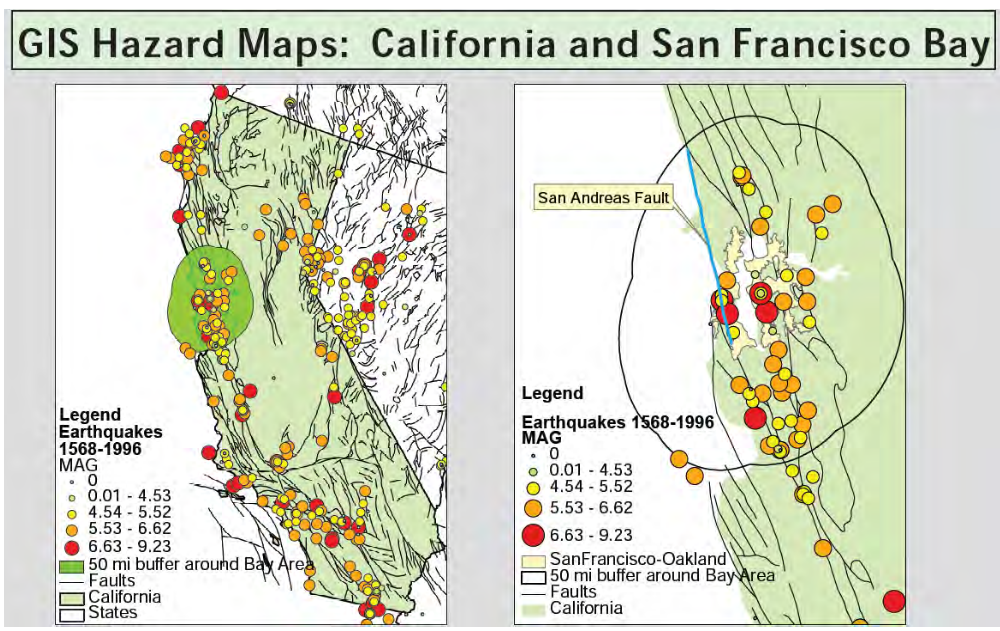
San Francisco 1906



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Pile Drag Load and Downdrag in a Liquefaction Event

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Abstract: Sandy soil layers may undergo compression during liquefaction. Areview of published design manuals, including the *2004AASHTO LRFD Bridge Design Specifications,* indicates that some recommendations for pile design may not represent the pile response in a manner consistent with the actual axial response of the pile during liquefaction. The actual response is discussed in light of the unified pile design method and separated between liquefaction occurring above and below the static nonliquefied neutral plane location before the liquefaction event. In the former case, the effect on the pile is minor regardless of the magnitude of liquefaction-induced settlement of the surrounding soil. In the latter case, the axial compressive load in the pile increases and additional pile settlement (downdrag) will occur when the force equilibrium is reestablished through the necessary mobilization of additional toe resistance. This means that the magnitude of the downdrag is governed by the pile toe load-movement response to the downward shift of the neutral plane. While there is a reduction in shaft resistance due to the reduction in strength within the liquefied layers, this reduction will only influence the pile design length where the liquefying layer is very thick.

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Introduction

Sandy soil layers may undergo compression during liquefaction (Tokimatsu and Seed 1987; Ishihara and Yoshimine 1992). This compression results in a downward movement of the overlying soil layers. For pile foundations, the movement may influence the distribution of the axial load distribution in the pile, notably the magnitude of the drag load and the location of the force equilibrium in the pile—i.e., the neutral plane. Depending on the site conditions, the computed change in axial load resulting from liquefaction-induced settlement can have a significant impact on the pile design and foundation costs for projects in seismically active regions. Liquefaction is addressed in a few recently published design manuals, such as the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2004) and AASHTO based state highway documents (e.g., MoDOT 2005; WSDOT 2006). The AASHTO specifications recommend adding the factored drag load from the soil layers above the liquefying layer to the factored loads from the structure and requires the factored shaft resistance in the soil layers below the liquefying layer plus the factored toe resistance to be equal or larger than the combination of the factored loads mentioned. However, the AASHTO specifications do not recognize that a drag load is typically present in the pile prior to the earthquake (Fellenius 2006) and that, if the load applied to the pile would cause it to move downward relative to the soil, the drag load is eliminated. In the writers' opinion, the AASHTO specifications (2004) concept of designing for drag load is fundamentally flawed. Indeed, the treatment of liquefaction-induced drag load on piles, as presented in the AASHTO specifications, can have a substantial ramification on foundation costs.

