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DISCUSSION

Discussion of "Plugging effect of open-ended piles in sandy soil"[1]

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The authors have presented an interesting account of a full-scale study of the response of a pipe pile driven with an open toe to the presence of an inside soil column in the pile (Ko and Jeong 2015). The tests consisted of dynamic measurements during the driving and static loading tests some time after the driving on double-wall pipe piles. Strain-gage measurements were used to determine the load distribution.

The double-wall test piles were fabricated with the inside pipe centered in relation to the outside pipe. The steel cross section of the outside pipe was about 10% smaller than the inside pipe. The about 40 mm "gap" or void between the outside of the inner pipe and the inside of the outer pipe was welded closed at the pile toe to prevent soil from entering the "gap". The welding resulted in a firm and fixed connection between the two pipes, ensuring that the lower ends of the two pipes moved in unison in the tests. I understand that no such fixed connection was made at the pile head. However, the lengths of the pipes were the same, which means that a plate placed on the pile head rested on both pipes enabling the upper end of both pipes to be engaged approximately simultaneously. Presumably, the dynamic gages (accelerometer and strain-gage pairs, I assume) were placed on only the outer pipe and the force delivered to the pile by the hammer impact was determined from the sum of the two pipe areas as based on the assumption of perfect connection to the pile head plate. Because the test pile was made up of a pair of pipes welded together at the lower end, I would expect that the dynamic gages will have recorded considerable reflections during the tests. I would not have expected a good correlation between the CAPWAP-determined capacities (Table 2) and those determined by the offset limit method from the static loading tests (fig. 11). It would be interesting if the authors could provide details of the pile and soil models employed in the dynamic analyses.

As reported by the authors, development of an inside soil column and plugging during the driving of open-toe pipe piles has been addressed by several researchers. All depict the forces acting on the pipe pile during driving as similar to that shown in the authors' fig. 2, i.e., with upward-pointing shear force vectors both along the outside and inside of the pile and the inside vectors shown along the full length of the inside column. That is, the vectors indicate the forces as acting on the pipe and not on the core. The suggestion is that the open-toe pipe is forced down over the inside core.

The force vectors also show soil forces acting on the steel pipe pile both at the base (Q_b) and along the inside of the shaft (Q_m), but the response cannot be both, it must be one or the other. That is, if the pile experiences a toe resistance, it has a rigid plug and there is no inside shaft shear (but for along a very short length of that rigid plug). If the pipe slides down over the core, there is inside shaft resistance, but no toe resistance. Apart from this minor misrepresentation, the figure represents the typical response *during driving*. The authors' measurements show that the length of the inside soil column increased throughout the driving of the test piles. In driving, therefore, shaft resistance along the inside of the pipe can be assumed to be mobilized along the full soil column length as indicated in fig. 2. The full picture is a complex combination of shear forces, wave travel, wave reflections, and inertia, which I will not attempt to discuss here.

However, the response of the pile to a static force is very different to that shown in fig. 2 and is more similar to what I show in Fig. D1. In static loading, the pipe is pressed down engaging shaft resistance along the outside and toe resistance on the annulus area, the relatively small area of the steel pipe wall. The inside column — the core — is made to follow the downward movement, but the movement meets resistance at the pile toe, which generates a base force that compresses the core and causes a relative movement between the inside wall and the core. That relative movement only acts along a distance represented by the length of the core compressed by the total base force, the length necessary to "spend the force" in a spring action with the compression of the core being equal to the toe movement.

The actual load values determined in the static loading test reported by the authors are impaired because of the interaction between the outside and inside pipes caused by the welding the pipe together at the lower end. This fact becomes obvious in Fig. D2, which combines the authors' pile-head load–movement curves for pile 2 (fig. 11) with the loads separated on the outside and inside pipes measured at depths of 1.9 and 3.7 m, respectively (figs. 12*a* through 12*f*). The sum of the outer and inner pipes should be about equal to the applied load (curve labeled "Head both pipes"), but they are not. As can be expected from the response of the inner pipe, no change of resistance is likely to have developed between the pile head and the first gage level. In contrast, between the pile head and the first gage level in the outer pipe, an extrapolation indicates that up to 80 kN might have developed as shaft resistance along the outer pipe before the 1.9 m gage level. I believe the indicated about 400 kN difference between the sum of the outer and inner records of load and the 2000 kN applied load is due to the interaction between the two pipes, as follows.

