Bearing Capacity of Tubular Piles: Technological Improvements and Model Testing

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Abstract. Some offshore and marine structures often include steel tubular piles of essential length (80–100 m and more) that should provide rather high bearing capacity regarding essential axial load. To increase piles bearing capacity under static pressing load, such an additional element as the internal diaphragm has been applied in some practical cases. Presented research aimed to study two connected processes during steel tubular pile driving: soil plug formation at the tip of the open-end pile and soil behavior under the internal diaphragm fixed inside the tubular pile's shaft. Results of physical modelling in laboratory conditions and their numerical analysis are discussed. Numerical analysis of the gained experimental data gave the possibility to apply approximating function with good correlation indexes. Obtained information of internal diaphragm application may be useful to provide an increase of pile's bearing capacity (in case of bearing capacity deficit) or to justify pile length reduction. Gained conclusions may facilitate design and construction of deep-water piled structures.

Keywords: Bearing capacity · Tubular pile · Soil plague · Internal diaphragm · Laboratory modelling

1 Introduction

Modern marine transportation and offshore structures such as deep-water port's berths, oil and gas platforms, raid and offshore fixed single point moorings, submerged stores and others often include steel tubular piles of essential length (80–100 m and more) as main bearing elements. Some examples of these structures are presented in Figs. 1, 2 and 3 [1].

Such tubular piles should provide high bearing capacity in case of external axial loads application [1–4].

One of the interesting peculiarities of long tubular open-end piles behavior is the formation of soil plugs at the piles' tip [5, 6].

From this point of view, we support a known opinion that it is important to study the influence of the soil plug not only on the pile's tip bearing capacity but also on soil behavior inside the tubular shaft [4].

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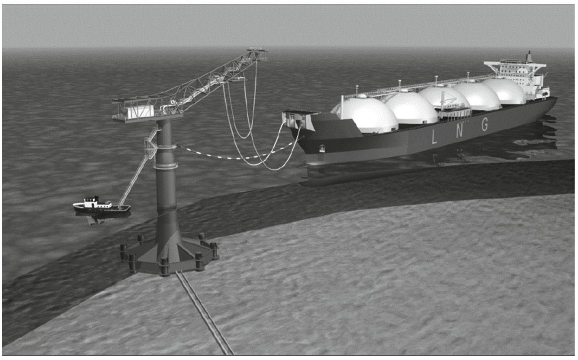

Fig. 1. Single point mooring fixed to the sea bottom by long steel tubular piles (LNG carrier service).

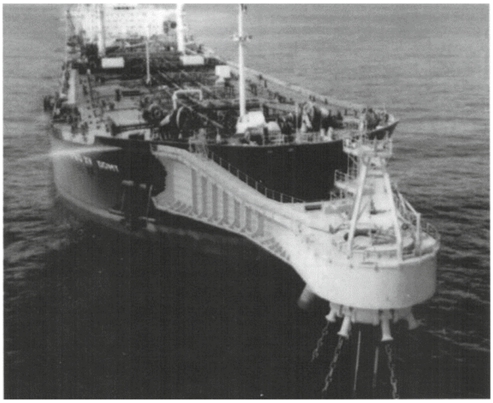

Fig. 2. Piled cluster applied to fix mooring device of the tanker.

Fig. 3. Use of large-diameter bearing monopile as fixed single point mooring for the offshore industry.

In case of necessity, the bearing capacity of long tubular piles may be increased by different methods:

- by driving a pile with a closed end to develop increased end-bearing resistance (but it requires application of too powerful hammers);
- by installing the pile at larger depth in order to reach the bearing soil strata (but such penetrations are often much greater than those required for fixity against lateral loading);
- by grouting beneath the pile toe (but the operations of cleaning-out the pile and grouting are slow and relatively costly);
- by welding a steel plate diaphragm across the interior of the pile in order to increase bearing capacity by use of soil reaction under the diaphragm (such method demonstrated good results on some marine projects and got positive references [4]).

Regarding that, in many cases, large diameter tubular piles of shelf structures are installed without plugging effect (so called "fully coring mode" [7]) or with partial plugging, the last approach (closure of the pile's shaft) looks rather attractive for deep water port, marine and offshore engineering but needs detailed consideration and study aiming to determine method's peculiarity, an appropriate sphere of application, details of diaphragm construction and proper location along the pile's shaft.

2 Application of Internal Diaphragm in Steel Tubular Piles

Recommended technology to install the internal diaphragm as described in [4] (Fig. 4).

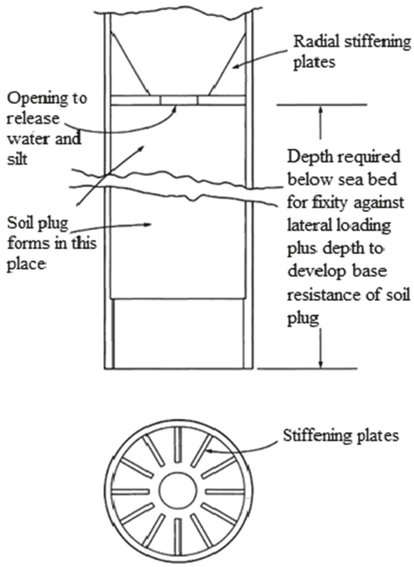

Fig. 4. Steel tubular pile with a diaphragm.

