust only) as a function of the S_{rel}/D, where D=pile diameter. As shown in Figure 2, the ratio of $f_s/(f_s)_{peak}$ was modeled as degrading to a value R when S_{rel} is 10 percent of the pile diameter.

The example problem in Figure 3 involves a single vertical 0.4-m-square, 17-m-long pile carrying a dead load of 445 kN. The soil profile consists of 4 m of clay, over 6 m of liquefiable sand, over stiff clays to large depths. The water table is at a depth of 4 m, and all soils weigh 20 kN/m^3. Shaft friction is 40 kPa in the upper clay and 50 kPa in the lower clay. Degradation of f_s is not included (i.e., R=1). For the sand, K_o is 0.5, δ is 28 degrees, and m_{vo} is 1.0×10^{-4} m^2/kN. The tip bearing capacity (Q_p) was taken as 144 kN, assuming sufficient tip movement has occurred for Q_p to be fully mobilized. If sufficient tip movement does not occur, then only the mobilized portion of Q_p should be used to calculate the location of the neutral plane (Fellenius 1972). The sand is assumed to completely liquefy during shaking (i.e.,

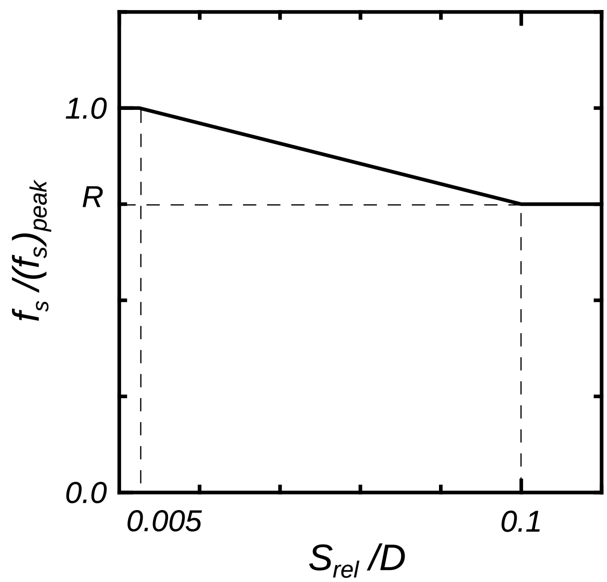

Figure 2. Normalized skin friction in the crust versus relative displacement.

r_u=1.0), and then down-drag develops as the sand reconsolidates and the ground surface settles. Lastly, this analysis only considers settlement due to liquefaction-induced down-drag, and does not include prior settlements under long-term static loads or settlement of the soils beneath the liquefied layer.

Isochrones of Δu at various times during reconsolidation of the liquefied sand are shown in Figure 3. These isochrones follow the patterns that have been observed in physical modeling studies and predicted by numerical analyses for liquefied layers bounded above and below by lower-permeability soils (e.g., Florin and Ivanov 1961, NRC 1985). Soil profiles with different boundary drainage conditions would have different isochrone shapes and would produce different estimates of pile head settlements. Four different times are shown in Figure 3, with t_o being immediately after r_u=100% develops and t_3 being when Δu has fully dissipated.

The remaining plots in Figure 3 show the corresponding values of shaft friction, soil settlement, and shaft loads (Q) at the same four times. Soil settlement is calculated by integrating the vertical strain (ε_v) in the soil profile as the sand reconsolidates. Vertical strains are calculated by numerically integrating the product of $\Delta\sigma_{vo}'$ and m_v over time. The shaft loads (Q) are, as for the conventional neutral plane solution of Fellenius (1972), calculated for two conditions: loads are summed downwards from the pile head (Q_{down}), and upwards from the pile tip (Q_{up}). The neutral plane is then identified as the depth at which Q_{down} equals Q_{up}, which corresponds to the pile being in equilibrium with relative soil-pile displacements being downward above the neutral plane and upward below the neutral plane. In addition, it is assumed that full shaft friction is mobilized everywhere along the pile, with its direction only depending on the direction of relative soil-pile displacements. The neutral plane location at time t_3 is labeled on Figure 3 to complete the illustration.