At the 2000 kN maximum applied test load, figs. 12*c* and 12*d* indicate the loads measured at the first gage level in the inner and

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Fig. D1. Force vectors in a static loading test on an open-toe pipe pile with inside soil column.

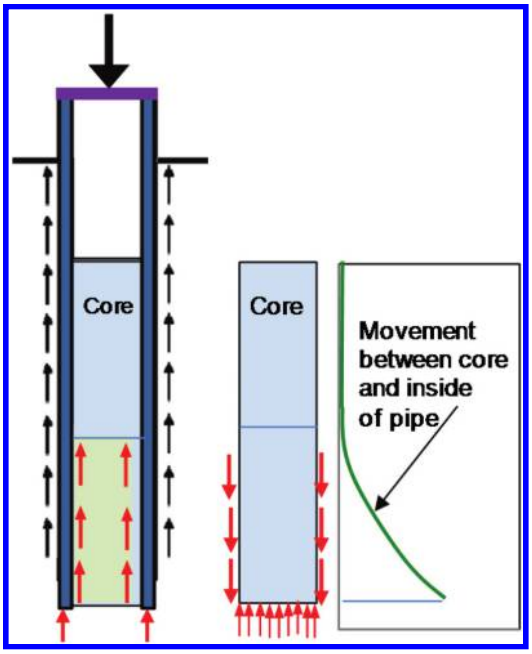
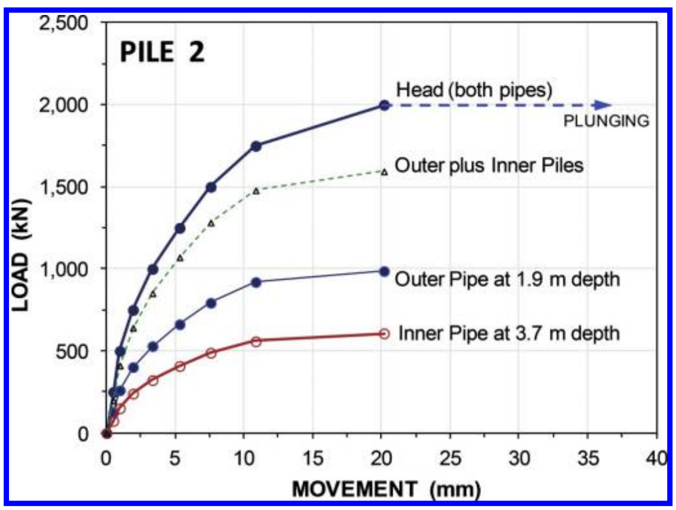


Fig. D2. Pile 2: Load–movement curves for pile head and for gage depths at 1.9 and 3.7 m for the outer and inner pipes, respectively.



outer pipes were 610 and 990 kN, respectively. As the test arrangement theoretically ensured that the original equal length of the pipes was maintained, the imposed strain should be about equal for the two pipes (disregarding the shaft resistance acting along the outer pipe to the 1.9 m depth). Thus, the fact that the loads were converted from measured strain means that the differences suggest that the cross section of the inside pipe would be about two-thirds of the cross section of the outside pipe. Yet, it was actually about 10% larger. Moreover, the tension loads reported for the pile 2 outside pipe (fig. 12*d*) were, in my opinion, caused by the unfortunate lower-end pipe connection. In its passage down the pile, the load applied to the outer pipe had to overcome shaft resistance, whereas the inside pipe met no resistance until encountering the effect of the core compression. Therefore, the total compression of the outer pipe was larger than that of the inner pipe. However, as the movements at the pipe ends (upper and bottom) were forced to stay equal, the result was that the compression measured at the lower strain gage level in the inner pipe increased by the amount necessary to suit the common length, the outer pile forced the inner pipe to compress. Either the sealing of the "gap" between the pipes should have been arranged to allow the outer pipe to slide past the inside pipe with no possibility of transferring force between the pipes, or the inner pipe should have been free at the upper end so that all applied load would have gone to the outer pipe. (While a disconnect at the lower end would have made for more reliable static load measurements, I realize that the dynamic test would then have become more complicated.)