The AASHTO specifications (2004) only recognize the development of drag loads where significant settlement occurs, defined as 10 mm, and computes the required geotechnical resistance as the sum of the drag load plus the sustained and transient structure loads. This is in conflict with the approach of the *FHWA Manual* (Hannigan et al. 2006), which is "FHWA's primary reference of recommended practice for driven pile foundations" and other enlightened codes and standards, such as the *Canadian Foundation Engineering Manual* (CGS 1992), the Australian Piling Standard (1995), and the Hong Kong *Foundation Design and Construction* manual (Hong Kong Geotechnical Engineering Office 2006). The latter four documents recognize that the appropriate design conditions for drag load and downdrag are: (1) use of the shaft resistance along the entire pile length in determining the geotechnical axial capacity of the pile; (2) calculation of the maximum axial compressive load at the neutral plane (which is affected by sustained load and drag load) to determine the pile's required structural axial strength; and (3) computation of the pile downdrag as the settlement of the soil at the pile's neutral plane. This approach is termed "the unified pile design" (Fellenius 1984, 2004). The authors' recognize that there are other failure modes, e.g., buckling, in addition to the axial pile conditions that should be considered in a comprehensive pile design.

Review of Terms

Because of the complexity of the concepts involved, it may be helpful to define the terms used herein in describing the phenomena of drag load and downdrag with respect to the structural and geotechnical axial performance of piles.

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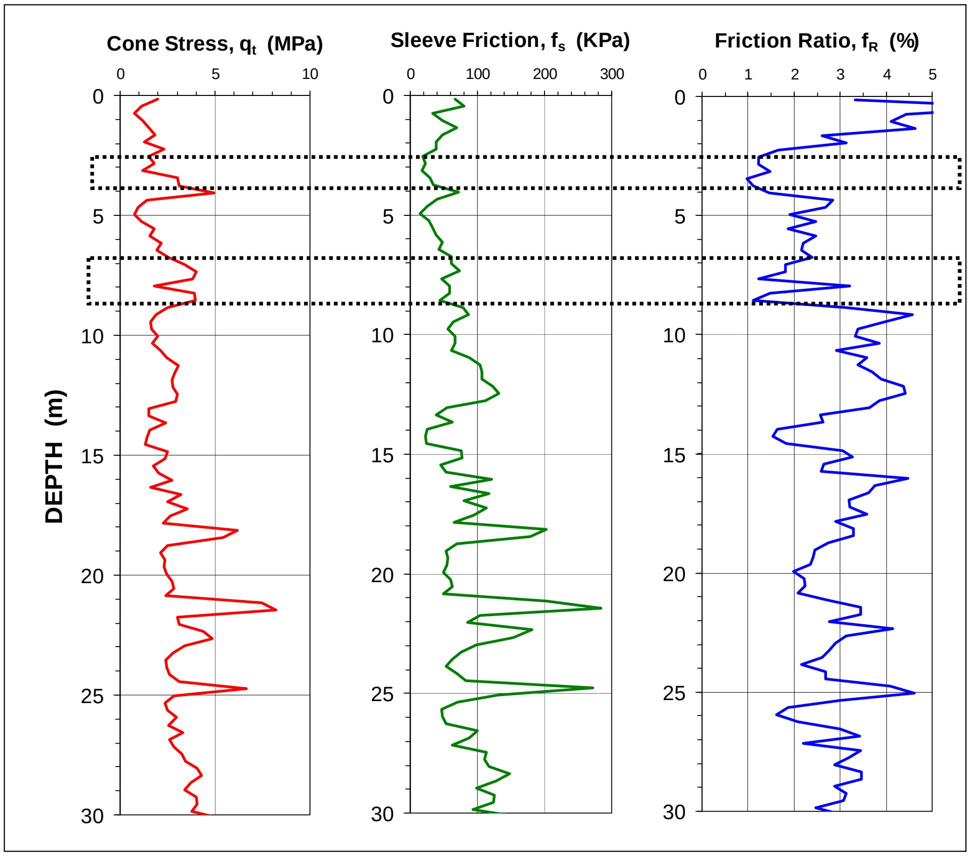