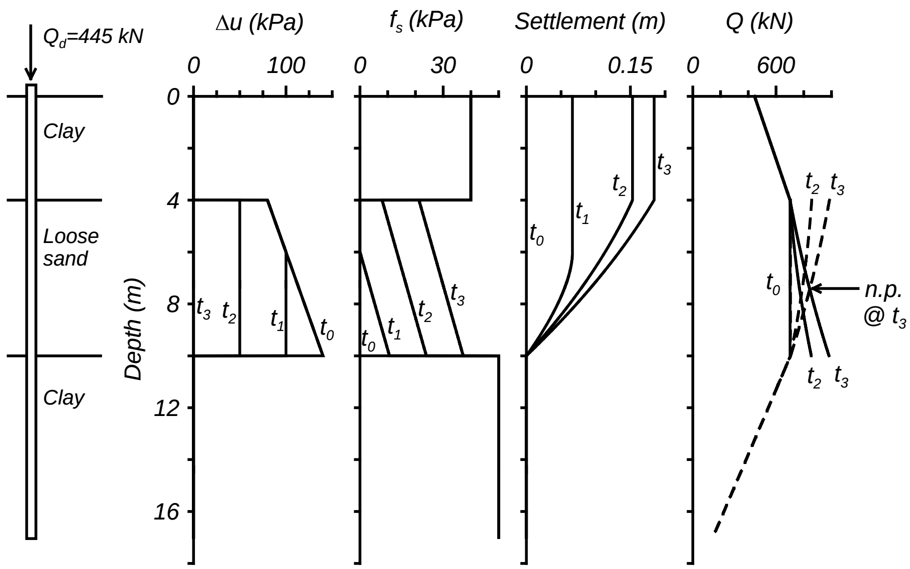

Figure 3. Example of neutral plane solution for down-drag due to liquefaction.

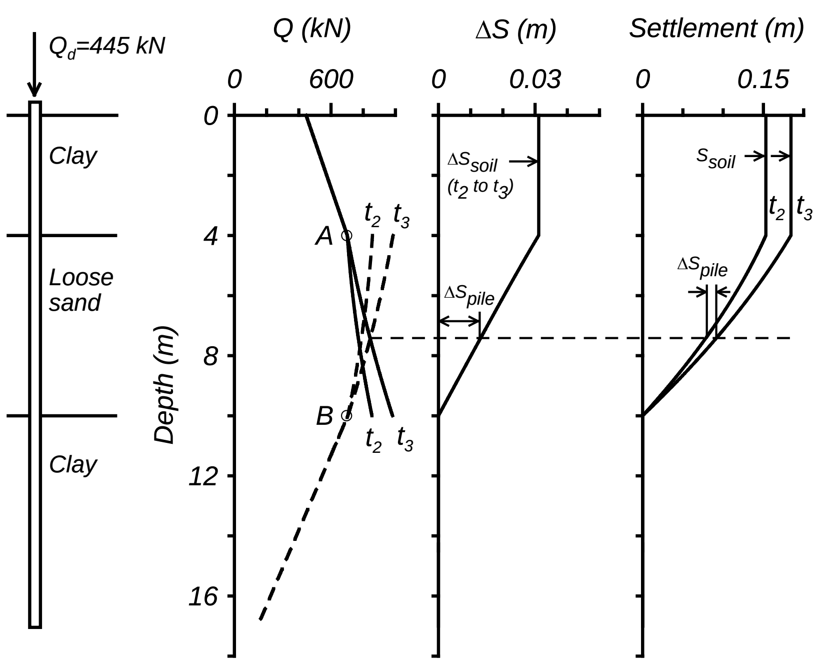

Figure 4. Solution for an increment of liquefaction-induced down-drag.

Pile settlements are calculated differently than in the traditional neutral plane solution. In the approach of Fellenius (1972), the pile settlement equals the soil settlement at the neutral plane location at the end of consolidation. In the present analysis for liquefaction conditions, the neutral plane location varies with time as the shaft friction in the liquefied sand increases during consolidation. Hence, the pile settlement is calculated incrementally over time as illustrated in Figure 4. For example, consider the increment of time from t_2 to t_3. The neutral plane shifts

