EXPERIMENTAL STUDY OF STABILITY OF SUBMARINE OUTFALLS IN MUDDY SOILS

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Summary

Marine structures built on muddy soils, such as breakwaters and submarine outfalls, are subject to settlement, scour and liquefaction and the mechanics of these processes are still uncertain. The aim of the present experimental study is to investigate stability of submarine outfalls in low quality soil. Physical modeling, carried out in the wave flume at UNAM, Mexico, is described and the results are presented and discussed. Tests were performed for different wave conditions (wave heights and periods), different soil characteristics (several mixtures of kaolinite and sand) and three structural layouts: the pipe resting on the soil, the pipe partially buried and a totally buried pipe.

The pore pressure for different positions on the soil and the movements of the pipes were measured, together with surface elevation along the flume.

Keywords

Low quality soil, submarine outfalls, physical model, liquefaction, waves

Introduction

The good operative working of a marine outfall is extremely important for the environment, for the welfare of the population and for the local economy. The structure must be safe and reliable throughout its lifetime. One of the factors influencing this is the variations of strength and soil behavior due to direct wave action.

Although in recent decades the understanding of the flow and erosion around marine structures has improved, little is known about the impact of soil liquefaction on these structures (Sumer, 2006). With the European project LIMAS (Liquefaction around Marine Structures), important steps were taken in the study of the failure modes of different marine structures, including submarine pipelines, due to wave induced liquefaction. Physical model tests and theoretical work were conducted, focused on the processes that occur during soil liquefaction. However, these tests were made mainly for a soil composed of sand or silt. For marine structures, such as breakwaters and submarine outfalls, built on muddy soils, there is still great uncertainty, especially related to settlement, scour and liquefaction.

To understand the impact of soil liquefaction on marine structures, including submarine outfalls, the project "AREDIS - Adjustments to Strengthen Stability of Breakwaters and

Submarine Outfalls in Muddy Soils" was established by UNAM, UGR, LNEC, WW, Consultores de Engenharia, SA, PROES and Apleph Ingenieros Consultores SA in 2011 . The main areas of focus for the present paper are: i) to analyze the soil-pipe-wave interaction, specifically in the areas of erosion, liquefaction and loss of soil strength; and ii) to apply the knowledge acquired to solve typical problems in the construction and operation of submarine outfalls on muddy soil.

To achieve these objectives an experimental program is currently underway at the Institute of Engineering at UNAM (Mexico). The experimental program, where the outfall structure is simulated through a circular pipe with a diameter of 0.03 m, consists of two-dimensional (2D) tests, with different wave heights and periods, comprising three structural layouts: a pipe resting on the soil, a pipe partially buried and a totally buried pipe.

This article describes the work that has been done so far by the project members, which focuses on 2D physical model tests of pipelines set in a muddy soil.

Experimental set-up

A two-dimensional experimental study is being performed in the UNAM laboratory in a wave flume 22.0 m long, 0.4 m wide and 0.6 m deep (Figure 1). The flume is equipped with an active wave absorption system and a passive dissipation gravel beach placed at the end of the flume.

The soil is placed in a soil tray 0.2 m high, 0.307 m wide and 0.9 m long (Figure 1), filled with different combinations of materials, and located 13 m from the wave-maker.

The outfall is simulated by a circular PVC pipe, with an outside diameter of 0.03 m and a length of 0.28 m, placed in the soil box.

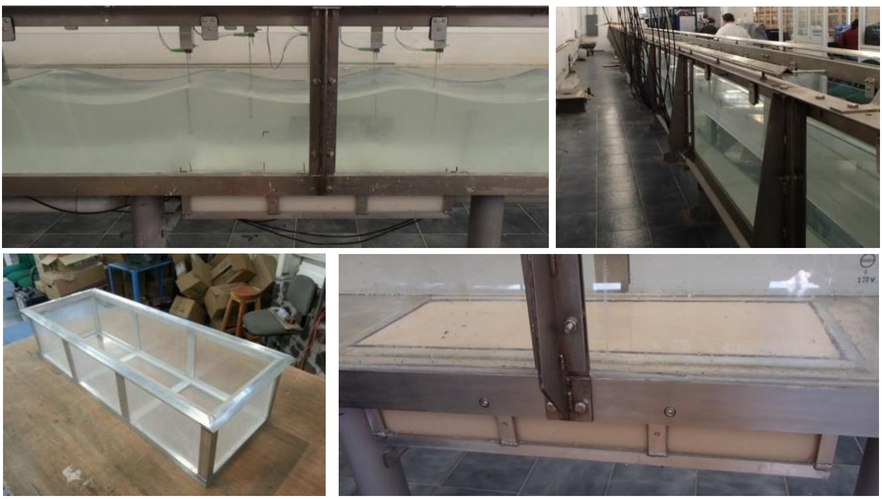

Figure 1. Aspects of the UNAM wave flume and the soil tray.

