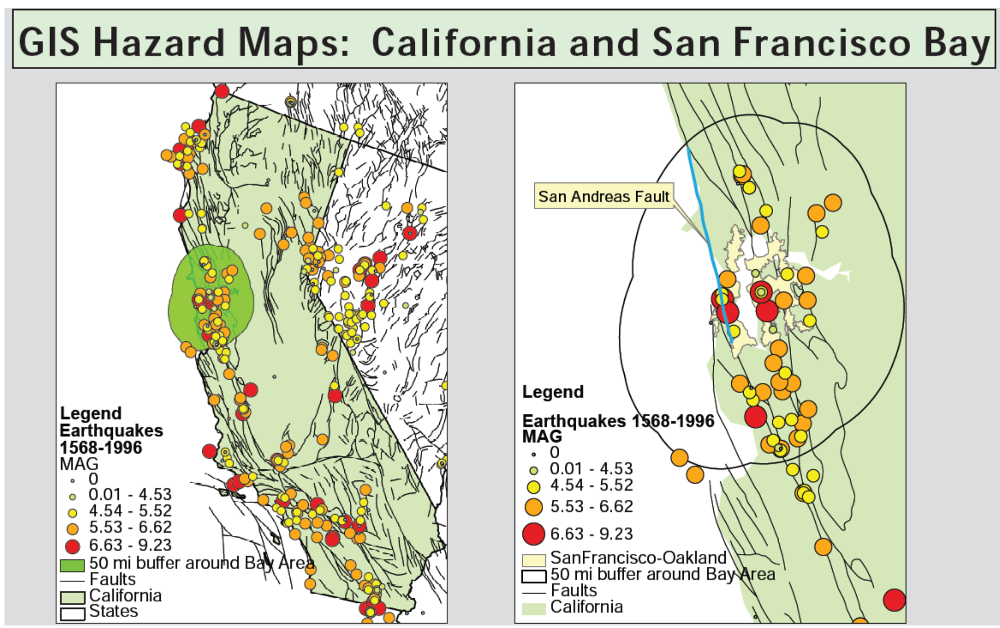
San Francisco 1906



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Pile Drag Load and Downdrag Considering Liquefaction

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Abstract

Most piles, be they driven or bored, are constructed through soft and compressible soils to competent soils. Under long-term conditions, even if the settlement in the surrounding soil is small, negative skin friction will develop and the piles will be subjected to drag load down to a so-called neutral plane — the location of the equilibrium of forces between the pile and the soil. Whatever the settlement magnitude is at the neutral plane, it is also the settlement of the pile. Therefore, the design problem to address is not the drag load, but the location of the neutral plane and the settlement of the soil at the neutral plane, as expressed in the unified pile design method.

This note addresses the special problem of sandy soils undergoing compression during liquefaction in light of the general principle of development of a neutral plane. A review of published design manuals including the AASHTO LRFD Bridge Design Specifications indicates that some recommendations for pile design in the AASHTO Specs do not represent the pile response in a manner consistent with the actual axial response of the pile during liquefaction. Liquefaction needs to be separated between that occurring above and below the static neutral plane location before the liquefaction event. For the former case, the effect on the pile due to liquefaction is minor regardless of the magnitude of liquefaction-induced settlement of the surrounding soil. For the latter case, the axial compressive load in the pile increases and additional pile settlement (downdrag) will occur when the force equilibrium is re-established 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 required pile length where the liquefying layer is very thick.