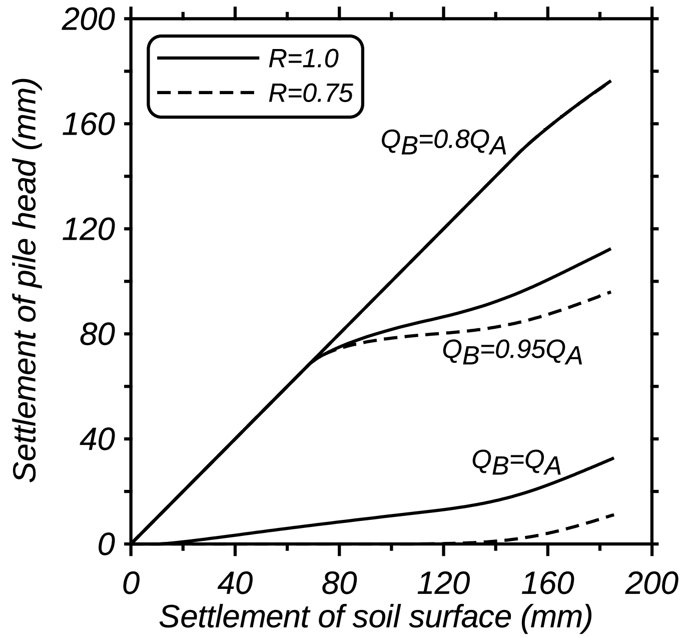

Figure 5. Pile versus ground surface settlement for different combinations of capacity and post-peak softening of down-drag.

upward between these two time steps because the shaft frictions are increasing in the reconsolidating sand. The increment of pile settlement (ΔS_{pile}) equals the increment of soil settlement (ΔS_{soil}) at the neutral plane location for the end of this time step (i.e., at time t_3). The resulting value of ΔS_{pile} is labeled on Figure 4. The total pile settlement is then obtained by numerically integrating the increments of pile settlement over the time for reconsolidation. For the example in Figure 4, the final pile settlement is 33 mm and the final settlement of the ground surface is 184 mm. The depth of the final neutral plane is 7.3 m, where the final soil settlement is 94 mm. Hence, a traditional neutral plane solution based on end-of-consolidation conditions alone would have over-predicted pile head settlements by a factor of almost three (94 mm versus 33 mm).

The sensitivity of the pile settlement to other combinations of axial capacities and shaft friction degradation is illustrated in Figure 5, showing pile settlements versus ground surface settlements. In this Figure, Q_A refers to the sum of the downward loads (Q_d plus peak down-drag from the crust) at point A in Figure 4, while Q_B refers to the sum of the upward resisting capacities (Q_p plus shaft capacity from the lower clay layer) at point B in Figure 4. For the previous example, Q_A was 701 kN and Q_B was 704 kN, for a ratio of QB/QA of 1.0. For Figure 5, the pile length was changed for two additional cases such that Q_B had values of 666 kN and 561 kN (giving Q_B/Q_A of 0.95 and 0.80, respectively). Each case was analyzed without shaft friction degradation (R=1) and with some nominal shaft friction degradation (R=0.75). Note that the ratio Q_B/Q_A is based on the peak down-drag from the crust even if R<1.0.

For the case with Q_B/Q_A=1.0, the inclusion of shaft friction degradation reduced the final pile head settlement, from 33 mm for R=1 to 11 mm for R=0.75. This benefit can be explained as follows. First, the initial portion of ground surface settlement is associated with Δu dissipation at the bottom of the sand layer, and thus f_s in the liquefied layer increases at the bottom first, which acts to offset any increase in down-drag from the upper portions of the liquefied sand layer. Since Q_B/Q_A=1, the neutral plane stays in or near the lower clay layer and so pile settlements are small. Consequently, the ground surface settlement is larger than the pile settlement, which degrades the down-drag load from the crust (since R=0.75). This degradation of down-drag load allows the neutral plane to remain within the lower clay layer until near the very end of settlements. In addition, the final pile settlement was negligible if R was further reduced to 0.5.

For the case with Q_B/Q_A=0.8, the pile essentially settles with the ground surface regardless of the R value. As a result, relative displacements between the pile and the crust are small, and the shaft friction does not degrade. The case with Q_B/Q_A=0.95 shows behavior intermediate to the other cases. In summary, Q_B/Q_A ratios less than about 0.8 result in the pile settlement matching the ground surface settlement, while Q_B/Q_A ratios much greater than 1.0 result in very small pile settlements.