While the loads reported from the static tests are questionable as to accuracy, the main observation and conclusion from the static tests is valid, viz., as mentioned by the authors, the measured load distributions of the inside pipes (figs. 12*a*, 12*c*, and 12*e*) show that the load was more or less constant in the upper length of the piles and inside shaft resistance only started to reduce the load along the lower 1.3 to 2.3 m length of the core. These results agree with the results presented by Paik et al. (2003) who reported similarly arranged static tests on a strain-gage instrumented, double-wall pipe pile, driven to 7.0 m depth into a compact gravely sand containing no fines and subjected to a static loading test.

Figure D3 shows the distribution of load in the inner pipe confirming that the conclusion from the test by Paik et al. (2003) as to the shaft resistance along the soil core inside an open-toe pipe pile only develops along a short distance up from the pile toe. For this test, the seal of the "gap" between the two pipes was made with a silicon compound that allowed the two pipes to slide past each other. Moreover, the test was arranged with strain gages measuring the load applied to each pipe at the pile head separately at the ground surface; the sum of the loads in each pipe was equal to the total load applied.

No leap of imagination is needed to realize that the core can be modeled as a pile turned upside down or, perhaps, as the upward portion of a pile tested in a bidirectional test. The key point to realize is that such a core pile is soft in relation to a real pile. Its axial deformation modulus, *E*, is about equal to that of soil, albeit compressed under a confined condition. The stiffness of the core is, therefore, about 3 to 4 order-of-magnitudes smaller than that of a real pile of the same diameter. Moreover, as indicated by O'Neill and Raines (1991), the effective stress in the core is constant (uniform material is assumed). Therefore, the ultimate unit shear resistance between the core and inside of the pipe is more or less constant and modeling the shear force distribution along the core should be by means of average shear force; by total stress analysis so to speak. (In contrast, the shaft resistance along the outer pipe, of course, must be modeled using effective stress principles).

In modeling the core as a soft pile pushed upward a distance equal to that of the pile toe movement in a static loading test, with the toe force compressing the core, we can appreciate that the imposed movement can never result in a large force at the bottom of the soft core and that the base force will have been "spent" within a short distance up from the core bottom. Although, the force–movement response of the core — unit shear resistance along the inside of the pipe — is more or less an elastic–plastic response, combined with the gradual mobilization of the core length, the response is similar to a pile toe response, i.e., an almost linear or relatively gently curving, force–movement of a pile toe. The difference is the magnitude of toe force and the stiffness, i.e., the slope of the curve.

The customary approach to analyzing the effect of the core in an open-toe pipe pile, as opposed to that of a closed-toe pipe pile, is to resort to a capacity comparison. However, this approach will not address the real difference between the two pile types: a soil core inside the pile results in a soft toe response, as opposed to the much stiffer toe response for the closed-toe pipe pile. Moreover, if a comparison is made for cases at different magnitudes of pile toe movements, obviously no apple-to-apple correlation exists.

Fig. D3. Load distribution in a 7 m long open-toe, double-wall pipe pile (data from Paik et al. 2003).