A hole is necessary for the diaphragm for the release of water pressure in the soil plug and to allow expulsion of silt. Stresses on the underside of the diaphragm are high during driving, and radial stiffeners are needed.

According to [4], the minimum depth above the pile toe for locating the diaphragm is the penetration below the sea bed required for fixity against lateral loading. There are formulas in some norms allowing determination of the fixity's depth depending on soil properties and pile's bending rigidity; roughly, this depth may be determined in the interval of (5–7) d. However, further penetration is necessary to form the soil plug under the diaphragm by compacting the soil within the plug and to develop the necessary base resistance. Thus, mentioned authors considered two locations for two soil plugs formation during the tubular pile driving: at the open end of the pile and under the internal diaphragm.

As an example of the diaphragm's practical application, we may refer to the piling works at the Hadera coal unloading terminal near Haifa [4]. Open-end piles 1424- and 1524-mm OD were proposed, but initial trial driving showed that very deep penetrations, as much as 70 m below sea bed in calcareous sands, would be needed to develop the required axial resistance. The blow count diagram showed quite low resistance at 36 m below the sea bed. Trials were then made of the diaphragm method. A diaphragm with a 600 mm hole giving 83% closure of the cross-section was inserted 20 m above the toe. This increased the driving resistance at 39 m below sea bed and another trial with a 300 mm hole (95% closure) gave a higher resistance at 37 m. It was supposed that such improvement of piles bearing capacity was stipulated by soil plug formation below the mentioned diaphragms.

3 Laboratory Model Testing

Regarding that obvious effect (an increase of the pile's bearing capacity) has been achieved by the use of the rigid diaphragm, our intention was to study peculiarities of the considered approach providing model static tests in the laboratory conditions. Our aim was to obtain parameters describing the considered pile driving processes – both qualitative (related to the process in general) and quantitative (characteristic for the applied model pile-soil system) ones.

As to the method of pile's installation, we suppose that traditional approaches (use of impact hammer or vibro hammer) are not reliable enough to provide safety of the rigid diaphragm fixed by welding inside the pile's shaft and interacting with soil under the diaphragm. In order to avoid dynamic actions upon the diaphragm during pile penetration, we prefer to consider a safer but more effective method of pressing load application [6].

To clarify above mentioned items related to the tubular pile with the internal diaphragm, we have started a series of experimental studies in the Geotechnical Laboratory of the Department "Sea, River Ports and Waterways" at Odessa National Maritime University (Odessa, Ukraine).

For pile testing, we used a soil box of dimensions: width 600 mm, length 750 mm, depth 1100 mm (Figs. 5, 6, 7 and 8). For the model of the tubular open-end pile, we apply steel pipe d = 50 mm external diameter, pipe wall thickness 1 mm, $l = 800$ mm length. To drive the pipe into fine sand mechanical jack has been applied.

For experimental studies, we used fine sand with the following characteristics: internal friction angle 33°; density 14.5 kN/m^3; void ration 0.71; moisture 0.07%; Young modulus 16 MPa, Poisson's ratio 0.3.

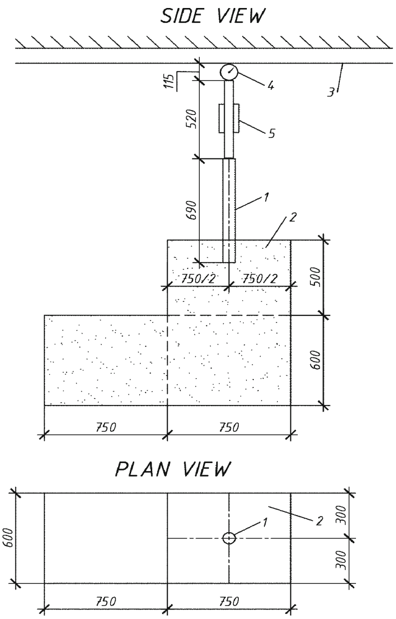

Fig. 5. Scheme of the experiment: 1 – pile model; 2 – sand box; 3 – bearing beam; 4 – force gauge (dynamometer); 5 – jack loading system (all sizes in millimeters).

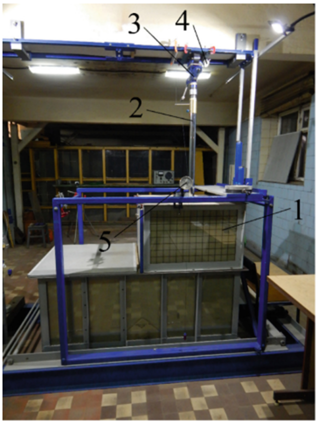

Fig. 6. Experimental system: side view. 1 – soil box; 2 – model pile; 3 – mechanical jack; 4 – dynamometer; 5 – displacement gauge.

