XVIII Brazilian Conference on Soil Mechanics and Geotechnical Engineering
The Sustainable Future of Brazil goes through our Minas
COBRAMSEG 2016 — 19-22 October, Belo Horizonte, Minas Gerais, Brazil

MECHANISMS AND GEOMETRY OF PILE INSTALLATION BY WATER JET FLUIDIZATION IN FINE SAND

Larissa de Brum Passini
Federal University of Paraná, Curitiba, Brazil, larissapassini@hotmail.com

Fernando Schnaid
Federal University of Rio Grande do Sul, Porto Alegre, Brazil, fschnaid@gmail.com

Marcelo Maia Rocha
Federal University of Rio Grande do Sul, Porto Alegre, Brazil, marcelo.maiarocha@gmail.com

SUMMARY: The mechanism of pile installation by vertical jet fluidization in saturated fine sand were examined in order to define the relevant parameters controlling pile installation geometry and pile depth of embedment. The work describes a series of laboratory model tests with a pile installed by its own weight, in both medium and dense sands using downwardly directed vertically water jets. Laboratory tests were performed in a reduced model using tubes to simulate torpedo without wings, following the law of similarity of Froude number. Measurements from model tests at reduced scale indicate that the geometry of fluidized cavities is not influenced by the initial density of the sand, and that the perturbed zone is constraint to a distance of about 2 diameters from the pile centreline during pile installation, independent of the applied flow rate. By equating gravitational and upward drag forces on sand particles through Froude number, an expression for the fluidized zone is derived, which shows to be correlated with fluidization water jet flow rate, as well as the soil and fluid properties, for tests executed at the centre and at the lateral of the acrylic chamber/tank. Further, the critical depth of installation of fluidized piles is expressed as a function of the fluidized zone. Under fluidization, penetration increases with increasing flow rate and decreasing soil relative density, showing a critical depth of installation around 15 to 50 times the pile diameter, showing the advantage of the technique.

KEYWORDS: Torpedo Piles, Fluidization, Mechanisms and Geometry, Fine Sand.

1 INTRODUCTION

The torpedo pile (Figure 1) is an anchoring offshore structure system widely used in Brazil in deep waters. The installation procedure consists in the release of the torpedo from a high position from the sea bottom to allow the device to reach its terminal velocity. Penetration depth is a function of kinetic energy, weight and geometry of the torpedo, seabed shear resistance and penetration angle.

Since penetration in sand may not be sufficient to provide the necessary anchor bearing capacity, an alternative procedure is to combine the free-fall energy to soil fluidization. This alternative is applied in this paper by considering the use of circular, single and continuous vertical jets from the tip of model torpedo piles. Furthermore, this technology can be applied not only at offshore conditions, but also at nearshore and onshore situations.

Recent studies on the mechanisms, geometry and parameters involved in the fluidization process have been presented by Mezzomo (2009), Stracke (2012), Schnaid *et al.* (2014), Passini & Schnaid (2014a,b), Passini *et al.* (2015) and Passini & Schnaid (2015). All of them developed as part of the project signed between Petrobras and Federal University of Rio Grande do Sul (UFRGS). The present article extends these early views by exploring the installation mechanisms and geometry of torpedo piles in granular soils.

Figure 1. Torpedo pile T-120 tons (Fluke Subsea, 2016).

2 EXPERIMENTAL PROGRAM

2.1 Material and Methods

The tests were performed in nonplastic, clean, air dried and sieved fine silica sand with uniform particle size (Table 1) from a natural deposit located at the city of Osorio in southern Brazil. It is a standard research material used at Federal University of Rio Grande do Sul in fluidization studies (Mezzomo, 2009; Stracke, 2012; Passini, 2015) The particles are predominantly quartz.

Table 1. Soil properties.

Characteristics	Fine sand
Soil particle density, ρ_s (kg/m³)	2667
Median soil particle diameter, d_{50} (mm)	0.207
Characteristic soil particle diameter, d_{95} (mm)	0.500
Minimum void ratio (e_{min})	0.60
Maximum void ratio (e_{max})	0.90
Uniformity coefficient (C_u)	2.2
Curvature coefficient (C_c)	1.1
Coefficient of hydraulic conductivity, k (m/s)	1.28×10^{-4}
% Medium sand	13.46
% Fine sand	81.90
% Silt	3.03
% Clay	1.61

Two sectioned rectangular acrylic transparent chambers with dimensions of 450 mm × 450 mm × 1100 mm (700 mm bottom + 400 mm top part) and 15 mm thickness walls were used in this set of experiments. Specimen prepared using the moist tamping method to simulate the seabed were carefully filled in 30 mm layers compacted manually with a hand socket to achieve the target relative densities (D_r) of 50% and 90%.