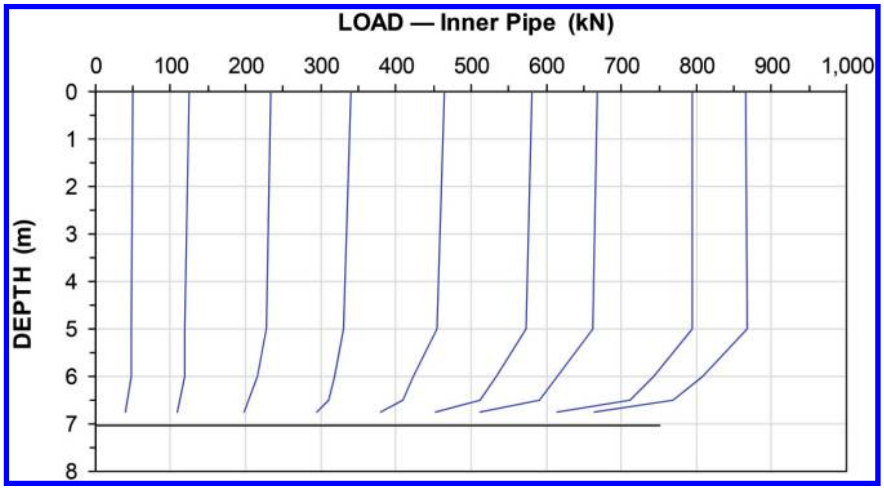


Fig. D4. Load–movement curves for a static loading test on a pile similar to the outer pipe of pile 2: (*a*) test on open-toe pipe pile and (*b*) test on closed-toe pipe pile.

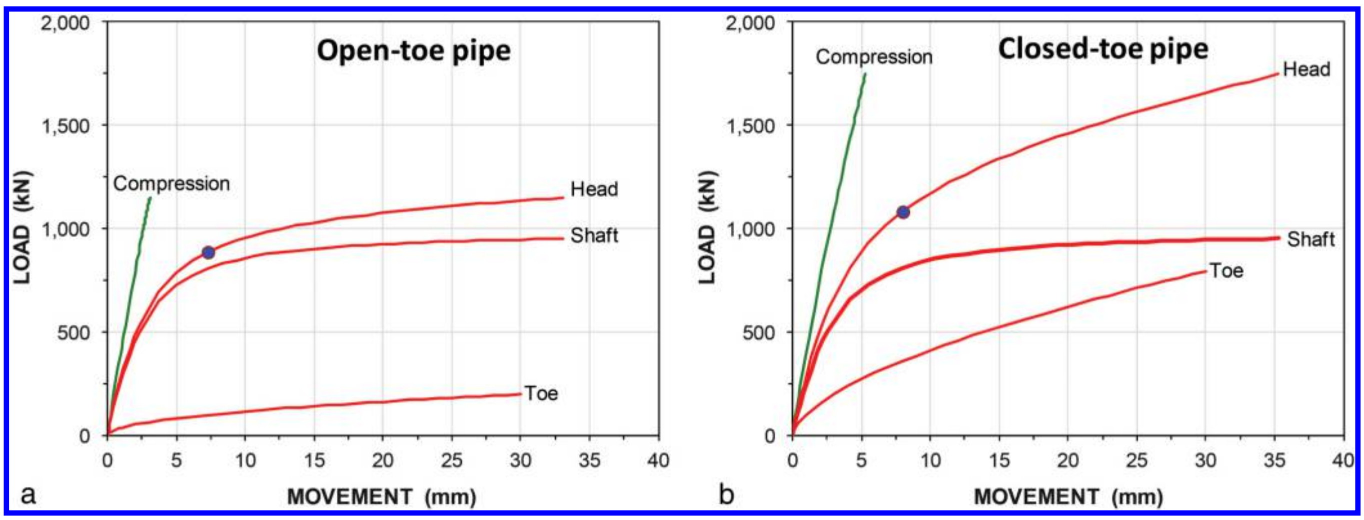


Figure D4 shows a simulation of an open- and closed-toe pipe pile of dimensions similar to the outer pile of the authors' pile 2 (outer dimater 711 mm, wall 7 mm, and length 11 m) driven into a sand similar to that of the case history. The simulation is made using UniPile software (Goudreault and Fellenius 2013) using an average beta-coefficient of 0.40 and a toe resistance of 2 MPa. The shaft response is described by a hyperbolic function with the 0.40 beta-coefficient shaft resistance occurring at a relative movement of 5 mm between the shaft and the soil and the shaft resistance at 2.5 MPa. The toe resistance is described by a ratio function with an exponent of 0.600 and the 2 MPa toe-resistance mobilized at 30 mm toe movement (Fellenius 2014).