Fig. 1. CPT profile from example site. Two zones surrounded by dashed lines are identified as consisting of sand and silty sand and are considered susceptible to liquefaction during design earthquake.

Negative Skin Friction

Shaft resistance mobilized as the soil moves downward relative to the pile. Observations made during long-term field monitoring support the fact that negative skin friction develops in essentially all piles.

Drag Load

The axial compressive load induced on the pile element due to accumulated negative skin friction.

Neutral Plane

The location along the pile at which the sustained forces (i.e., drag load plus sustained structure load) are in equilibrium with the combination of (positive direction) shaft resistance (below the neutral plane) and toe resistance. This is the location where the maximum compressive load occurs in the pile. It is also the location at which there is zero relative movement between the pile and soil.

Downdrag

The downward movement of the pile due to settlement of the surrounding ground. The downdrag is equal to the settlement of the soil at the location of the neutral plane.

Geotechnical Axial Capacity

The combined shaft and toe resistances where the pile will no longer reach static equilibrium and will experience continued downward movement.

Structural Axial Capacity

The compressive axial strength of the pile section.

Example

In an effort to demonstrate the phenomena of drag load and downdrag in liquefiable soil, the effect of liquefaction-induced compression is considered for a site in northern California described by Knutson and Siegel (2006). The site is located approximately 70 km southeast of downtown San Francisco in Milpitas, Calif., and is underlain by Quaternary alluvial deposits (California Division of Mines 1951). The upper soil conditions consist of interbedded clays and sands and are represented by the cone penetration test (CPT) data presented in Fig. 1. Potentially liquefiable layers are indicated in the figure. The liquefaction potential was evaluated for a M 7.8 earthquake and a horizontal ground acceleration of *0.6g* using CPT data and the method presented by Robertson and Wride (1998) combined with the recommendations presented by Youd et al. (2001).

The effects of drag load are assessed for 460 mm diameter, augered pressure-grouted piles installed to a depth of 30 m with a