Introduction

Several well-documented case histories, a few summarized by Fellenius (1984, 2004, 2006), have shown that essentially all piles will be subjected to drag load due to accumulated negative skin friction even if the settlement of the soil surrounding the piles is very small. In fact, the more able a pile is to withstand the soil settlement, the larger the drag load and the safer the foundation. Conversely, a pile with only a small drag load in a settling soil will experience downdrag, i.e., it will settle. Indeed, "negative skin friction" is not a geotechnical capacity issue. It is necessary, however, to consider negative skin friction when computing the settlement of piles and pile groups. This is recognized in enlightened codes and standards, such as the FHWA Manual (2006), which is “FHWA's primary reference of recommended practice for driven pile foundations”, the Canadian Foundation Engineering Manual (1992, 2006), the Australian Piling Standard (1995), and the Hong Kong Foundation Design and Construction Manual (2006). These 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 plus toe resistance 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 axial structural strength, and (3) computation of the pile downdrag as the settlement of the soil at the pile’s neutral plane due to changes of effective stress in the soil surrounding the pile. This approach is termed “the unified pile design” (Fellenius 1984; 2004). In contrast, the AASHTO Specifications (2004, 2006) 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

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drag load plus the sustained and the transient loads from the structure. Design according to the AASHTO Specifications does not represent actual pile behavior. As a result, the resulting pile design may be unnecessarily costly and, as it does not address the main aspect, the settling soil, it may result in an unsafe foundation.

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 in respect to the structural and geotechnical axial performance of piles.

Negative skin friction – Shaft resistance mobilized as the soil moves downward relative to the pile. Observations from long-term field monitoring support 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 when the soil tends to move downward relative to the pile.

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 depth is also where 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 magnitude 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. Positive shaft resistance occurs along the full length of the pile and drag load is eliminated.

Factor of safety on geotechnical capacity – The ratio between the geotechnical axial capacity divided by the sum of dead load plus live load, drag load is not included.

Structural axial strength – The compressive axial strength of the pile section affected by dead load plus drag load.

Factor of safety on structural strength at the neutral plane – The ratio between the structural axial strength at the neutral plane divided by the sum of dead load plus drag load, live load is not included.

Although the issue of design of pile foundations is rather broad, this note will address the special condition of consequence for a piled foundation in liquefying soil during a seismic event.

Axial Pile Design for Liquefied Conditions and AASHTO

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 piled 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 (2004, 2006) 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 dead and live 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 mentioned factored loads. 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. Nor do the Specs recognize that live load (transient load) and drag load cannot occur simultaneously (cannot coexist!). In the authors' 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.

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, California, and is underlain by Quaternary alluvial deposits (Division of Mines 1951). The upper soil conditions consist of interbedded clays and sands and are represented by the CPT data presented in Figure 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.* (2000).

The effects of drag load are assessed for 460 mm diameter piles installed to a depth of 30 m with a geotechnical capacity of 3,000 kN (obtained from static loading test) and an unfactored sustained axial load of 1,100 kN. According to the AASHTO Specifications (2004, 2006), 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 develop also under static conditions and accumulate to a drag load of about 900 kN at a neutral plane located at depth of about 13 m. The load and resistance distribution curves for static conditions are shown in Figure 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).