Once the soil preparation was completed, the tank was inundated by water percolating upwards with hydraulic gradient much lower than critical, with constant flow rate of 0.7 L/min, until the water reaches a steady level (h_w), ranging from 30 mm and 50 mm above the soil surface. It is then maintained constant throughout the test by allowing the water to overflow through the edges of the tank to ensure that the flow rate and therefore the jet velocity are constant. Tests were carried out in the center position inside the tank and adjacent to the tank wall. All experiments were photographed and filmed.

Carbon steel metal tubes have been used to simulate model piles with internal diameters (d_i) and external diameters (d_e) of 11.6 and 16.2 mm. No special nozzle at the pile tip was used, once the intention was to keep the geometry as simple as possible in order to study the basic concepts of fluidization, following procedures earlier adopted by Mezzomo (2009) and Stracke (2012). These geometries were chosen according to scaling laws for future comparison to Petrobras anchors.

To start the test, model was positioned vertically, resting against the soil surface. A downwardly directed vertical water jet velocity was applied through the tubes, allowing installation by the combined effect of the tube own weight and fluidization jet. The top of the pile was secured by a cable connected to a dead weight through a pulley system that provides the necessary net pile weight in the model test. A supporting structure equipped with a vertical guide enforces the pile to slide down in a precise upright position.

The hydraulic circuit consisted on a polyvinyl chloride pipeline where the water from a 500 liters reservoir flowed driven by a centrifugal pump. The pump motor was connected to a frequency inverter, which regulate the rotation speed of the pump and thus the flow rate, measured by a low flow sensor with a digital transmitter/totalizer also connected to the frequency inverter encoded PID(proportional-integral-derivative controller). For each flow test the frequency of rotation of the pump was selected by a previous calibration of the equipment, i.e. the equipment was designed to maintain a constant flow rate during the fluidization process. Flow rates (Q_0), corresponding to 0.7, 1.0 and 1.6 L/min were applied during pile installation. The insertion tubes were the final part of the circuit. The water produces a jet emanating from the tubes that fluidizes the soil zone when inserted inside the sand. A schematic representation of the experimental setup is presented in Figure 2.

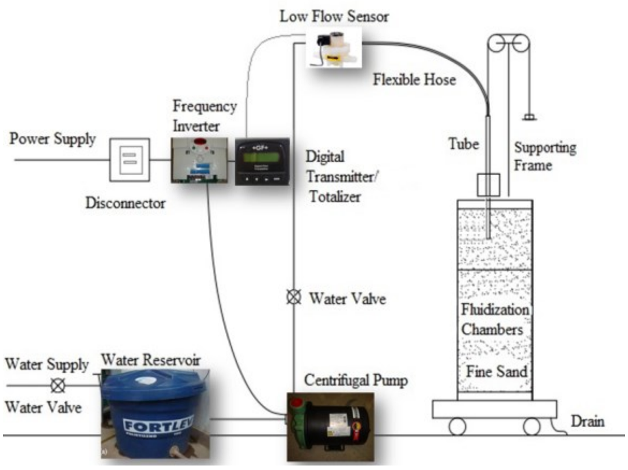

Figure 2. Experimental set up.

Boundary effects were not observed. The radial flow percolating through the sand may occasionally reach the lateral wall of the tank, flowing upwards and out the tank, without affecting the actual geometry and depth of the fluidized sand.

Controlled variables were the internal diameter (d_i), which is the same as the injection diameter (d_j), external pile diameter (d_e), pile model dry mass (m), sand relative density (D_r), water height above the soil (h_w), and the dimensions of the tank.

Measured parameters were the flow rate (Q_0), jet velocity (U_0), pile length inside sand specimen (L), and installation time (t), from tests carried out in the center position of the tank, and the radii around the pile (d_h), for tests adjacent to the tank wall.