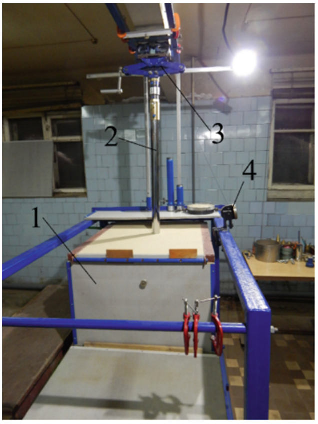

Fig. 7. Experimental system: front view. 1 – soil box; 2 – model pile; 3 – mechanical jack; 4 – displacement gauge.

Fig. 8. Loading system. 1 – model pile; 2 – mechanical jack; 3 – dynamometer.

The first series of experiment was aimed to determine the conditions of the soil plug formation at the tip of the open-end pile model.

Measured parameters (at each stage of load application) were:

- applied vertical axial pressing force (named *Load, F* on the diagrams)
- pile's penetration depth (named *Displacement, t* on the diagrams)
- soil level inside the pile's model (the initial position was fixed when soil levels inside and outside the pile were equal).

In order to describe the process of the model tubular pile plugging, we applied:

- earlier proposed IFR (Incremental Filling Ratio) and PLR (Plug Length Ratio) characteristics (for example, recommendations [7–10] and others) presented in Figs. 9 and 10;

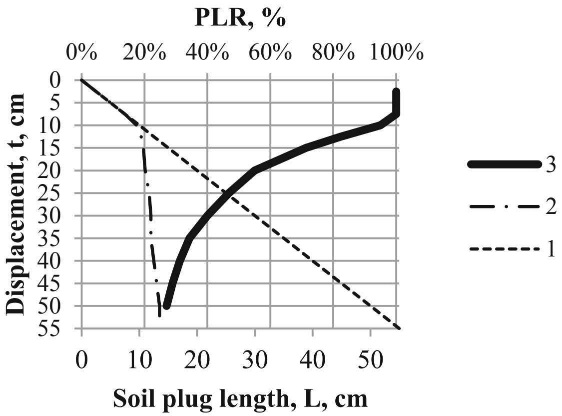

Fig. 9. Typical dependencies for the tests of the first series: 1 – fully coring mode; 2 – L = L(d); 3 – PLR.

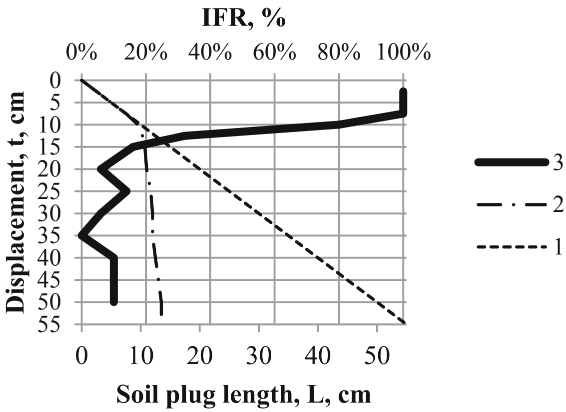

Fig. 10. Typical dependencies for the tests of the first series: 1 – fully coring mode; 2 – L = L(d); 3 – IFR.

- Assumption that as evidence of completing the process of soil plug formation, we may consider equal pile's vertical displacements and related settlements of the sand surface inside the shaft at each stage of further axial load application (related diagrams are presented in Figs. 11 and 12).

According to the PLR and IFR diagrams (Figs. 9 and 10) at the initial stage of pile installation [till t = (1.5–2) d] PLR = IFR = 100%, i.e., the pile is driven according to the "fully coring mode". From the penetration depth of some (4–5) d, soil plug length is almost unchanged; average IFR value is stable too.

Some IFR fluctuations below the penetration depth = 20 cm may be explained by technical reasons: resetting of the jack each 20 cm of the penetration process and corresponding installation of extension tubes.

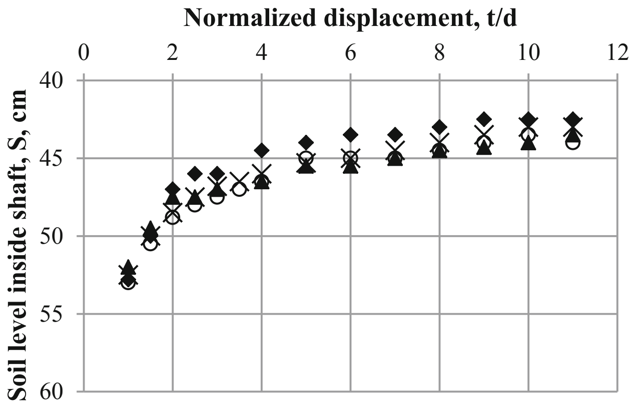

Fig. 11. Results of open-ended model pile jacking (4 similar tests): S – the distance between pile's top and soil in the shaft.