DISCUSSION

Down-drag loads from a settling nonliquefied crust may also degrade in response to other factors, including the formation of sand/water boils along the sides of the piles and the formation of gaps due to the seismic lateral loading. Sand boils and water ejecta are often observed alongside piles, likely because the soil-pile interface provides a preferential path for the escaping materials. Boiling and gapping may contribute to degradation of the down-drag loads, but their effects are hard to quantify. For minimizing settlements, it is therefore prudent to require that Q_B/Q_A be greater than 1.0 for peak down-drag loads, and accept that the reduction of down-drag loads due to boiling and gapping are an additional but unpredictable benefit.

The neutral plane solution indicates that shaft friction in the liquefied layer will increase to its drained capacity as pore pressures dissipate. The maximum shaft load (Q) after pore pressures have fully dissipated will have returned to the maximum shaft load that existed prior to liquefaction (assuming R=1). This maximum shaft load is greater than either Q_A or Q_B and thus may be important for the structural design of the pile.

Some immediate (elastic) settlement or heave of the pile head may also occur when the shaft friction in the sand layer first drops to zero at the onset of liquefaction. However, this elastic settlement or heave is recovered at the end of consolidation, assuming the stress state returns to the pre-liquefaction condition. Furthermore, elastic pile settlements in the example problem were orders of magnitude smaller than those induced by ground settlement and could reasonably be neglected.

Uncertainties in the structural dead loads, down-drag loads from any settling crust, and axial capacities below the liquefied layer are extremely important, and likely outweigh the uncertainties associated with the neutral plane solution's approximations.

Down-drag loads can be expected to develop largely after shaking as the liquefied soils reconsolidate, and thus should be applied in conjunction with the expected service loads. While down-drag loads are a possible consequence of seismic shaking, they should not be applied in conjunction with the design seismic loads because they will not occur at the same time.

Estimating tolerable pile settlements is a key step in designing for down-drag loads and evaluating mitigation strategies. For example, the magnitude of pile settlement that a bridge can tolerate depends on the performance objectives (e.g., functionality versus life safety), the type of bridge structure, the span length, and the variability of settlements between bridge supports (i.e., differential settlements). Detailed guidance is not currently available for specifying tolerable settlements for bridges under these types of loading conditions, and therefore the specification of tolerable settlements requires careful consideration by the structural design team. Note that the tolerable settlements to maintain life safety under down-drag loads can be much greater than commonly specified for static settlements of newly constructed bridges. In some cases it can be cost effective to repair a bridge after it experiences down-drag induced settlements rather than retrofitting it to prevent settlements.

CONCLUSION

The modified neutral plane solution presented herein provided a method to evaluate the potential increase in down-drag load (or skin friction) within a liquefied soil as Δu dissipates (σ_v' increases) during reconsolidation. The analysis method accounts for the variation in Δu and ground settlements over time as a liquefied layer reconsolidates, the dependence of m_v on r_u, and the dependence of f_s on r_u. The modified solution predicts substantially smaller pile settlements than obtained from a traditional neutral plane solution for end-of-consolidation conditions.

The results of a parametric study using the modified neutral plane solution for the case of a lower permeability layer settling over a liquefied sand layer leads to the following simple guideline, which is very similar to that for conventional down-drag problems. Pile settlements will be small provided that the sum of the expected service load and peak down-drag load from any settling crust do not exceed the sum of the resisting capacities from below the liquefied layer; Down-drag loads from within the liquefied layer do not need to be included in this criterion. For design, the amount by which the predicted capacity should exceed the sum of the predicted service load and down-drag load from the crust depends on the uncertainty in estimating pile loads and resistances, and is left to the discretion of the designer. As a practical consideration, a 20% decrease in pile capacity (i.e. from $Q_B/Q_A = 1.0$ to $Q_B/Q_A = 0.8$) resulted in more than 5 times as much pile head settlement, which indicates that pile head settlement can very sensitive to relatively small changes in pile capacity.

Pile settlements can be as large as the ground surface settlements if the sum of the expected service loads and down-drag loads exceed the resisting capacities from below the liquefied layer. The specification of tolerable pile settlements for different performance targets is a key consideration in evaluating mitigation strategies, and this aspect of the design process warrants additional study.

ACKNOWLEDGMENTS

Caltrans supported this study under contract 59A0162. However, the contents of this report do not necessarily represent a policy of these agencies or endorsement by the State or Federal government. The authors appreciate the comments and suggestions of Bruce Kutter, Dongdong Chang, Abbas Abghari and Angel Perez.

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