The tests were carried out with a constant water depth of 0.3 m, different wave conditions (wave heights and periods) and different soil characteristics (several mixtures of kaolinite and sand). Three structural layouts were tested: the pipe resting on the soil, the pipe partially buried and a totally buried pipe.

Surface elevation is measured with wave gauges positioned at 12 points along the flume. Pore pressure of the soil is measured in 4 soil tray depths and in 7 sections, making a total of 28 pressure sensors (Figures 2 and 3). The measurements are synchronized using a SPARTAN datalogger, which enables the integration of 48 analog and 16 digital signals, establishes a sampling rate of up to 500 Hz, calibrates the signals, determines trigger conditions and performs real-time calculations.

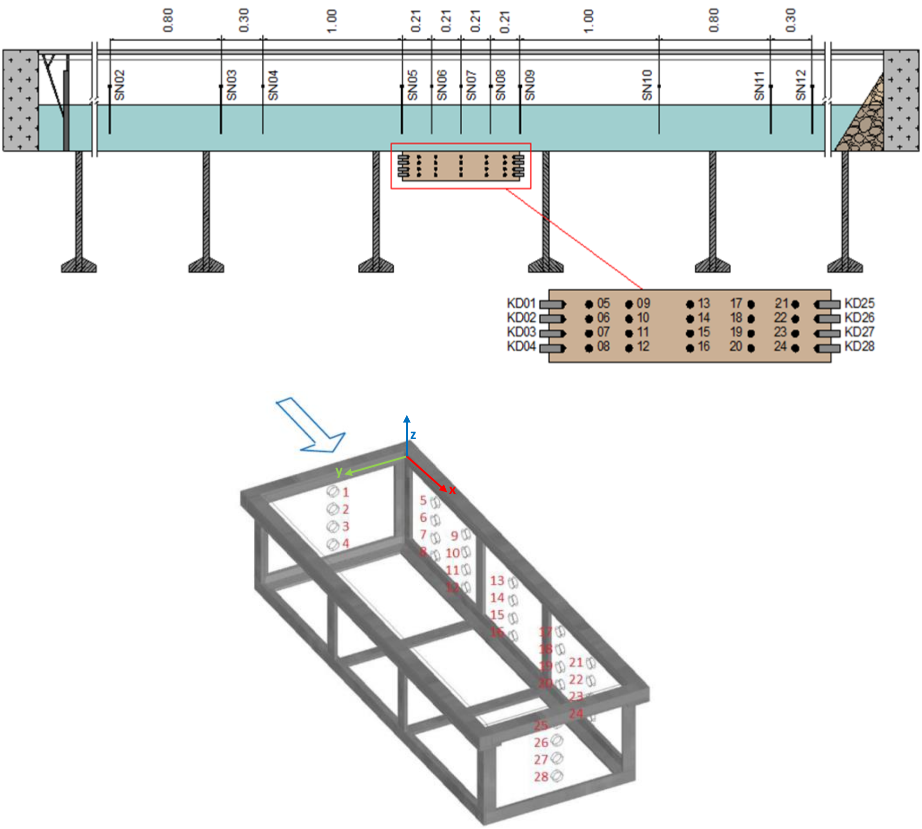

Figure 2. Schematic location of the equipment used in the tests: SN# - wave gauges, KD# - pressure sensors. Dimensions in (m).

Figure 3. Aspects of the flume with the equipment used in the tests: wave gauges and pressure sensors.

Five different types of soil are used, ranging from a natural sand, identified as 100A, taken from the beach at Puerto Morelos, Quintana Roo, Mexico, to a mud, a commercial kaolinite, identified as 100C. Between these, three different combinations of sand (A) and mud (C) are tested: 85% sand – 15% kaolinite (85A15C), 60% sand – 40% kaolinite (60A40C) and 30% sand – 70% kaolinite (30A70C). Some of the mechanical properties of the soils, estimated in the laboratory, are shown in Table 1, where e_{max} and e_{min} are the maximum and minimum void ratios, ρ_s is the soil mass density, ρ_w is the water mass density and d_{50} is the value of the particle diameter at 50% in the cumulative distribution.

Table 1: Soil properties.