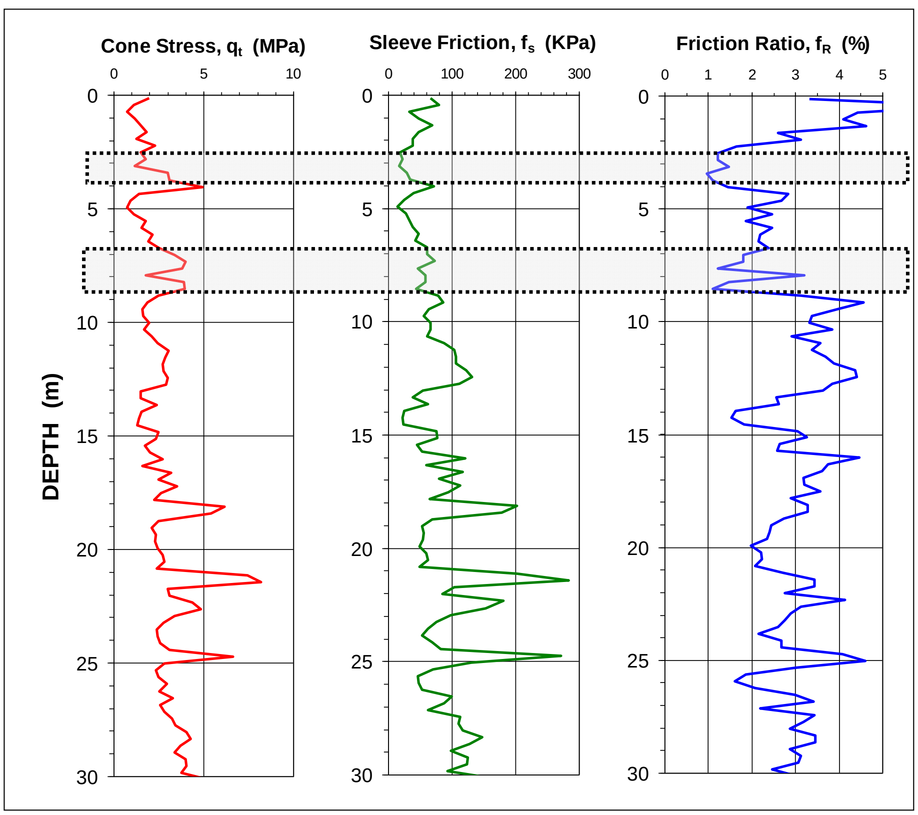

Fig. 1. CPT profile from the example site. The two zones surrounded by the dashed lines consist of sand and silty sand and are considered susceptible to liquefaction under the design earthquake.

The load and resistance distribution curve shown in Figure 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 load represent a factor of safety of 2.7. The addition of the transient load of 400 kN would reduce the factor of safety to 2.0, 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 pile settlement or the maximum compressive load in the pile at the neutral plane.