2.2 Scaling Considerations

The scaling criteria for model piles are here established according the Froude number at 1*g* conditions. The Froude number (Fr) is a dimensionless number defined as the ratio of a characteristic velocity to a gravitational wave velocity. It may equivalently be defined as the ratio of a body's inertia to gravitational forces.

$$Fr = \frac{U}{\sqrt{Lg}} \tag{1}$$

where U is the velocity, g is the acceleration due to gravity, and L is the length.

Petrobras torpedo piles are a reference to the present study. The prototype presents dimensions of 1.07 m external diameter, 17 m length and dry masses of 66 ton (647.46 kN in weight) and 120 ton (1177.20 kN in weight). Given these prototype properties, length scale were chosen as approximately 1:67, being the modeling ratio N described in Table 2.

Table 2. Some fulfilled correspondences between model and prototype.

Model		Prototype	
Scale	1:67	Scale	1:1
d_e (mm)	16.2	d_e (m)	1.07
d_i (mm)	11.6	d_i (m)	0.80
L_m (mm)	254	L_p (m)	17
m_A (g)	220	m_A (ton)	66
m_B (g)	400	m_B (ton)	120

In situ stresses and soil particle distributions cannot be replicated in the model, since any characteristic particle size of the sand is not scaled down. Consequences are that soil porosity was maintained, as so was the intrinsic permeability of the soil mass and the fluid properties. Since the hydraulic gradient is dimensionless in Darcy's law, the scale factor for jet velocity is equal to 1. The flow velocity inside the soil pores, known as seepage velocity is a function of Darcian flow velocity and soil porosity, being also equal to 1. As the flow is continuous, it must be the same throughout the system. Furthermore, soil fabric was the same in prototype and model conditions, as the scale factor for the flow is 1. These are fundamental considerations in the interpretation of fluidized model tests.

Besides soil under fluidization, there is another relevant point to be mentioned which is related to scale effects arising from the relative magnitude of the soil grain size and structural model pile dimensions. The diameter of the model piles is of the order of a few centimeters and the tested fine sand would correspond to gravel size material in prototype scale. In conclusion, the installation results are affected by some scale effects and studies in larger model scales are required to validate the observed pile response to field conditions.

2.3 Test Parameters

Two parameters form the dimensionless group that controls soil fluidization: the densimetric Froude number of particles, Fr_{p} (which depends on Reynolds number of particles, Re_{p}) and Reynolds number of the jet, Re_{j}.

The Froude number of particles (Fr_p) is between 1.89 and 4.33, the Reynolds number of particles (Re_p) goes from 22.74 to 51.99 and the Reynolds number of the jet (Re_j) from 1274.57 to 2913.29.

The assumptions for water at 20 °C were fluid density (ρ_f), 998.29 kg/m^3, fluid dynamic viscosity (μ_f), 1.01×10^{-3} kg/m.s, kinematic viscosity (ν_f), 1.01×10^{-6} m²/s and the acceleration due to gravity is (g) 9.81 m/s^2. All these relevant parameters are 1:1 scaled.

3 GEOMETRIES AND DEPTHS

3.1 Measured Results

Lateral Tests

A typical tests performed at the lateral of the tank wall is presented in Figure 3 and is used to illustrate the geometry and mechanism around the penetrating pile. The solid lines represent the fluidized geometry during pile free fall installation under fluidization and the dashed lines represent the final geometry of the fluidized zone at the end of the penetration maintaining the applied flow rate. The fluidized zone can be separated into a disturbance zone and unchanged material. In the disturbed zone the original soil structure is invariably altered after fluidization, the grains are loosened by the jet, the pore pressure increases as the effective stresses rapidly reduces to zero and eventually the particles start to circulate around the pile toe. Outside of the fluidized zone, the grains remain fixed or circulate slowly locally, without any significant degree in the soil original properties.

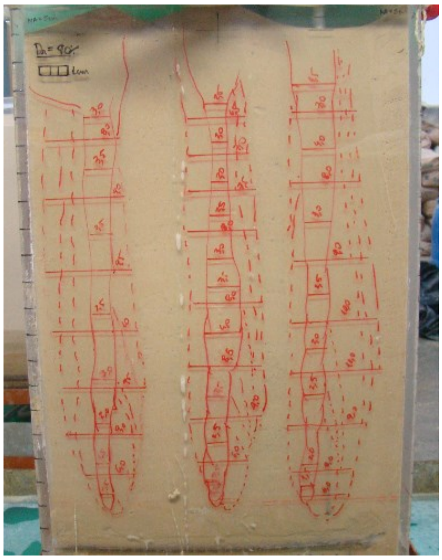

Figure 3. Lateral test for d_e = 16.2 mm, Q_0 = 1.6 L/min, m_B = 400 g, D_r = 90%.