	ρ_s/ρ_w	d_{50} (mm)	e_{max}	e_{min}
100A	2.85	0.267	1.845	0.992
85A15C	2.82	0.245	1.551	0.795
60A40C	2.73	0.180	1.998	0.726
30A70C	2.60	0.070	2.383	0.995
100C	2.17	0.002	3.032	1.100

Soil behavior

The Technical Standards and Commentaries for Port and Harbor Facilities in Japan (2002) presented a graphic which show the soil properties that make it prone to liquefaction. Based on the analysis of the granulometric curves of the 5 types of soil used in the tests (Figure 4) and the comparison with the mentioned zones, it was expected to have liquefaction for the soils with a percentage of kaolinite smaller or equal to 30%, with the uniformity coefficient $U_c=d_{60}/d_{10}$, and d_{60} e d_{10} denote the grain sizes corresponding to 60% and 10% passing, respectively. However, according to the criteria of the initial mean normal effective stress (Sumer et al., 2012) no liquefaction should occur for these soils. Furthermore, the critical mean normal effective stress, σ_0', shows that no liquefaction potential is present within the soil mass for the soil with a percentage of sand greater than 15%, i.e., the maximum period average pore pressure, $\bar{p}$, reached during the tests does not exceed σ_0', given by:

$$\sigma_0' = \gamma' z \frac{1+2k_0}{3} \tag{1}$$

with z the depth into the soil, γ' the submerged specific weight of the soil and k_0 the ratio of horizontal to vertical effective stress. These soil parameters were determined in the laboratory (e.g. Chavez et al., 2014).

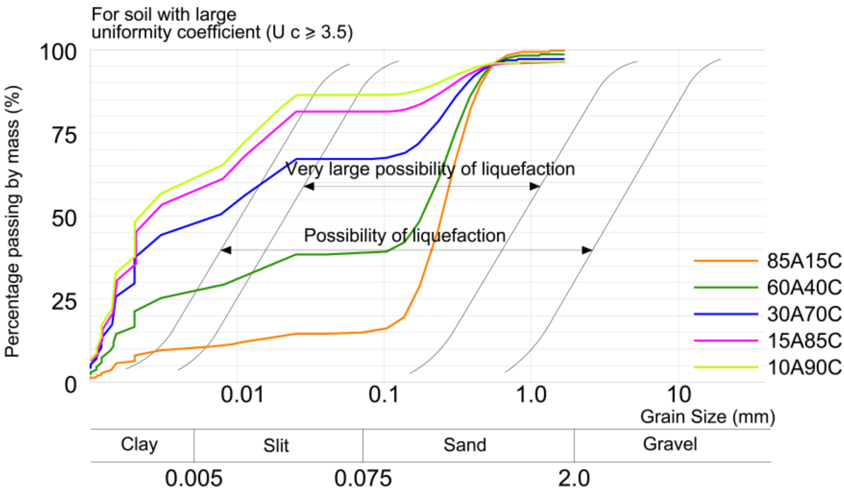

Figure 4. Granulometric curve of soils used in the tests vs curves presented in the Technical Standards and Commentaries for Port and Harbor Facilities in Japan (2002).

The soil response under progressive waves is considered to be independent along the x-axis in the flume (Figure 2) (Kirca et al., 2013). However, for the experimental conditions developed in the tests, a spatial variation of the pore pressure is noticed. It is also observed that, as consolidation continues, the overall maximum pore pressure values tend to decrease, indicating that the failure potential is reduced and the soil will not liquefy under the tested conditions.

Preliminary tests

Preliminary tests (Chávez, 2014) were made without a structure in order to analyze the response of the different types of soils in terms of pore pressure (Figure 5):

- firstly, analyzing the influence of different contents of mud (see Table 1), and
- secondly, analyzing the influence of the initial water content in the soil at 100C, with different solids-water concentrations:
    - 1200 kg/m^3 (100C-1.2),
    - 1500 kg/m^3 (100C-1.5) and

- 1800 kg/m^3 (100C-1.8).

Figure 5. Aspect of the soil after the preliminary tests without a structure: 100A (left), 30A70C (right).

The wave conditions consisted of different wave periods, ranging from 1.2 s to 1.5 s, different wave heights, ranging from 0.04 m to 0.13 m, and a constant water depth of 0.30 m. Tests were made 3 days after filling the flume, in order to allow the soil to consolidate.

With regard to the values of the dimensionless parameters, the UNAM tests were generally within the range of values presented in the literature. Table 2 summarizes the values of the dimensionless parameters in previous tests and for the UNAM tests. In this table, Re is the Reynolds number, KC is the Keulengan-Carpenter number, D is the pipe diameter, h is the water depth, H is the wave height, L is the wave length and d is the thickness of the soil layer.

Table 2: Dimensionless parameters for the UNAM tests and for pervious tests.