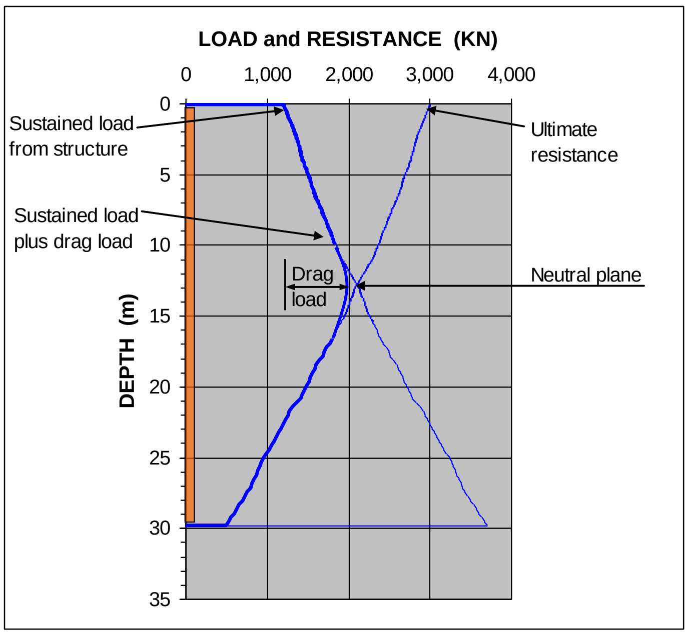

Fig. 2 Distribution of load and resistance along the pile before liquefaction

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 lies in non-settling 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 layers at 3 m depth and between depths of about 7 m to 9 m. During a liquefaction event, the sand would experience compression and the overlying soil layers would move downward relative to the pile. The unfactored drag load due to accumulated negative skin friction above 9 m depth is about 700 kN. According to the AASHTO Specifications, the drag load, factored up by 1.25, is to be added to the factored sustained and transient structure loads, resulting in a total factored load of 3,060 kN after applying specified load factors on sustained and transient loads of 1.35 and 1.75, respectively (one could argue the actual factors and which AASHTO Specs edition to apply, and including the transient load along with the drag load is incorrect, the two cannot occur simultaneously, but that is beside the point here,). The sum of the factored shaft and toe resistances below 9 m depth is smaller than this load. Indeed, already the unfactored resistances are smaller. Therefore, the approach specified in the AASHTO Specifications implies that the pile is severely under-designed in the event of liquefaction. As a consequence, longer piles would be required (to increase capacity), or the number of piles would have to be increased (to reduce the sustained load per pile). However, the liquefying layer lies above the neutral plane, and the shaft shear down to 13 m depth is in negative direction before the liquefaction occurs. Therefore, as discussed below, the liquefaction will not change the forces in the pile and soil, nor cause the pile to settle. The factor of safety is only marginally affected by the small reduction of shear strength in the liquefying layer. Indeed, there is no need for increasing pile length or number of piles.

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 the Unified Pile Design Method

The authors propose to apply the unified design method to analyze the effect of liquefaction on the behavior of piles under axial load (Fellenius and Siegel, 2008). The load and resistance distributions in the pile when liquefaction occurs in soil above the static (or pre-liquefaction) neutral plane are shown in Figure 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 minor effect on the axial conditions of the pile.

Indeeed, in the event that the earthquake triggering liquefaction occurring in the soil layer located above the eventual neutral plane before the neutral plane has developed, then, the effect would be limited to speeding up the development of the neutral plane.

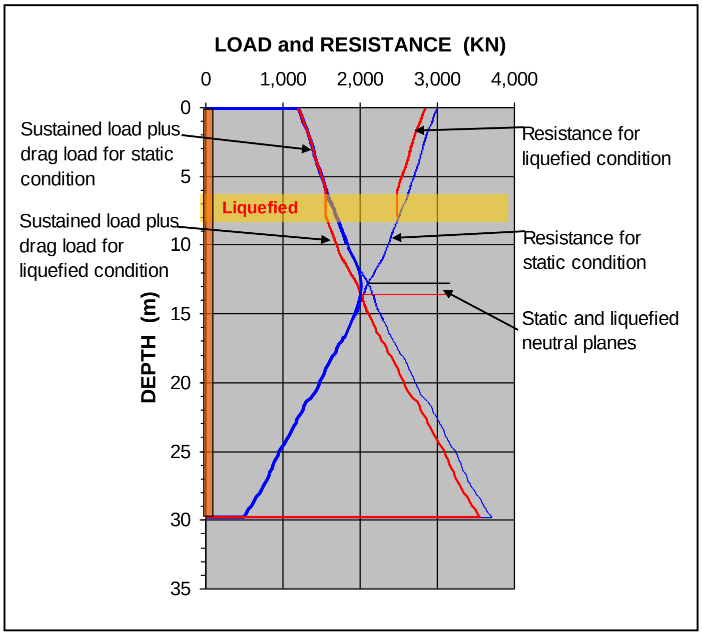

Fig. 3 Distribution of load and resistance during liquefaction above the neutral plane

If, on the other hand, the liquefying layer is located below the static neutral plane, the resulting pile conditions are quite different, as is indicated in Figure 4. The effect of the liquefaction is the lowering of the neutral plane to the lower boundary of the liquefying layer, an increase of the drag load and, most important, subsequent toe penetration as necessary to mobilize additional toe resistance required to re-establish 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 immediately 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 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 sustained loads from the supported structure, then, the shaft resistance along the entire pile is positive and the problem ceases to be drag load issue. However, the pile will fail and the "supported" structure will suffer.

Discussion and Conclusions

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 Sanction 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 authors 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.