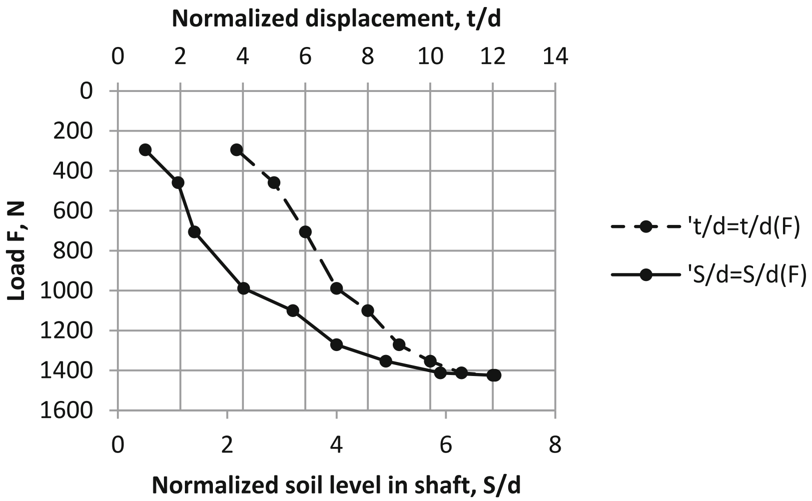

Fig. 12. Results of open-ended model pile jacking (average points of 4 similar tests).

A similar conclusion may be made on the basis of diagrams in Figs. 11 and 12. Soil level inside the pile's shaft becomes stable at the relative penetration depth (normalized pile displacement) approx. t/d = 5–6 (Fig. 11). Dependencies for t/d and S/d describing pile and soil displacements also become parallel starting from the t/d = 5−6 and till reaching the pile bearing capacity at t/d = 8–9 (Fig. 12).

Some important results of the first series of the laboratory experiments may be formulated in the following way:

- in fine sandy soil, the plug is formed at the comparatively early stage of pile's driving (in the considered case – at the penetration depth of around 4–5 pile's diameters);

- if to locate the internal diaphragm at the recommended depth required for fixity against lateral loading as described above (approx. at the penetration depth of around 5–7 pile's diameters), we may meet the situation of no contact between the diaphragm and the soil inside the shaft (clearance space); i.e., the diaphragm does not catch up with soil.

The second series of the experiment was devoted to clarification of the role and contribution of the internal diaphragm. For the model pile, the diaphragm was produced as a circular steel plate (4 mm thickness), with its diameter corresponding to the inner diameter of the pile.

By use of the rigid steel bar (located in the pile's shaft) the diaphragm was connected with the pile head. Varying the length of the mentioned rigid bar, we had the possibility to locate the internal diaphragm at different places along the model pile's shaft.

At the second series of the experiment internal diaphragm was fixed at several positions by changing the distance from the tip of the model pile: 0 (closed end); 3d; 6d; 9d (total length of the pile was equal to 16d).

It has been discovered that due to sand settlements inside the pile shaft during pile installation, there is an empty space under the diaphragm and, correspondingly, no contact between soil and diaphragm.

In order to avoid clearance space under the diaphragm and to provide constant contact of the sand with the underside of the diaphragm, we applied the diaphragm with several small holes allowing sand to fill into the space under the diaphragm (Fig. 13). For above mentioned options of the diaphragm location, the sand was filled after the pile driving on the depth 3d, 6d and 9d correspondingly (i.e., at the moment of the first potential contact of the soil surface with the underside of the diaphragm).

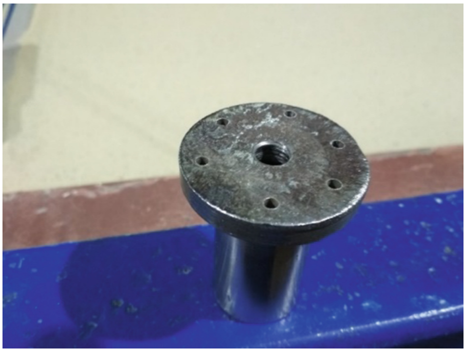

Fig. 13. Underside of the internal diaphragm with peripheral holes for sand filling.

The volume of the filled sand was calculated to provide the required diaphragm-sand contact during the whole process of model pile jacking.

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As it is demonstrated by diagrams presented in Figs. 14, 15, 16 and 17, application of the internal diaphragm provides increasing open-end pile bearing capacity. The degree

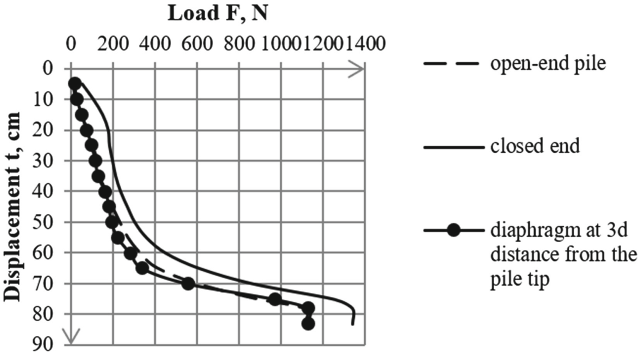

Fig. 14. Dependencies between vertical axial load upon the model pile and its displacements for the diaphragm at 3d distance from the pile tip.