From lateral tests at Figure 4 (a,b), it was observed during the free fall pile installation that the diameter of the fluidized zone (d_h) is approximately two times the piles external diameter. At the end of each test, after penetration, the tube was suspended and the perturbed zone around the pile extends to about six times the external pile diameter, increasing with the increase in Q_0 and U_0, approaching the behavior described for suspended tubes (Passini & Schnaid, 2015).

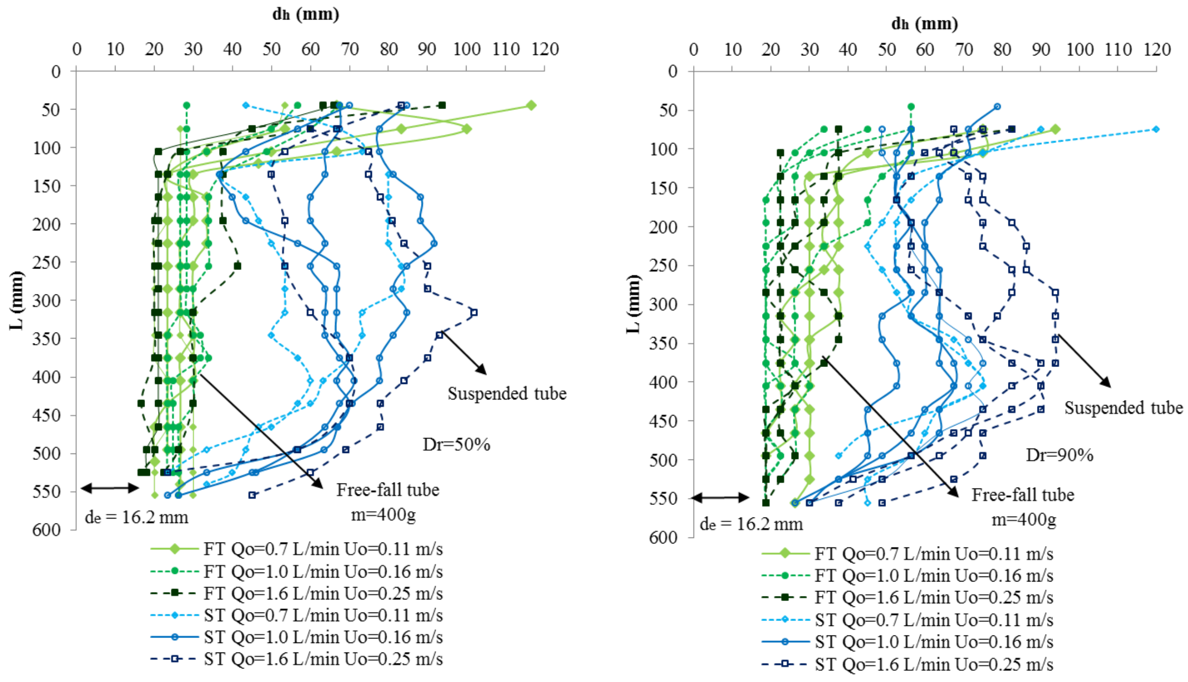

Figure 4. Fluidized zone *versus* pile penetration, from lateral tests in free fall (FT) and suspended tube (ST) at D_r = 50% and 90%.

Center Tests

Test results from pile installation depths in the center of the tank for samples prepared at 50% e 90% relative densities (D_r), showed that penetration (L) increases with increasing dry mass (m), flow rates (Q_0) and jet velocity (U_0), for the same relative densities. Also, increasing the relative density decreases the penetration for the same pile dry mass, flow rate and jet velocity applied, as shown at Figure 5.