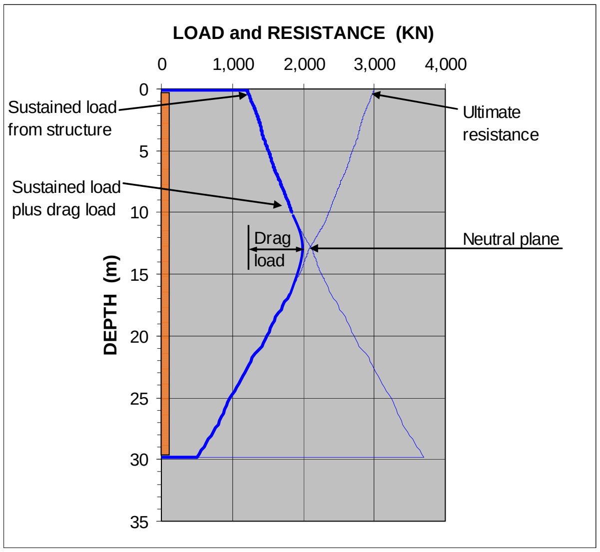

Fig. 2. Distribution of load and resistance along pile before liquefaction

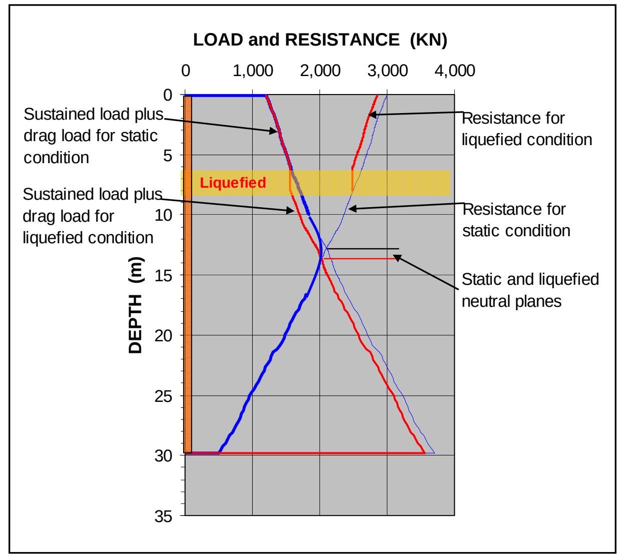

Fig. 3. Distribution of load and resistance during liquefaction above neutral plane

geotechnical capacity of about 3,000 kN and an unfactored sustained structure load of 1,100 kN. According to the AASHTO specifications (AASHTO 2004), in the absence of an earthquake, the design is not required to consider negative skin friction and drag load. In reality, negative skin friction will also develop under static conditions and accumulate to a drag load of about 900 kN at a neutral plane located at a depth of about 13 m. The load and resistance distribution curves for static conditions are shown in Fig. 2. These curves are calculated applying recommendations of O'Neill and Reese (1999) and values of N_{60} and undrained shear strength from correlations with CPT cone resistance. For this case, the curves are also approximately equal to values calculated using the Eslami-Fellenius CPT method (Eslami and Fellenius 1997).

The curves shown in Fig. 2 can only be determined using unfactored values, as factored values will distort the magnitude of the maximum axial compressive load in the pile and the location of the neutral plane. The 3,000 kN capacity and the 1,100 kN unfactored sustained structure load represent a factor of safety of 2.7. The addition of a transient load of up to 400 kN would reduce the factor of safety to 2.5, and reverse the direction of the shaft resistance (from negative to positive) in the upper portion of the pile, but it would have no influence on the pile settlement or the maximum compressive load in the pile.

Design of a pile foundation for downdrag cannot appropriately be considered in the context of geotechnical axial capacity, as it is a settlement issue. At the neutral plane, the soil and the pile move equally. Therefore, the magnitude of the settlement of the soil at the neutral plane is also the settlement of the pile—also known as downdrag. The proper design approach is to ensure that the magnitude of the soil settlement at the neutral plane is within acceptable limits or to ensure that the neutral plane is located in nonsettling soil. It is noteworthy that the location of the neutral plane depends on the magnitude of the mobilized toe resistance and corresponding toe movement.

It was determined that earthquake-induced liquefaction could occur in the sand layer between depths of about 7 and 9 m. (Liquefaction of the layer at 3 m depth is of less concern for the subject discussed here). During a liquefaction event, the sand layer would experience compression and the overlying soil layers would induce negative skin friction as they move downward relative to the pile. The unfactored drag load above this zone is about 700 kN. According to the AASHTO specifications, the drag load, factored by a load factor -y =1.25, is to be added to the factored structure loads, resulting in a total factored load of 2,250 kN. The sum of the unfactored shaft and toe resistances below 9 m depth is about equal to this load. However, applying the approach specified by the AASHTO specifications implies that the pile would not have an adequate geotechnical axial capacity in the event of liquefaction, despite actually having a corresponding factor of safety of 2.5 or better. As a consequence, longer piles would be required or the number of piles would have to be increased (to reduce the structure load per pile).