Dimensionless parameter	Previous tests*		UNAM tests
	min	max	
Re	1.08E+04	6.52E+06	1.06E+04
KC	1.01E+00	3.69E+03	1.59E+01
h/L	9.05E-02	3.31E-01	1.46E-01
H/L	1.23E-02	5.89E+00	5.35E-02
d_{50}/L	1.56E-05	4.45E-05	3.16E-04
H/D	3.13E-01	8.50E+02	3.67E+00
d/L	5.89E-02	4.37E-01	9.72E-02
D/h	4.76E-02	5.33E-01	1.00E-01
D/d	1.14E-01	4.71E-01	1.50E-01

*Sumer et al. (1999, 2006a, 2006b), Teh et al. (2003, 2006), Foda et al. (1988), Kumar et al. (2005)

The main differences are of d_{50}/L for the UNAM tests, which is slightly higher than the values for the tests found in the literature, and on the type of soil, since most of the previous tests were performed with a soil composed of sand or silt, except in Kumar et al. (2005), where clay was used.

The results of the preliminary tests without a pipe showed that:

- For a soil with high mud content (60A40C, 30A70C and 100C), the water-pore pressure accumulates in the soil mass and the so-called “buildup” was present.
- For 100A and 85A15C, as was expected theoretically, the pore pressure amplitude increased with the wave height but no residual pore pressure accumulation was observed for the granular material.
- Tests with different initial water content in the mixture showed that this is a critical condition for the soil response.
- In all tests, soil liquefaction was observed, with the formation of a liquefied mud layer and considerable sediment suspension was also noted that the maximum pore pressure, as well as the critical values, take longer to be reached with the increasing solids-water concentration

For the tests with a pipe, some preliminary tests were also made with a soil with a constant percentage of kaolinite, 15A85C and an initial water content of 2.00 kg/l. The tests were made for T = 1 s and H = 0.12 m, with a pipe with specific gravity, s_p=1.8, which is a typical value of an outfall made of HDPE with concrete weights. In these tests, the pipe was resting on the bed or buried 1.5, 3.0, 4.5, 6.0 and 9.0 cm from the soil surface. Some liquefaction conditions were obtained in these cases, the three pipes rotates and sinked totally, as can be seen in Figure 6.

Figure 6. Aspect of the pipes at the beginning (top) and at the end (bottom) of the preliminary tests.

Figure 7 presents, in the top panel, the time series of surface elevation on wave gauges 5 (at the beginning of the soil tray) and 10 (after the soil tray) (see Figure 2), where only a slight shoaling of the waves can be seen due to the presence of the model. In the bottom panel of Figure 7 the time series of pressure at pressure sensors 13 (located in the center of the soil tray; and closest to the mud-line) and 16 (located in the center of the soil tray and closest to the bottom of the soil pit). In these time series it can be seen that a buildup pressure occurred, along the mud depth, from the beginning of the tests with a peak of 12 cm H_2O reached after nearly 210 s of wave action. After 300 s of wave action the pressure was released and the soil seemed to have lost its loading capacity, thus indicating that possibly the soil structure failed. The movement of the pipes (burring and rotation) is also evidence of soil failure.

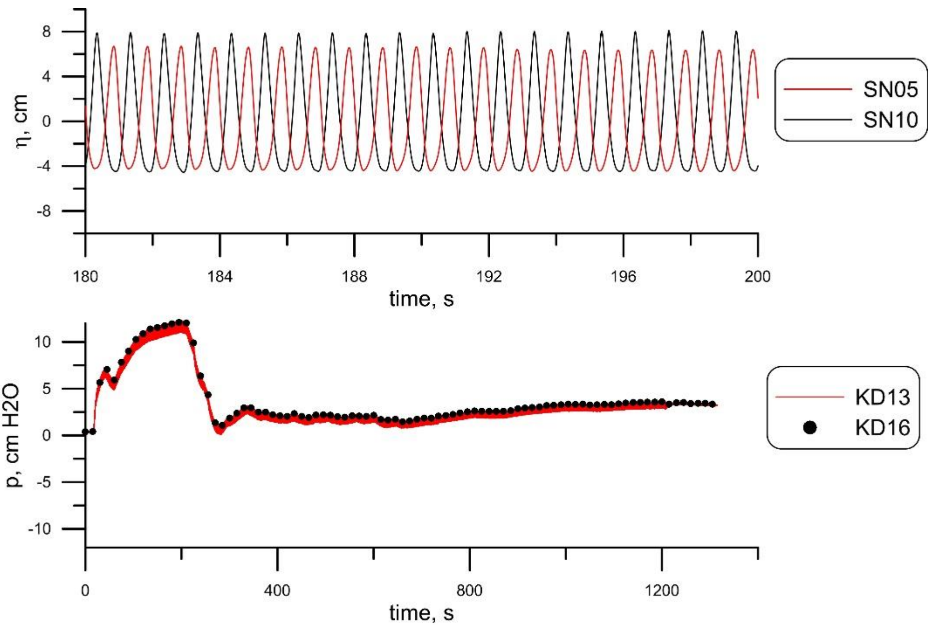

Figure 7. Surface elevation on wave gauges 5 (red line) and 10 (black line) and pressure at pressure sensors 13 (red line) and 16 (black dots).