3.2 Predicted Results

The most important condition for particle suspension to occur is that particle drag forces upward (F_d) must be larger than particle weight downward (F_w). By assuming an equivalent spherical shape for each sand particle, with diameter (d_p), the drag force may be calculated as:

$$F_d = \frac{1}{2}\rho_f U_p{}^2 C_d A_p \tag{2}$$

where U_p is the water flow velocity around the suspended particle (not to be confused with the jet velocity, U_0), C_d is the drag coefficient, which depends on Reynolds number, and A_p is the reference (frontal) area of sand particle, considering its spherical, given by:

$$A_p = \frac{\pi}{4} d_p^2 \tag{3}$$

Holding on the assumption of an equivalent spherical grain, the average grain weight force is given by:

$$F_w = \left(\rho_p - \rho_f\right) g \left(\frac{\pi}{6} d_p^3\right) \tag{4}$$

Combining the Equations 2 and 4 above leads to a particular definition of the densimetric Froude number of particles (Fr_p). The velocity (U_p) and can be called the terminal velocity (U_{p0}) of granular soil particle, if the fluid is at rest and the particle scroll down, as well as upward if the particle is less dense than the fluid. The velocity U_{p0} is expressed as a function of the Reynolds number of particles (Re_p) and the drag coefficient (C_d). For the sake of reference, the drag coefficient may be experimentally obtained from a simple test with a small amount of sand in free fall through a water column.

$$U_{p0} = \sqrt{\frac{4}{3}\frac{(\rho_s\text{-}\rho_f)\,g\,d_p}{\rho_f C_D}} \quad (5)$$

From the available experimental data, the particle free fall velocity has been estimated for $d_p = d_{50} = 0{,}207\text{mm}$ as U_{p0} = 27 mm/s, with Reynolds number $Re_p \approx 5.6$ and drag coefficient $C_d \approx 6$.

However, it has been noticed that, being the terminal velocity, U_{p0} higher for larger sand particles, the choice of d_{50} as a representative grain size is not suitable. The upward flow velocity must be able to carry also the largest particles; otherwise the flow would act as a sieve carrying only the fine grains, being the fluidization process stopped by large particles accumulating at the bottom of the fluidization zone. For this reason it is herein proposed to define $d_p = d_{95}$ as the maximum particle size thoroughly present in the soil. If $d_p = d_{95} \approx 0.5$ mm, then U_{p0} = 73 mm/s, with Reynolds number $Re_p \approx 36$ and drag coefficient $C_d \approx 2$.

From the previous equations, by setting $U_p = U_{p0}$, the maximum external diameter of the fluidized zone may be estimated as:

$$d_{h0} = \sqrt{\frac{4}{\pi}\frac{Q_0}{U_{p0}} + d_e^2} \quad (6)$$

where the following auxiliary definitions have been used:

$$A_h = \frac{\pi}{4}\left(d_h^2 - d_e^2\right) \quad (7)$$

$$U_p = \frac{Q_0}{A_h} = \frac{4}{\pi}\frac{Q_0}{\left(d_h^2 - d_e^2\right)} \quad (8)$$

being Q_0 the (experimentally controlled) flow rate, A_h the free cross section area surrounding the fluidization probe (fluidization zone), which is approximately a circular diameter, d_h the fluidization zone around the pile shaft and d_e the external diameter of the pile.

Hence, the condition $d_h < d_{h0}$ implies that the flow is fast enough to drag the largest sand particles upward, what makes fluidization possible. It is important to observe that this condition disregard the losses due to flow percolation, which means that Equation 9 probably overestimates d_{h0} by a certain amount.

The most unfavorable case is given by a velocity U_{p0} equal to 7.3×10^{-2} m/s, for $d_p = d_{95}$, corresponding the greater volume of the particle to be suspended by fluidization. For applied flow rates of 0.7, 1.0 and 1.6 L/min and the external diameters of the piles 16.2 mm. The predicted values of the fluidized zone diameter, d_{h0}, in the center of the tank are expressed by Equation 6.

In order to compare the measured results (Figure 6) from the tests carried out at the lateral of the tank, for half of the cross section area surrounding the fluidization probe (fluidization zone), Equation 9 is applied.

$$d_{h0} = \sqrt{\frac{8}{\pi}\frac{Q_0}{U_{p0}} + d_e^2} \quad (9)$$

The geometry of the fluidized zone for free fall piles can be expressed by the set of equations denned in the present paper. A flow rate equal zero yields $d_{h0} = d_e$. For other flow rates, the predicted values of d_{h0} are close to measured values of d_h from fluidization tests with free fall piles at the wall of the acrylic tank (Figure 6), with around 25% difference between measured and predicted.