It is interesting to note that some AASHTO-based designs allow the use of reduced (residual) strengths when computing the drag load in a liquefaction event. As a result, the design depends on the decrease in strength in layers above the liquefying layer in order to maintain an acceptable load-to-resistance ratio. Because of the inherent uncertainty involved in the liquefaction prediction and soil behavior during earthquakes, this seems imprudent.

Drag Load Evaluation according to Unified Pile Design Method

The writers propose to apply the unified design method to analyze the effect of liquefaction on the behavior of piles under axial load. The load and resistance distributions in the pile when liquefaction occurs in soil above the static (or preliquefaction) neutral plane are shown in Fig. 3 for comparison to the static conditions. The effect of the liquefaction is limited to a loss of negative skin friction in the liquefied zone, and a slight reduction of the drag load and geotechnical axial capacity. No change occurs below the neutral plane and no pile movement or settlement occurs. This application of the unified design method illustrates that liquefaction occurring above the static neutral plane has a minor effect on the axial conditions of the pile.

If the liquefying layer is located below the static neutral plane, the resulting pile conditions are quite different, as is indicated in Fig. 4. The effect of the liquefaction is the lowering of the neutral plane to the lower boundary of the liquefying layer, an increase of

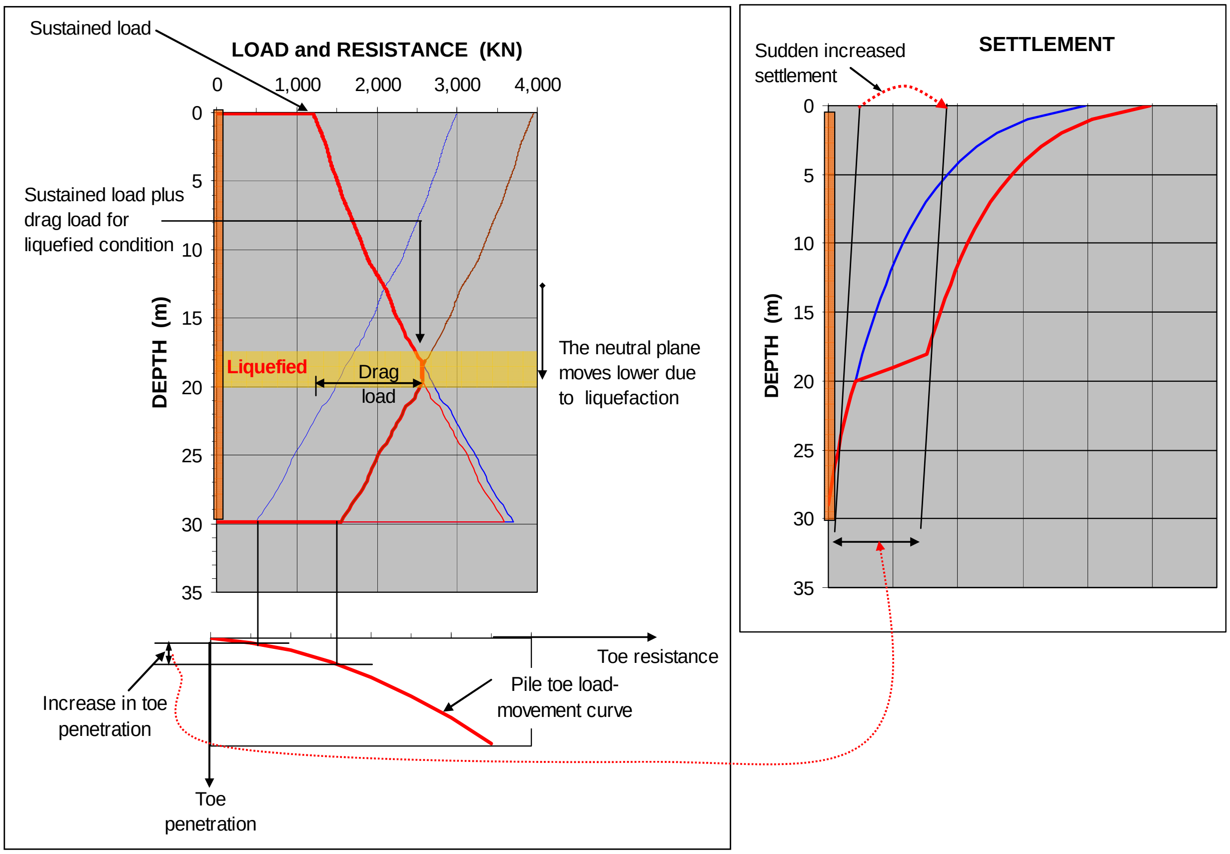

Fig. 4. Distribution of load and resistance during liquefaction below neutral plane and settlement before and after liquefaction event

the drag load, and most important, an increase in the mobilized toe resistance accompanied by an additional toe penetration. The additional toe penetration corresponds to the toe movement needed to further mobilize the toe resistance for reestablishing a force-equilibrium neutral plane at the lower boundary of the liquefying layer. If the pile toe response is stiff in providing the necessary resistance, then, the liquefaction-induced settlement of the pile may be small. Conversely, if the soil conditions are such that increased toe penetration does not provide much increase in toe resistance, then, the neutral plane will move to a location above the liquefying layer and the pile settlement will be equal to the full compression of the liquefied layer. Unless the liquefaction is so extensive that the geotechnical axial capacity (toe and positive shaft resistance along the full length of the pile with an appropriate reduction to account for the reduction in soil strength) is exceeded by the structure loads, the governing aspect of the axial design for liquefaction is the ensuing pile settlement. In the extreme, if the geotechnical axial capacity during liquefied conditions is so reduced that it is exceeded by the structure loads, then, the shaft resistance along the entire pile is positive and the problem ceases to be a drag load issue.