The results of the preliminary tests with a pipe and in a soil 15A85C with an initial water content of 2.00 kg/l showed that:

- For the tested conditions, the water-pore pressure accumulates in the soil mass and the so-called buildup pressure was present for some pressure positions.
- The three pipes sink into the soil and rotate.

Test program

After carrying preliminary tests to analyze the soil behavior, using different combinations of sand and mud and different values of initial water into the soil, some conclusions were drawn and improvements were made in the physical model setup. The most important improvements were: i) the enlargement of the soil box, in order to increase the distance of the soil before the structure, and ii) fixing the pipes in order to allow only vertical movement.

The tests presented here are the last tests made, after the enlargement of the soil box, now with 5 cm more in height (25 cm in total) and 45 cm in length at each side of the pit (180 cm in total), and with semi-fixed pipes. Figure 8 shows the old and new setups of the model close to the soil box.

Figure 8. Flume during the preliminary tests (top) and with the soil tray enlarged and with the three structures semi-fixed to reduce the horizontal movements of the pipes (bottom).

With the new soil tray, the pressure sensors were located at 9, 13, 17 and 21 cm from the soil-water surface. For those depths, the values of the critical mean normal effective stress, σ_0' (Eq. 1), are 5.21, 7.64, 10.00 and 12.35 m H_2O.

The soil used in these tests was 15A85C – 2.0 kg/l and has a specific soil gravity, *s*=2.82. Three pipes were deployed at the flume: one resting on the bed (pipe 3), one partially buried (1/2D, pipe 2) and the last totally buried (pipe 1, see Figure 9).

Tests were performed with regular incident wave conditions, with two wave periods and two different wave heights, and a water depth of 0.30 m. The duration of the test was around 1200 s. Table 3 summarizes the wave characteristics of tests 13 to 16, where a is the wave amplitude

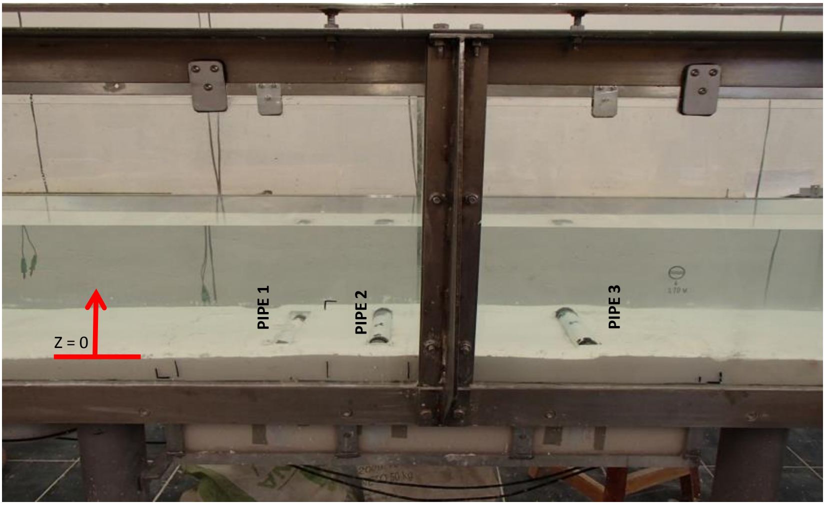

Figure 9. Pipes in the flume at the beginning of the tests.

Table 3: Wave conditions.

Test	T (s)	H (m)	a (m)
13	1.00	0.12	0.06
14	1.00	0.15	0.075
15	1.20	0.12	0.06
16	1.20	0.15	0.075

Results

Tests 13 to 16 (see Table 3) were made sequentially, without changing anything in the soil. As a result, the soil in the last test shows the cumulative soil behavior. As explained earlier, during the tests, the pore pressure was registered at z = 9, 13, 17 and 21 cm, along seven columns distributed in the soil tray (see Figure 2), as well as the surface elevation along the flume. The pipe position was determined at the end of each test.

The overall analysis of the time series of the pressure shows that:

- the pressure exceeds σ'_0 in some of the sensors in the first and second lines and seems to reduce with depth;
- with the progress of the tests, the pressure exceeds the value of σ'_0 further away from de paddle "downstream" and the pressure seems to decrease.

The visual analysis of the soil during the test shows that the soil behaves as a fluid. This leads to the conclusion that the pore pressure close to the soil-water interface was higher than the measured pressure at the sensors and that liquefaction occurred.