Despite the fact that some flow radially percolate from the fluidized zone to the nonfluidized zone, would leads to expect values of d_h measured at the edge of the tank smaller them d_{h0} predicted, both values were similar. In addition, the equation for predicted d_{h0} leads to the increasing of the perturbed zone diameter with the increase in the flow rate Q_0, which does not correspond to laboratory results, what is acceptable since it is a theoretical equation.

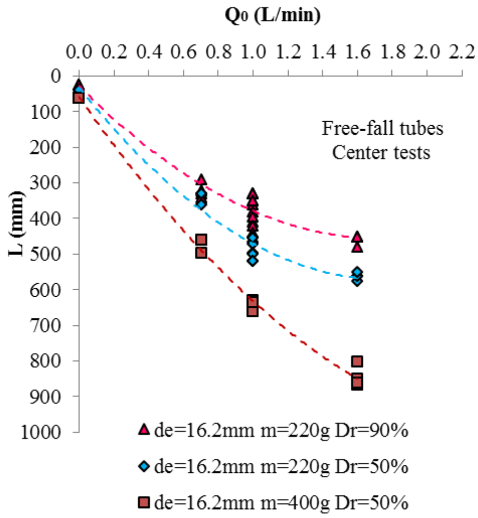

Figure 5. Pile penetration depths expressed as a function of flow rate at D_r = 50% and 90%.

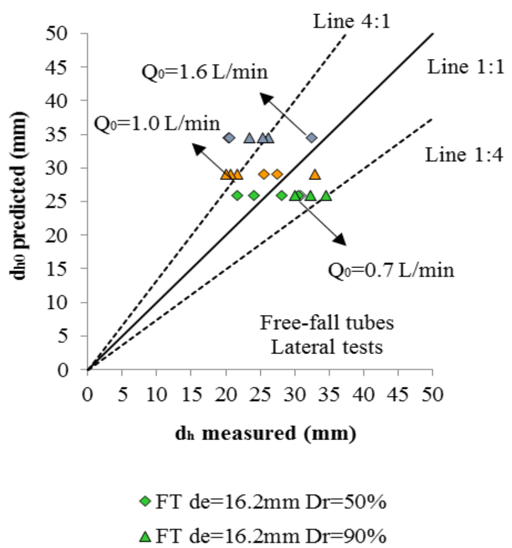

Figure 6. Comparison between measured d_h and predicted d_{h0} (diameter of the fluidized zone).

Considering the above observations, a straightforward procedure may now be proposed to estimate the penetration depth, L, from the relevant fluidization parameters. In Figure 7, the maximum external diameter of the fluidized zone, d_{h0}, calculated from Equation 6 (center tests) and normalized by the particle diameter d_{95}, and squared is plotted against the penetration length, L, normalized by the tube external diameter, d_e. It is shown that:

$$\frac{L}{d_e} \cong C \left(\frac{d_{h0}}{d_{95}}\right)^2 \text{ for } C = 0.010 \text{ and } 0.018 \qquad (10)$$

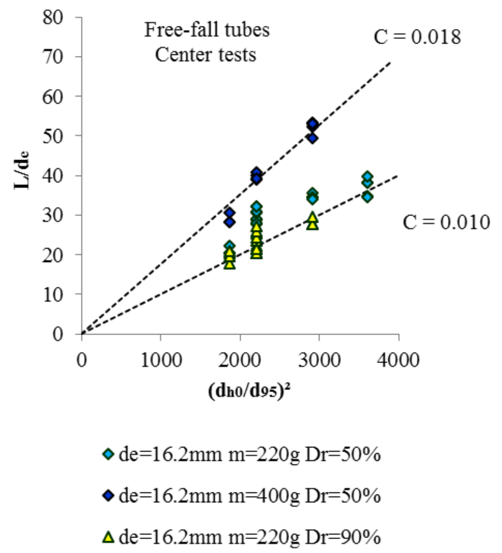

Figure 7. Normalized L/d_e *versus* $(d_{h0}/d_{95})^2$ from fluidization center tests at D_r = 50% and 90%.

