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Discussion on Bitumen Selection for Reduction of Downdrag on Piles (J-L. Briaud, 1997 ASCE Journal of Geotechnical Engineering, Vol. 123, No. 12, pp. 1127-1132)

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The author has addressed the pile foundation engineering quandary of negative skin friction by reducing it with a bitumen coat applied to the pile surface. Unfortunately, the author's introductory example intended to portray the need for the bitumen coat is considerably off-base.

The author's example consists of a 300 mm diameter square concrete pile with an embedment of 30 m in a homogeneous soil with a constant pile-soil interface shear resistance of 25 kPa (unit shaft resistance and negative skin friction are taken as equal). The author uses the assumed shear value in a total stress analysis obtaining a pile shaft resistance of 900 kN. The stated reason for using a constant unit shear is "to keep the example simple". However, total stress analysis of pile shaft resistance is not realistic. Instead, the shaft resistance should be derived from an effective stress analysis. The same 900 kN capacity can be obtained in an effective stress analysis by introducing a few simple assumptions: a groundwater table at the ground surface, a hydrostatic pore pressure distribution, a total saturated density of 1,700 kg/m^3, and a Bjerrum-Burland beta-coefficient of 0.25. This analysis is physically correct—realistic—because pile load-transfer is governed by effective stress, not total stress.

As to pile toe resistance, the author simply states that it is equal to 1,000 KN. A pile will not obtain this magnitude of toe resistance in a soil that has a shear strength of only 25 kPa. Therefore, although the author does not state this, the example must be from a pile that is bearing in a competent soil at the pile toe (and having some embedment into this soil, say, at least 3 pile diameters, or about 1 m). The 1,000-kN toe resistance corresponds to a toe bearing capacity coefficient slightly in excess of 50, which is representative for a competent granular soil. Such a soil is not particularly compressible.

The author gives the distribution of settlement as 200 mm at the ground surface, reducing almost linearly to zero at the pile toe. However, a homogeneous soil, that is, a soil of inherently constant compressibility, does not have anywhere near linear settlement distribution. Instead, settlement reduces invariably and progressively with depth (as shown, for example, by Clemente, 1979). If the load assumed to generate the 200-mm settlement is from a uniformly distributed 0.5 m thick fill on the ground (imposing a stress of 10 kPa), and the soil is normally consolidated with a compression ratio, CR, equal to 0.085 (or a modulus number of 27, or a compression index, C_c, of 0.2 with a void ratio of 1.4), then, the calculated settlement at a depth of 20 m is a mere 14 mm, not the 50 mm indicated by the author. If the fill area would be limited in size, the subsequent reduction of stress with depth (Boussinesq distribution) would have resulted in an even greater reduction of the settlement with depth. To obtain an approximately linear settlement distribution, the compressibility of the soil would have had to diminish by a factor of 20 between the ground surface and the depth of 30 m., which is not realistic.

Fig. 1 presents two diagrams illustrating the errors resulting from the author's simplification of assuming linear distribution of unit shear along the pile-soil interface and combining this distribution with a practically linear settlement distribution. The diagram to the left shows the load and resistance distribution curves and the construction of the neutral plane. The curved, thick lines show this construction when determined in an effective stress analysis and indicate a neutral plane a depth of 26 m. The straight, thin lines show the results of the author's construction and indicate a neutral plane at a depth of about 23 m.

The diagram to the right in Fig. 1 shows the distribution of settlement. The curved, heavy line is determined from assuming constant compressibility and uniform distribution of the fill. The approximately straight, thin line shows the author's distribution of settlement. The calculations behind the graph and the graph itself were produced by the UniPile program (Fellenius and Goudreault, 1997; details are available on Internet address [www.unisoftltd.com]. For a case as simple as this, a hand-calculation would have been almost as expedient.