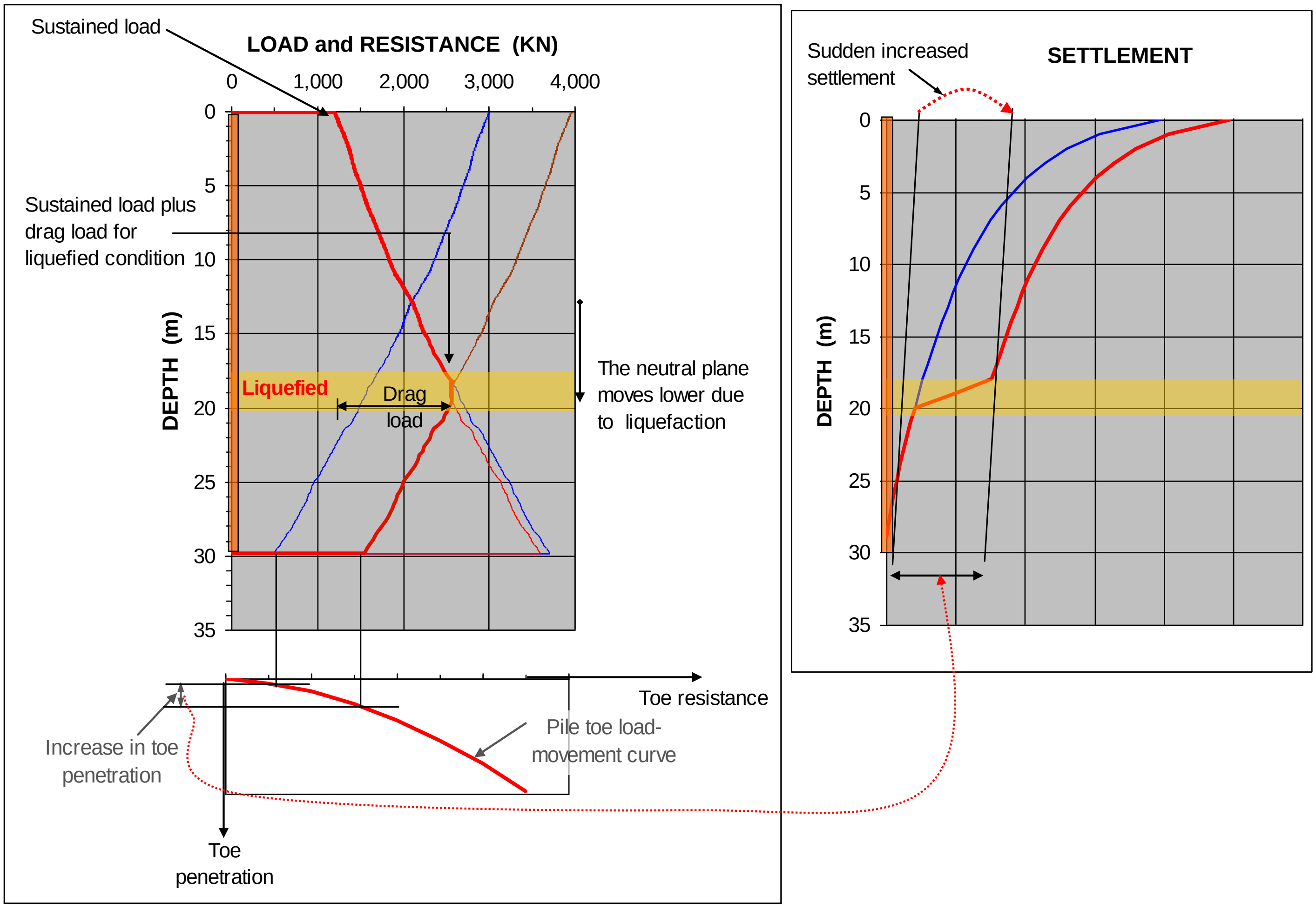

Fig. 4 Distribution of load and resistance during liquefaction below the neutral plane and settlement before and after a liquefaction event.

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 of the soils above the liquefying layer.

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 sustained loads from the supported structure, then, the shaft resistance along the entire pile is positive and the problem ceases to be 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. (Note, transient loads neither affect the location of the neutral plane, the maximum axial compressive load at the neutral plane, nor the pile settlement).

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