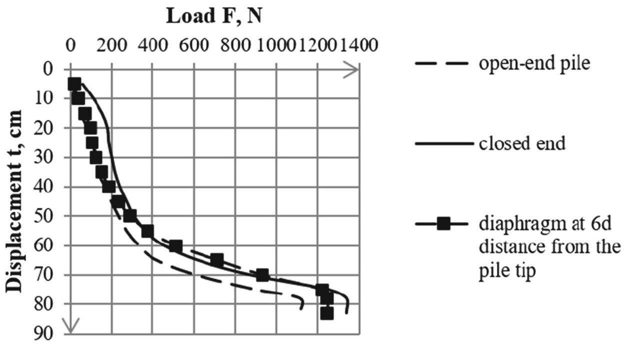

Fig. 15. Dependencies between vertical axial load upon the model pile and its displacements for the diaphragm at 6d distance from the pile tip.

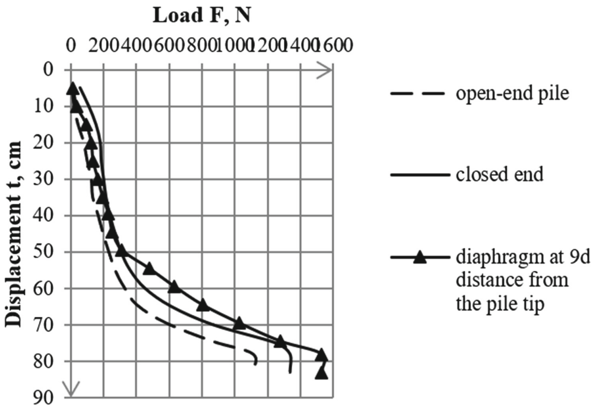

Fig. 16. Dependencies between vertical axial load upon the model pile and its displacements for the diaphragm at 9d distance from the pile tip.

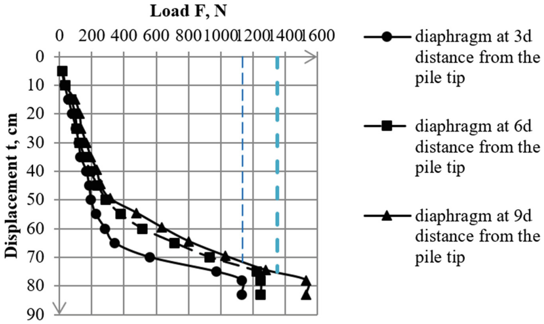

Fig. 17. Dependencies between vertical axial load upon the model open-end pile and its displacements.

of such increase depends on the diaphragm location. For the considered options of the diaphragm fixing point, the minimum increment of the open-end pile bearing capacity relates to the 3d distance between the diaphragm and the pile tip (Fig. 14 and 17) and the maximal increment is measured at the 9d distance (Fig. 16 and 17).

Perhaps mentioned circumstances may be commented by the following way. The upper plug under the diaphragm may be formed if there is the proper base reaction developed inside the shaft. Such a situation may occur if the upper plug (being in the process of formation) meets the already formed lower plug. The last transfers additional pressure to the soil under the toe and provokes an additional base reaction. Thus, an additional external force acts on the plug and increases soil density in it. In fact, after that stage, two plugs are combined and work as one large plug between the diaphragm and pile's toe. Obviously, the creation of the mentioned large plug and its effective contribution to the pile bearing capacity may be provided only in case of the "right" location of the diaphragm (not too low and not too high). For our model tests, the maximal bearing capacity of the open-ended pile was measured in the case of 9d distance of the diaphragm from the pile tip.

It may be explained, particularly, by the fact that for the considered test conditions, approximate driving depth t = (4–5) d at the initial stage of pile installation is needed to dense a soil due to the development of the friction forces inside the pile's shaft and to form a lower soil plug at the pile tip. If then to apply similar consideration for the follow-on stage of the driving process – compaction of the soil under the diaphragm due to the similar friction forces, required penetration depth for this stage to form the upper plug may be of similar value (4–5) d. So the total distance between pile toe and the diaphragm may be considered as the sum of these two parts of the penetration depth, i.e., approx. (8–10) d. Such location of the diaphragm may be optimal to form two plugs consecutively and to combine them in one large plug.

Regarding quantitate parameters of open-end pile bearing capacity (Fig. 17), we would like to note that due to the diaphragm's contribution, pile bearing capacity may be increased (in our tests up to 15–20%). Another effect consists in the possibility of decrease pile driving depth (10–15%). Obviously, mentioned figures should be considered with regard to possible experimental errors stipulated by differences in the reproducibility of the model ground preparation as well as to measurement inaccuracy (perhaps up to 10% in total).

Regarding scale-effects for the considered problem, it should be noted the following. From the point of view of so-called "direct modelling" [11, 12], dependencies between limit axial force in a pile $N_{lim,p}$ (as well as related displacements $U_{lim,p}$) and similar parameters of the model may be presented as

$$\frac{N_{lim,p}}{N_{lim,m}} = C_L^n \tag{1}$$

$$\frac{U_{lim,p}}{U_{lim,m}} = C_L^m \tag{2}$$

where C_L is the sizes (scales) correlation between prototype and the model; n and m are parameters depending on soil properties and pile dimensions.