Discussion

The methods for the prediction of liquefaction and the design of foundations in liquefiable soil continue to evolve. Recent literature on the limitations of the use of CPT for liquefaction analysis (Li et al. 2007) and on the inadequacy of the Chinese criteria for assessing fine-grained soils (Bray and Sancio 2006; Boulanger and Idriss 2006) serve to illustrate that the available knowledge is incomplete. As a result, the tendency in the engineering community is to design with greater conservatism. It is within this atmosphere that AASHTO and other agencies have published design specifications for considering the effects of liquefaction induced settlement on the axial performance of piles. It may be hypothesized that the design approach presented by AASHTO and others is an attempt to be simple and conservative. In reality, the AASHTO design approach misrepresents the actual pile response and may lead to inappropriate design decisions.

In summary, the writers have proposed to apply the unified pile design for evaluating the influence of liquefaction-induced settlement on the axial behavior of piles that is consistent with the fundamental response of the pile in terms of movements and loads. The following conclusions have been established.

1. Liquefaction of soil layers above the static neutral plane (i.e., the neutral plane that exists prior to liquefaction) will have a minor effect on the pile regardless of the magnitude of the liquefaction-induced settlement.
2. Liquefaction of soil layers *below* the static neutral plane increases the axial compressive load in the pile and results in additional settlement. Considering this, the structural design of the pile section and pile settlement should be evaluated for liquefied conditions as part of a comprehensive pile design.
3. In the extreme, if the geotechnical axial capacity during liquefied conditions is so reduced that it is exceeded by the structure loads, then the shaft resistance along the entire pile is positive and the problem ceases to be a drag load issue
4. The construction of the neutral plane should use unfactored loads and resistances as the use of factored values will distort the magnitude of the maximum axial compressive load in the

pile and the location of the neutral plane. Transient loads have no effect on either the maximum axial compressive load in the pile or the pile settlement.

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