Analyzing test by test, for test 13, pressure clearly exceeds σ'_0 in sensors 13, 17 and 21 (first line), and 18 and 22 (second line), see Figure 10.

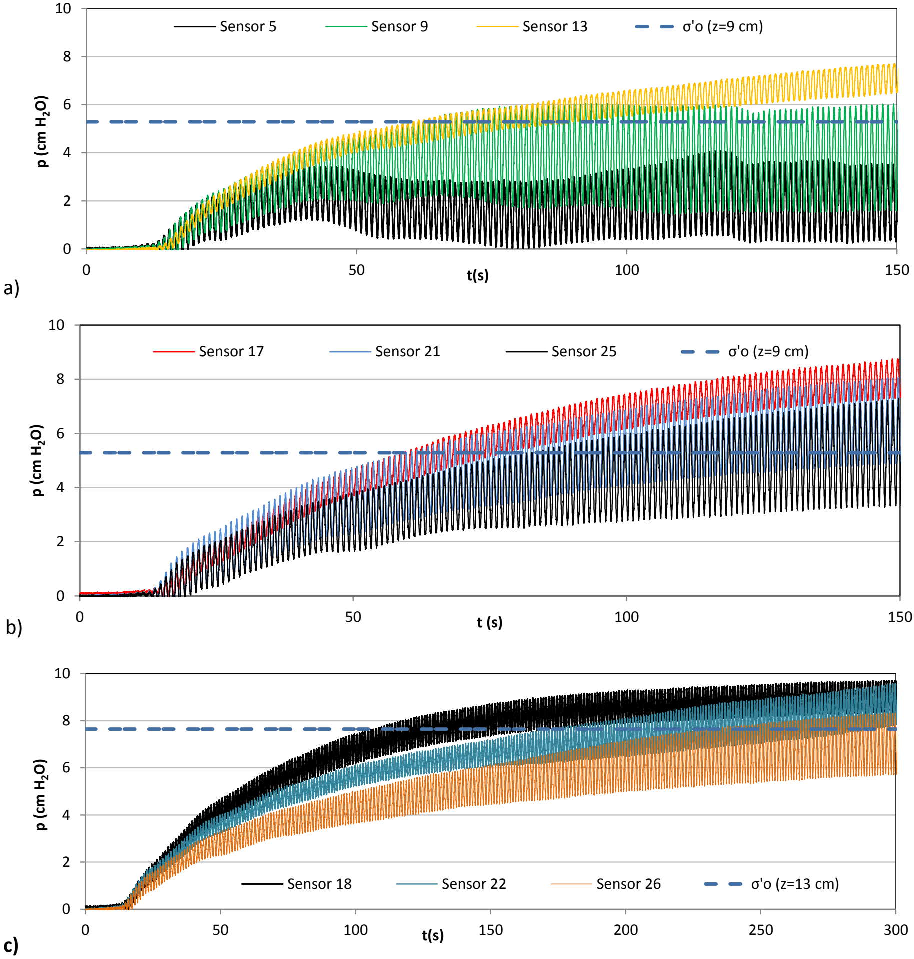

Figure 10. Time series of pressure measured in 1st line sensors (a and b) and 2nd line sensor (c) during test 13.

All sensors are located in the second half of the tray. For tests 14 and 15, only in sensors 21 and 25 (first line) and 22 (second line) the pressure exceeds σ'_0. Finally, for test 16, only in sensors 21 (first line, see Figure 11) and 22 (second line) the pressure exceeds σ'_0.

With reference to the pipe position, pipe 1 was totally buried at the beginning of the tests (see Figure 9) and was placed between sensors 5 to 8 and 9 to 12, which, except for sensor 5 (see Figure 10), did not register an important increase in the pore pressure during the tests.

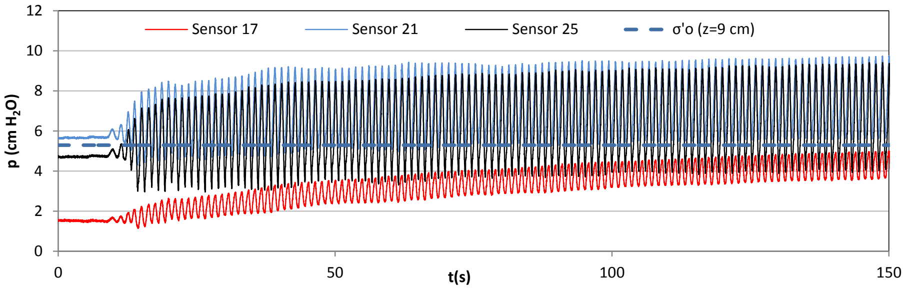

Figure 11. Time series of pressure measured by sensors 17, 21 and 25 during test 16.