For the conditions of our laboratory model testing (skipping the details of intermediate conversions and calculations), it was determined that the related prototype is a tubular pile of diameter 1.0 m driven up to 10 m into similar sandy soil. Its bearing capacity (sum of the toe and shaft bearing capacities) is 1723 kN. For comparison: the calculated value of the prototype bearing capacity according to the recommendation of the related Ukrainian code occurred to be 2020 kN (some 15% difference).

Also, for plugging effect assessment, we have to consider scale effects stipulated by the influence of internal pile diameter. This aspect is subject to a study for further investigations.

4 Numerical Analysis of Experimental Data

The graphical analysis of the experimental data led to the assumption that the relationship between the displacement of the pile (t) and the load (F) exerted on the pile can be described by power dependence. Assuming that the constraint equation is a function $F = a \cdot t^k$, by the least squares method after solving the linear (with respect to $ln\, a$ and k) system,

$$\begin{cases} n \cdot ln\, a + k \cdot \sum_{i=1}^{n} ln\, t_i = \sum_{i=1}^{n} ln\, F_i \\ ln\, a \cdot \sum_{i=1}^{n} ln\, t_i + k \cdot \sum_{i=1}^{n} ln^2 t_i = \sum_{i=1}^{n} ln\, F_i \cdot ln\, t_i \end{cases} \tag{3}$$

we have received the following values:

1) $a = 1{,}224$, $k = 1{,}485$ for pile with diaphragm at 9d distance from the pile tip;
2) $a = 0{,}272$, $k = 1{,}822$ for the open-end pile;
3) $a = 5{,}21$, $k = 1{,}1$ for the closed end pile.

The approximation error in all cases turned out to be greater than 7%:

1) $\overline{A} = 21\%$,
2) $\overline{A} = 22{,}2\%$,
3) $\overline{A} = 28{,}2\%$,

which suggests the need to select a more accurate model.

Empirical correlation relations are:

1) $\eta = 0{,}66$,
2) $\eta = 0{,}642$,
3) $\eta = 0{,}563$.

According to the Chaddock scale these relations indicate a salient correlation between t and F.

Correlation indexes for the considered cases

1) $R = 0{,}882$,
2) $R = 0{,}882$,
3) $R = 0{,}806$

are high enough, which means the tightness of the relationship of the considered values.

If we assume that the relationship between the considered values is described by exponential laws ($F = a - b \cdot e^{-mt}$, $F = n \cdot e^{kt}$), then the least squares method allows determination of the parameters b, m, n, k; the value of the parameter a may be obtained from the experiment. However, the discrepancy between the actual and calculated values of F is so great that further comparison becomes meaningless. A similar situation is observed with the inverse (hyperbolic) dependence.

However, the graphic interpretation of the "displacement – load" function (Figs. 14, 15, 16 and 17) prompted the idea to look for a model in the form

$$t = a \cdot \sqrt{F} - \frac{1}{F - b} + c \tag{4}$$

It turned out that in all cases it is possible to select the values of the coefficients a, b, c (see below) so that the approximation error lies in the range of 5%–7%:

1) $\overline{A} = 5{,}1\%$ for pile with diaphragm at 9d distance from the pile tip,
2) $\overline{A} = 4{,}6\%$ for the open-end pile,
3) $\overline{A} = 4{,}4\%$ for the closed end pile.

It confirms the correctness of the choice of the model. In such a case, the relationship between the quantities under consideration is very high:

1) $\eta = 0{,}985$,
2) $\eta = 1$,
3) $\eta = 0{,}986$.

Accordingly, the correlation indexes in all cases are practically equal to one.

Concretization of the function (4):

1) $$t = 2{,}141\sqrt{F} - \frac{1}{F-1355} + \begin{cases} (-3)\ as\ F \in [12; 130), \\ 2,5\ as\ F \in [130; 240), \\ 12\ as\ F \in [240; 800), \\ 0\ as\ F \in [800; 1355) \end{cases}$$

for pile with diaphragm,

2) $$t = 2{,}243\sqrt{F} - \frac{1}{F-1249} + \begin{cases} (-3)\ as\ F \in [7; 13),\ F \in [1000; 1249), \\ 0\ as\ F \in [13; 40), \\ 3\ as\ F \in [40; 70), \\ 6\ as\ F \in [70; 127),\ F \in [600; 1000), \\ 10\ as\ P \in [127; 240), \\ 9\ as\ F \in [240; 320),\ F \in [450; 600), \\ 15\ as\ F \in [320; 600) \end{cases}$$

for the open-end pile,

3) $$t = 2{,}152\sqrt{F} - \frac{1}{F-1255} + \begin{cases} (-8)\ as\ F \in [40; 120), \\ (-3)\ as\ F \in [120; 160), \\ 5\ as\ F \in [160; 180),\ F \in [1000; 1255), \\ 6\ as\ F \in [180; 230), \\ 9\ as\ F \in [230; 270),\ F \in [800; 1150), \\ 13\ as\ F \in [270; 300),\ F \in [500; 800) \\ 16\ as\ F \in [300; 500) \end{cases}$$

for the closed end pile.

Comparison of the experimental values and calculated graphs for the considered cases is presented below (Figs. 18, 19 and 20) .