From Figure 7 can be noticed that two well defined groups are formed. One group represented by constant C = 0.010 from results of d_e = 16.2 mm, m_A = 220 g, at D_r = 50% and d_e = 16.2 mm, m_A = 220 g, at D_r = 90%; and another by constant C = 0.018 from results of d_e = 16.2 mm, m_B = 400 g, at D_r = 50%. The critical depth of installation showed to be achieve from around 15 until 50 times the pile external diameter, depending on the pile dry mass and diameter, applied flow rate and soil relative density.

Some considerations are important to be mentioned: (i) only data from installation by fluidization, carried out in the center of the tank, are considered, i.e. for L results from $Q_0 \neq 0$, at D_r = 50% and 90%; and (ii) despite the fact that the geometry of the fluidized zone is independent of the relative density, the installation depth is, although the Equation 10 does not allow to visualize it.

There are no experimental information for corroborating the use of d_{95} in the normalization of d_{h0}. The choice was made based on conceptual arguments: the larger the sand particles, the larger will be the percolation losses and the smaller shall be the penetration length. This argument, as well as the proposed normalization, is still to be further investigated.

It must be emphasized that the diameter of the fluidized zone, d_{h0}, is not an observed experimental parameter, rather a reference quantity derived from other fluidization parameters for tests carried out in the center of the tank. It is supposed to the magnitude of the actual diameter, d_h, is always overestimate due to percolation losses.

4 CONCLUSIONS

The article presents data from laboratory model tests undertaken to assess the installation response of fluidized piles in saturated sand. Model tests indicate that the width of the fluidized field is not influenced by the initial density of the sand and the perturbed zone is constraint to a distance of about two diameters from the pile during installation independent of the fluidization flow rate applied. An expression for the diameter of the fluidized zone is proposed as a function of the pile diameter, fluidization flow rate, soil and fluid parameters. The critical installation depth slowed to be related with the width of the fluidized field. For a given soil relative density, increasing the weight of piles and the applied flow rate produces an increase in installation depth. Under fluidization, model tests reached a maximum penetration depth of the order of 50 times the pile diameter.

ACKNOWLEDGEMENTS

The authors wish to express their gratitude to the Brazilian Oil Company Petrobras for the financial support of the research group (UFRGS Ref. 4600346071: Offshore Technology: Foundations, fluidization and ground improvement). The Coordination for the Improvement of Higher Education Personnel – CAPES and the Brazilian National Council for Technology Development – CNPq for the financial aid during the research project.

REFERENCES

Fluke Subsea, Date of access: February 27th, 2016. http://www.flukesubsea.com.br/pt/acessancor.php

Mezzomo, S. M. (2009) *Study of Fluidization Using Water Jets in Sand*, M.Sc. thesis, Federal University of Rio Grande do Sul, Porto Alegre, Brazil. In Portuguese.

Passini, L. (2015) *Installation and Axial Load Capacity of Fluidized Model Piles in Sandy Soils*, Ph.D. thesis, Federal University of Rio Grande do Sul, Porto Alegre, Brazil. In Portuguese.

Passini, L., Schnaid, F. (2014a) Torpedo pile installation by vertical jet fluidization in granular soils, *XVII Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica* – COBRAMSEG, Goiânia, Brasil, 7p.

Passini, L., Schnaid, F. (2014b) Uplift capacity of torpedo piles installed by soil fluidization through vertical jets, *XVII Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica* – COBRAMSEG, Goiânia, Brasil, 7p.

Passini, L., Schnaid, F. (2015) Experimental Investigation of Pile Installation by Vertical Jet Fluidization in Sand, *Journal of Offshore Mechanics and Arctic Engineering* – OMAE, 137(4), pp. 1–10.

Passini, L., Schnaid, F., Salgado, R. (2015) Axial load capacity of model piles in sand. *15th Pan-American Conference on Soil Mechanics and Geotechnical Engineering* – PGSMGE, Paper ABS-1433, 8p.

Schnaid, F., Passini, L., Stracke, F., Mezzomo, S. (2014) On the Response of Fluidized Piles from Laboratory Model Tests in Granular Soils, *Journal of Geo-Engineering Science*, 1(2), pp. 69–81.

Stracke, F. (2012) *Fluidization of Sand Associated to Injection of Cement Agent for Applying in Offshore Structures*, M.Sc. thesis, Federal University of Rio Grande do Sul, Porto Alegre, Brazil. In Portuguese.