Pipe 2 was half buried in the beginning of the tests and was placed between sensors 9 to 12 and 13 to 16, in which only on the 1st line the pressure exceeded σ'_0.

Pipe 3 was placed resting on the bed, even though it quickly sank 1 cm at the very beginning of the tests, and was placed between sensors 17 to 20 and 21 to 24, where in all tests, at least at sensor 21 (see Figure 11) the pressure exceeded σ'_0.

The pipes changed their position as the tests progressed. Figure 12 shows the mean position of each pipe at the end of each test (13 to 16), with the initial position referred to as test 12. The mean position was calculated based on the displacement measured at the two extremes of the pipe, which was not always the same.

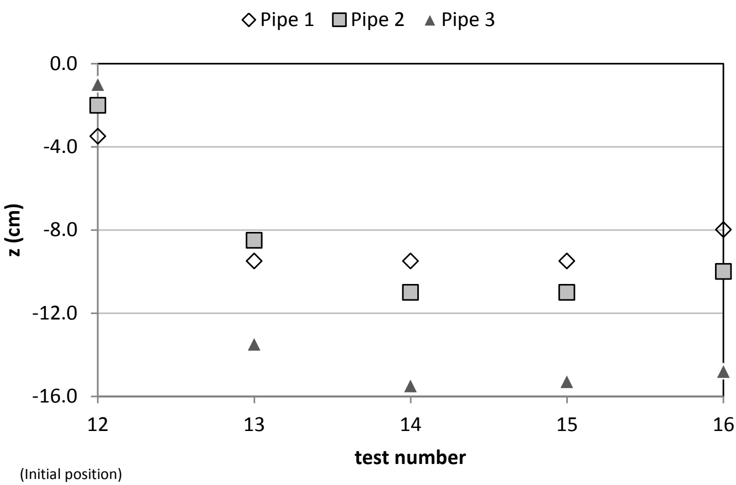

Figure 12. Medium pipe position after the different tests.

As can be seen, test 13 was the test where the largest movements were registered for the three pipes. However, the pipes presented different behavior:

- Pipe 3 was almost resting on bed at the beginning of the tests and sank 10.5 cm during test 13. After that it had displacements of less than 1.1 cm.
- Pipe 2 was initially half buried and sank 5.8 cm during test 13, sinking less from test to test, except in test 16 when it moved upwards by 0.5 cm.
- Pipe 1 was the pipe that was totally buried at the start and showed similar behavior to pipe 2, sinking in the first three tests and moving upwards 1.1 cm in test 16.

After test 16 the position of the three pipes differed less than 4 cm. However the initial relative position between the pipes was changed: the pipe that was initially buried was further up than the pipe that was originally resting on the bed.

Sumer et al. (2006 a) stated that, for a silt soil, a pipe floats for s_p less than 1.85-2.0. Teh et al. (2006) proposed an expression for determining the minimum specific gravity for the pipe to become self-buried, $s_{liq}=(s+e_{cr})/(1+e_{cr})$, where $e_{cr}=e_{max}$, $s_{liq}=1.7$. Since $s_p=1.8$ and $s_p>s_{liq}$, the pipe is expected to become self-buried as happened in the experimental tests.

In fact, there was no flotation for any pipe in tests 13 and 14. Pipe 1 started moving upwards during test 15 and all pipes seemed to move upwards slightly after test 16. In fact, pipe 1 stabilized around z=-8.0 cm, indicating, accordingly to Sumer et al. (2006 a), that the specific gravity of the liquefied soil between distances z=-3.5 to -8.0 cm is equal to that of the pipe. For pipe 2 these distances are z=-2.0 to -11.0 cm and for pipe 3 z=-1.0 to -15.0 cm. This means that the degree of liquefaction increases with the distance from the beginning of the soil box.

Figure 13 to 15 shows the initial and final depth of pipe 1, 2 and 3, respectively, normalized with pipe diameter as a function of H.

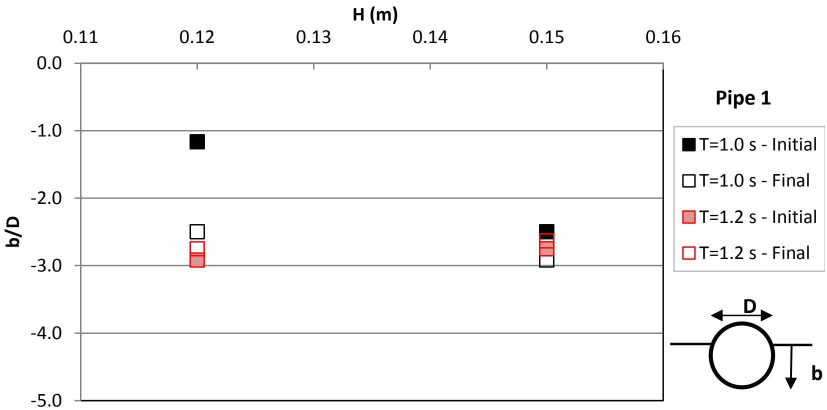

Figure 13. Pipe embedment due to different wave conditions for pipe 1.