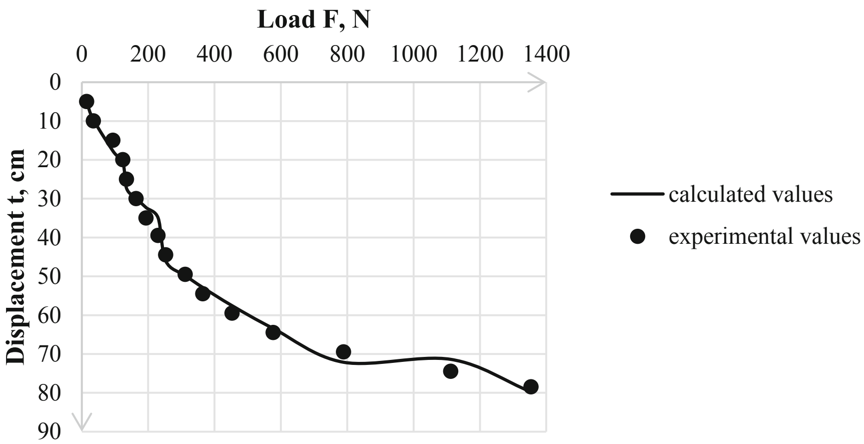

Fig. 18. Comparison of the experimental and calculated by formula (5) values for pile with diaphragm at 9d distance from the pile tip.

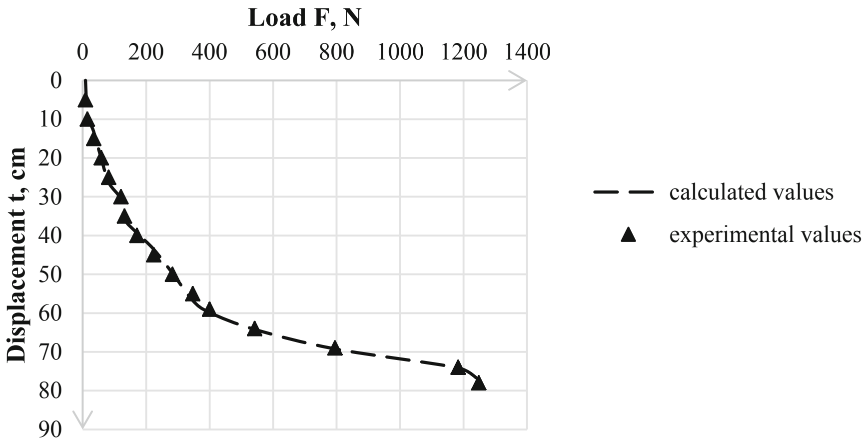

Fig. 19. Comparison of the experimental and calculated by formula (5) values for the open-end pile.

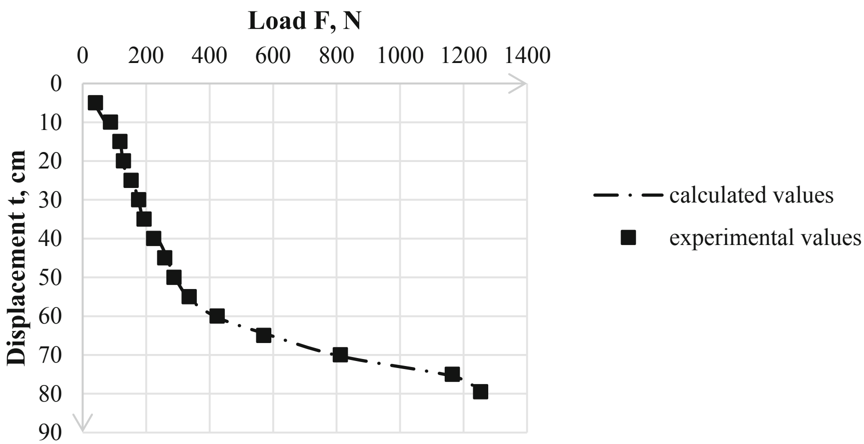

Fig. 20. Comparison of the experimental and calculated values for the closed end pile.

For the further development of the model, it will be useful to refine the coefficients *a, c* or to assume instead of them some functions depending on *F*, i.e., to look for a function in the form

$$t = a(F) \cdot \sqrt{F} - \frac{1}{F - b} + c(F) \tag{5}$$

In particular, for some considered experimental cases it has already been possible to obtain an approximation in the form:

$$t = a\sqrt{F} - \frac{1}{F - F_n - 1} + 3\sin(F - \overline{F}) + \begin{cases} \frac{\bar{t}}{\sqrt{n}} \; as \; F \in \left[\overline{F} - \frac{F_n - F_1}{k\sqrt{n}}; \overline{F} + \frac{F_n - F_1}{m\sqrt{n}}\right], \\ 0 \; as \; F \in [F_1; F_n + 1) \backslash \left[\overline{F} - \frac{F_n - F_1}{k\sqrt{n}}; \overline{F} + \frac{F_n - F_1}{m\sqrt{n}}\right] \end{cases} \tag{6}$$

where $\overline{F}$ is the sample average of the applied (experimental) loads, $\bar{t}$ is the average of the observed displacements, $a = \frac{\bar{t}}{\sqrt{\overline{F}}}$, k, m are some experimental parameters (constants).