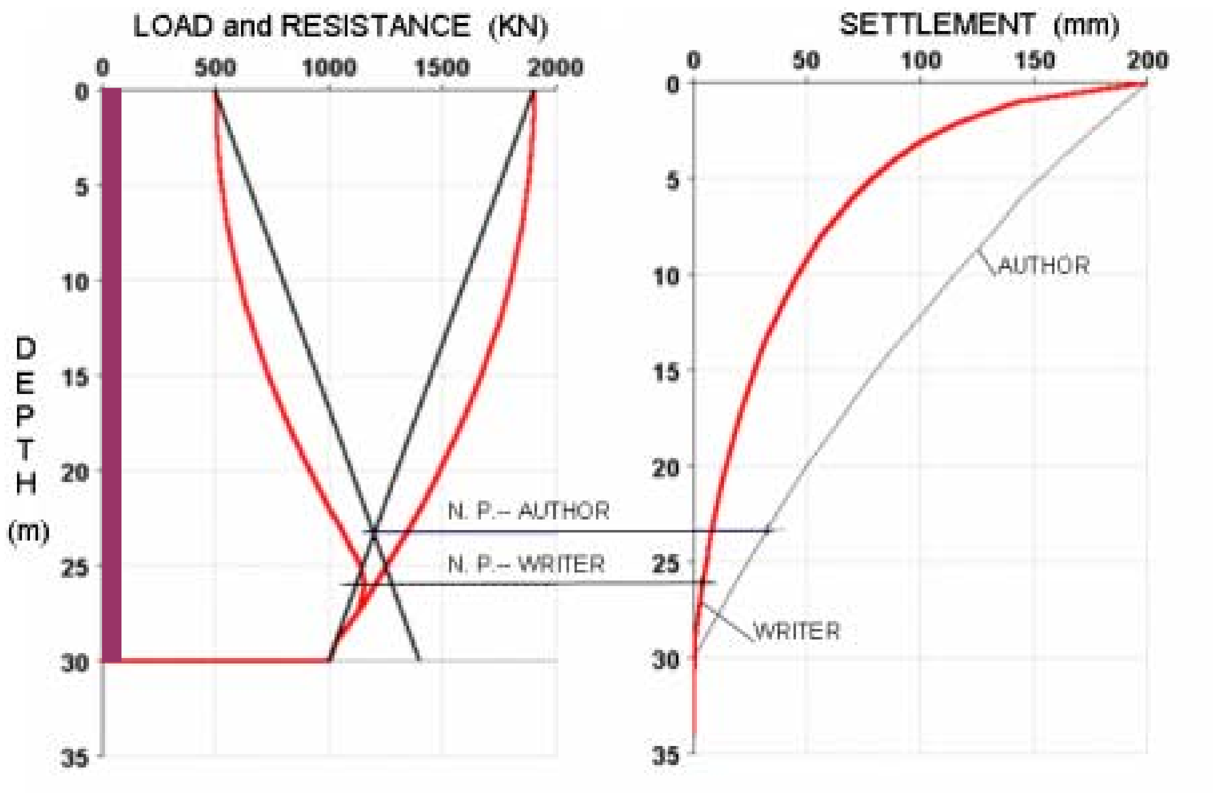

Fig. 1 Load and Resistance Diagram and Distribution of Settlement

By definition, at the neutral plane no relative movement occurs between the pile and the soil. That is, the settlement of the pile at the neutral plane is equal to the soil settlement at that depth. The settlement of the pile head, and, therefore, the settlement of the structure supported by the pile, is the sum of the settlement at the neutral plane and the compression of the pile for the dead load above the neutral plane. The author's example indicates a settlement of about 35 mm at the neutral plane, whereas the writer's analysis suggests a settlement of only about 2 mm.

The author suggests that the analysis should include the load-movement relation of the pile toe. This analysis affects the location of the neutral plane inasmuch that if the full toe resistance is not mobilized, the neutral plane is positioned higher up in the soil (where the settlement is larger), as opposed to when the resistance is fully mobilized. However, if adding this aspect, one must also include the effect of residual load. Residual load is imposed by the pile installation and develops further with time. By the time the dead load is applied, which is when any settlement starts to be of concern, much of the toe movement and the residual toe load have occurred. Therefore, this detail has little relevance for most cases.

Note also that if the soil settlement below the pile toe is zero and the full toe resistance is not mobilized, the pile settlement will be essentially only consist of shortening due to the applied dead load, which normally is negligible. The shortening is generated by the dead load alone, the compression caused by the drag force occurs earlier and can be disregarded. The compression for the 500 kN load assuming the Young modulus of the pile is as low as about 35 GPa is only about 3 mm, a negligible amount—the author states a three times higher value.

Thus, in direct contrast to the results of the author's analysis, the settlement of the foundation supported by a group of piles similar to the example pile will not be remarkable and no action to reduce it is required. Moreover, the maximum load in the pile, that is, the sum of the dead load and the drag force would be at most 1,200 kN corresponding to a maximum stress of about 1,300 kPa, which is about equal to or smaller than 0.25 times the axial strength of the pile. This would be true also after the increase of the drag force associated with the consolidation (the increase of effective stress that produced the settlement; note, the pile capacity would also increase). Therefore, the pile is structurally capable of accepting the design load.

The author uses the term “downdrag” interchangeably with “dragload”. However, while “dragload” is the load generated in the pile when the soil settles around the pile, “downdrag” is the term indicating that the pile is dragged down by the soil. In the former case, the pile movements are small and, if the pile can resist the load structurally, the dragload prestresses the pile and is essentially beneficial to the foundation. In contrast, “downdrag” is associated with small dragload and is a settlement condition, usually very undesirable. Just consider the consequence if the toe resistance of the example pile were to be a mere 100 KN. This would mean that the pile toe is located in a weak and compressible layer and that soil settlement would also occur below the pile toe as an effect of the stress increase due to the surcharge on the ground surface ***and*** the load on the pile group. The factor of safety would still be adequate, 2.0, and the dragload would be only about 200 KN. The neutral plane would be located at a depth of 15 m where the soil settlement would be about 30 mm. The settlement in the soil below the pile toe adds to this value. Consequently, the structure supported by a group of the piles would settle appreciably, that is, be subjected to a considerable downdrag.