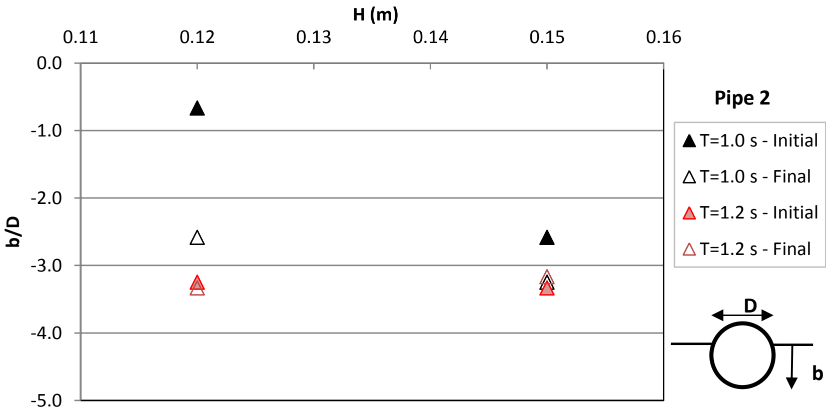

Figure 14. Pipe embedment with different wave conditions for pipe 2.

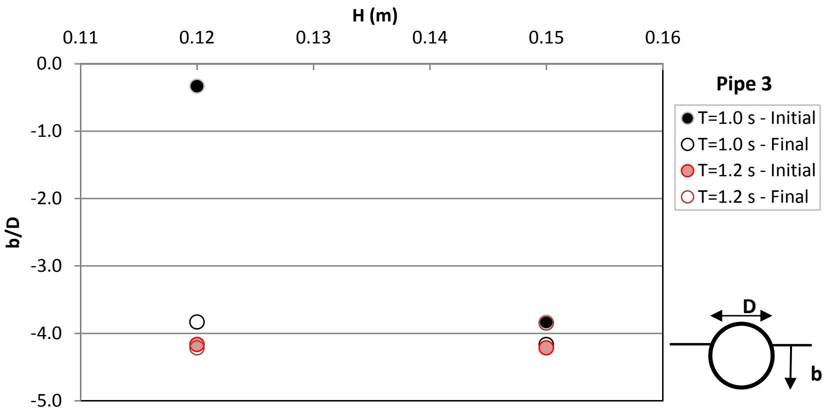

Figure 15. Pipe embedment with different wave conditions for pipe 3 .

As can be seen, the pipe sinks to a depth that depends on the test conditions, being larger, as referred before, for the first test (T=1.0 s and H=0.12 m) However, it does not seem to depend on H. The same result was presented in Teh et al. (2003) and Sumer et al. (1999), whose tests were made with a pipe of specific gravity 1.11.

The final depth of the pipe increased from pipe 1 to 3. Since the pipes differ in the initial embedment, decreasing from pipe 1 to 3, this suggests that the initial position can affect the final position for a constant pipe specific gravity. This is not in accordance with Teh et al. (2003), that says that the initial position is not a determining factor in the final pipe embedment. However they were comparing cases with different H but the same T and the tests were not made consecutively.

Conclusions

This paper presents a description of the objectives and the work done so far within the project AREDIS (Adjustments to Strengthen Stability of Breakwaters and Submarine Outfalls in Muddy Soils). This work is based on an experimental tests intended to complement the present understanding concerning the behavior of marine structures built on muddy soil and make recommendations for the design of these works, including breakwaters and outfalls.

Tests were carried out in a two-dimensional flume in the UNAM laboratory, simulating the outfall through a circular PVC pipe, with an outside diameter of 0.03 m and a length of 0.28 m.

Firstly, an extensive program of preliminary tests was done for different wave conditions (wave heights and periods), different soil characteristics (several mixtures of kaolinite and sand), with and without the pipe. After those tests were completed, some adjustments were made to the setup of the flume and the tests with the pipe were started.

Four tests were performed with three pipes: one almost resting on the bed, another half-buried and the last pipe totally buried. The results show that:

- Liquefaction seems to occur at least close to the bottom;
- The pipes sank especially in the first test, with smaller movements in the next three tests;
- The pipe that was initially almost resting on the bed, located after the larger extension of soil in the flume, was the one that sank most.

More tests are to be made in the coming months to further the knowledge on the behavior of the pipe in low quality soil and to confirm the conclusions presented here.

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