So, for the pipe with diaphragm at 9d distance from the pile tip the following parameters were obtained: $\overline{A} = 8{,}2\%$, $\eta = 0{,}957$, $R = 0{,}993$ (at $k = 2$, $m = 1/\sqrt{2}$); see Fig. 21.

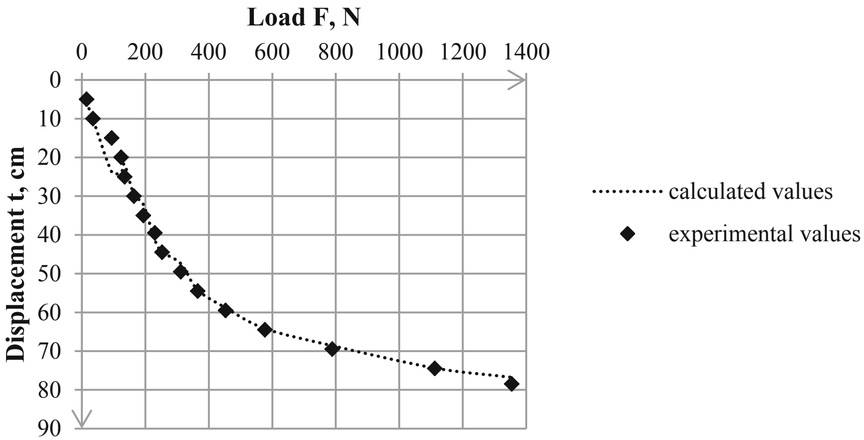

Fig. 21. Comparison of the experimental and calculated by formula (6) values for pile with diaphragm at 9d distance from the pile tip.

For the open-end pile the following parameters were obtained: $\overline{A} = 12{,}6\%$, $\eta = 0{,}997$, $R = 0{,}988$ (at $k = \sqrt{3/2}$, $m = \sqrt{2/3}$); see Fig. 22.

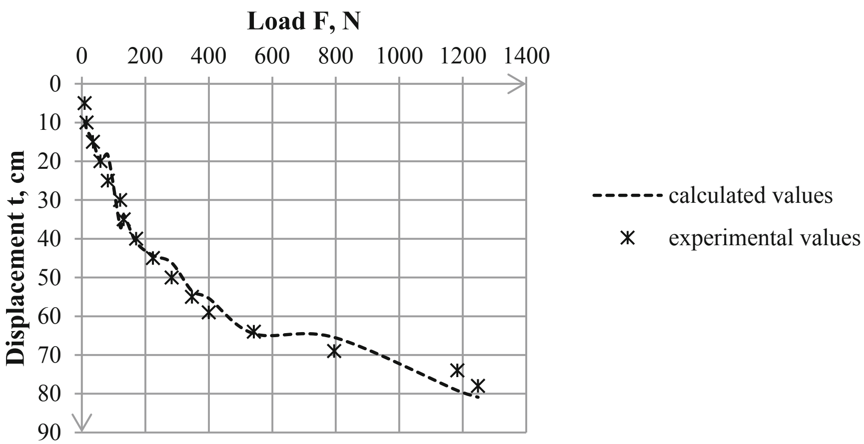

Fig. 22. Comparison of the experimental and calculated by formula (6) values for the open-end pile.

It is possible to note that there may be a model with another type of the function *c(F)*, but it is obvious that such a function should include some periodic component.

5 Conclusion

As obtained from the presented initial series of our experimental studies, a rigid diaphragm inside the tubular open-end pile may be a useful element for increasing the pile's bearing capacity.

Consecutive formation of two soil plugs (lower one formed just at the pile tip and then upper one formed under the diaphragm) leading to their partial or full integration is most effective when the optimal location of the diaphragm inside the pile shaft is provided. From the point of view of pile bearing capacity under axial compressive load and for the considered experimental conditions, such proper distance between the pile tip and internal diaphragm occurred to be around 9d (d – pile diameter).

As demonstrated by our tests, during the pipe jacking process, there is a possibility of clearance space between the diaphragm and soil inside the pile's shaft (no contact situation). For real construction site conditions and inhomogeneous soil base, it is a complex task to check proper diaphragm-soil contact and their interaction or to determine clearance space formation under the diaphragm. That's why it is proposed (and checked by our tests) the technological improvement based on sand filling into space under the internal diaphragm to provide constant diaphragm-soil contact and related soil resistance. Such an approach guarantees force interaction between the internal diaphragm and the soil inside the shaft via filled sand and may simplify the calculation scheme of such interaction (the last is a task of future development of the considered problem).

Improvement of the pile effectiveness determined by experimental modeling for the above-mentioned pile-soil conditions provides 15–20% increase of the bearing capacity or 10–15% reduction of the driving depth.

Numerical analysis of the gained experimental data gave the possibility to apply approximating function with good correlation indexes.

Presented experimental studies should be continued and developed in order to study installation peculiarities of piles with diaphragm by pressing technologies. Also, it looks prospective to investigate influence of internal diaphragm's design (different from the considered flat plate option).

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