Of course, downdrag and dragloads are important aspects to include in a geotechnical design and they are neglected at one’s peril. Excessive settlement may develop for pile foundations on shaft bearing piles with pile toes in compressible soils, where settlement develops due to the load on the piles (dead loads) and/or due to increases of effective stress (caused by lowering the groundwater table or placing a fill on the ground; the dragload does not cause settlement). Long toe-bearing piles may experience excessive stress at the neutral plane. “Long” in this context means piles longer than about 25 m.

When conditions warrant and associated costs are acceptable, a coat of bitumen will reduce the negative skin friction (as well as positive shaft resistance). The author mentions that the cost of applying a bitumen coat to the pile lies in the range of 15 % to 50 % of the cost of the uncoated piles. These numbers agree with the writer’s experience that the cost of bitumen coating piles are about 10 % to 15 % of the ***in-place*** cost of the piles. Such costs are not negligible and warrant a careful study to really demonstrate that bitumen coating, or other measures, really are technically justified. As illustrated above, an overly simplified analysis can lead to expensive solutions to a not-so-genuine problem.

The author summarizes several aspects of the behavior of bitumen in shearing and reports results from tests in the laboratory and field, presenting basic parameters of the investigated bitumen coats. In the writer’s experience, the most useful parameter is the penetration value (ASTM D5-86). The bitumen types reported by the author have penetration values ranging from 20 through 55. The results presented by the author indicate that the bitumen types with higher penetration values were also the types that best reduced the shear between the pile and the soil. This is what one should expect. Full-scale field tests and laboratory tests (Johannesen et al., 1965 and 1969, Walker and Darwall, 1973, Clemente 1979 and 1981, and Fellenius 1975 and 1979) show that a coat of bitumen, no thicker than a mere sixteenth of an inch (1 mm to 2 mm) of a penetration equal to or higher than about 80 - 100 will significantly reduce the shear between the pile surface and soil at actual rates of relative movement between pile and soil. The reported values ranged from insignificant to close to but below 20 KPa. The author’s results show that a bitumen more viscous than these values may not adequately reduce the negative skin friction. It is not a matter of achieving, as the author states, a reduction of the shear to 10 % of the soil shear strength because no direct correlation exists between the bitumen parameters and soil shear strength values.

However, instead of reaching the conclusion that the bitumen types used in the research were not suitable, the author recommends various measures to make these bitumen types work. As the recommendations are based on the assumption that the recommended approach for selecting the bitumen type is correct, the work leads to an unrealistically thick coat—a 10-mm coat of bitumen is far too thick. Not only will it be expensive, it will often be impossible to achieve without resorting to special efforts, such as substituting it with a very viscous bitumen. The latter may not have the desired effect of reducing the dragload. Moreover, the risk that sheets of bitumen will spall off from the pile during the driving increases progressively with the thickness of the bitumen coat. The writer can testify to the enthralling experience of seeing a sheet of bitumen spalling off the pile surface some 100 ft up in the air and come whipping toward him, as well as to the strong simultaneous disquieting thoughts directed toward the potential decapitational consequence of the event.

The author presents a study of gravel penetration into a layer of bitumen. However, the problem with coarse soil in contact with the bitumen lies not with the localized penetration of individual pieces into the coat with time, but with the risk of the bitumen being scraped off during driving through coarse-grained soil layers. The author reported that this was not observed for the studied cases. The writer has experienced, however, that a few metre thick surficial layer of gravelly sand can be very effective in removing a bitumen coat. A coarse-grained soil will grab the bitumen and may increase the shaft resistance during driving. Bitumen-coated piles driven through such layers would normally meet little toe resistance in the layers below. The author's treatment of the shear strain in the bitumen during driving does not consider the tension that could develop in a pile during such driving conditions. Chapman et al. (1991) report a case where pile damage during driving of precast concrete piles occurred for this very reason and, where the bitumen-coated piles that did not break required much prolonged driving as opposed to non-coated piles.

The author's "tips for proper bitumen coating" are useful. However, changing a job timing from summer to winter driving for reasons of having a bitumen unsuitable for use during a hot summer is not a practical solution. The presentation of viscosities and strain rates are interesting, but the material obscures that while the bitumen coating is a simple, as well as an assured and proven engineering solution to the problem of downdrag and excessive dragload, it is costly and its implementation can be awkward. The selection of bitumen is not the problem. A thin coat of the commonly available Asphalt Grade 85 - 105, heated and brushed onto the pile surface, will be sufficient for most cases. The main problem with bitumen coating, apart from high costs, is making sure it adheres to the pile surface (a primer is normally necessary) and that it is not scraped off in surficial coarse-grained soil layers. The potential prolonging of the pile driving and development of damaging tension must also be considered.

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