For the simulated load–movement curves for the pile driven with an open toe, I have assumed that after the driving, a soil core exists inside the full length of the pipe. The outer shaft resistance is the same as for the closed-toe case. Moreover, I have assumed that the shear force between the core and the pipe has been activated along a 2.5 m length, that the average shear force is 40 kPa, and that the core has an *E*-modulus of 50 MPa. This establishes that, for a toe movement of 30 mm, the toe force is about 200 kN. With a bit of allowance for the force on the steel wall (the 7 mm annulus of the pile; the steel cross section), this is the toe resistance of the open-toe pile at that toe movement.

While the shaft shear between the core and pile is assumed to be almost elastic–plastic, the gradual increase of force against the core base is best described by a gently rising ratio function, as established in an analysis of the core for the mentioned assumptions using the upward response of the core in a bidirectional test. This is indicated in the toe curve in Fig. D4*a*.

An ultimate resistance can always be established from the pile head load–movement curve by some definition or other. However, a pile is composed of a series of individual elements; therefore, ultimate resistances for the complete pile as determined from the pile head load–movement curve and that for the pile elements occur at different stages of the test. Whatever the definition based on the pile head load–movement, it has little relevance to the difference in response between the pile driven with a closed toe as opposed to that driven with an open toe. A useful relation in practice is the load at the pile head that results in a certain pile toe movement. The circles in Figs. D4*a* and D4*b* indicate the pile head load for a 5 mm toe movement, which is usually a safe value of "allowable load" that includes an allowance for group effects that can increase the toe movement during long-term service and, therefore, the pile foundation settlement, as well as for a moderate amount of future downdrag. Note that although the difference in toe resistance at the large toe movement (30 mm) is about 600 kN, at the more moderate 5 mm toe movement, the difference is only 200 kN between the closed- and open-toe pile alternatives.

In back-calculating the results of an actual static loading test on an open-toe pipe pile with a soil core and modeling the forces measured in various locations along the pile, the core effect cannot be treated as an ultimate toe resistance, but needs to be considered as an add-on movement-dependent resistance along a lower length of the core. This add-on shaft shear can be obtained by modeling the core effect separately assuming as if it were tested in a bidirectional test. While the core base (pile toe) movement is easily measured, the unit shaft shear along the core and the core stiffness will have to be assumed or determined in special tests.

Residual load distribution is rarely measured (it is rather difficult to do). However, its influence can be significant and a back-analysis would have to make allowance for this. The results are best presented as upper and lower boundary solutions. However, it is beyond the scope of this discussion to address this issue.

In the Conclusions section, the authors state that the "plugging effect" decreases with increasing pile diameter, but they do not state what the "plugging effect" is. I am unsure what the authors mean by the term. It is hardly a specific toe resistance value. I believe the authors refer to the fact that for increasing pipe diameter, the ratio between the core base area and the core shaft area increases (same core length). In driving a pile, a longer core would develop for a larger diameter pipe as opposed to a smaller diameter pipe. However, in a subsequent static loading condition, this is irrelevant.

The main questions are: (*i*) when in driving an open-toe pipe pile will it develop a rigid plug and, therefore, start responding like a closed-toe pile, (*ii*) as an inside soil core develops, what will the soil resistance response to the driving be, and (*iii*) what will the long-term static response be of the rigid plug or the soil column? Answer to the first two questions will come from compilation of field observations of full-scale pipe piles of different diameters and length driven in different soils. The answer to the third question will come from compilation of static tests on full-scale pipe piles of different diameters and length with measured core response and analysis of the static load–movement of the pile and the core as suggested above.

As demonstrated by the authors, the driving of a double-wall test pile engages the shaft resistance between the inside of a pile and the soil core over the full length of the pile, whereas the shaft resistance along the core in the case of a static loading test only engages the core along a limited length. Therefore, assuming the shaft resistance along the outside of the pipe and the toe resistance (of the pipe annulus) are equal in the dynamic and static tests, but not equal with regard to the shaft resistance along the inside soil column, the dynamic test on an open-toe pile would be expected to show a larger "capacity" than that of the static test. The "agreement" implied in Table 2 is therefore, in reality, a "disagreement".

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