The paper was published in the proceedings of XVI Pan-American Conference on Soil Mechanics and Geotechnical Engineering (XVI PCSMGE) and was edited by Dr. Norma Patricia López Acosta, Eduardo Martínez Hernández and Alejandra L. Espinosa Santiago. The conference was held in Cancun, Mexico, on November 17-20, 2019.

Geotechnical Engineering in the XXI Century: Lessons learned and future challenges
N.P. López-Acosta et al. (Eds.)

doi:10.3233/STAL190330

Geotechnical Forensic Engineering: Settlement of Structures in Industrial Facilities Located in Puerto La Cruz, Venezuela

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Abstract. The Forensic Geotechnical Engineering (FGI) treats the cases in which during the construction or the useful life of a project, failures associated with the soil-structure interaction have been presented. In this work, a case of FGI occurred in industrial facilities located in Puerto La Cruz, Venezuela, during 2017. In these installations, settlements up to 180 mm were recorded in several structures supported by isolated direct foundations with an embedment depth Df = 3 m. Through the application of the Scientific Method, and from the analysis of the available topographic information, the realization of boreholes, pits and *in situ* density measurements in several sectors of the studied area, as well as compaction tests in the laboratory at different energy levels, the causes of the ocurred settlements were determined. Subsequently, it was recommended that foundations be reinforced with micropiles, installed and connected to existing supports. Finally, the geotechnical design and the construction method used for the micropiles were verified by means of load tests, in order to ensure the functionality of the structures with the new foundation system, during the useful life of the project.

Keywords. Forensic Geotechnical Engineering, scientific method, micropiles.

1. Introduction

The Forensic Geotechnical Engineering (FGI) treats the cases in which, during the construction or useful life of a project, failures associated with the soil-structure interaction have been presented. In general, an FGI project focuses on the application of a methodology that presents the following sequence: 1) analysis of available information; 2) description of the fault; 3) analysis of the causes; 4) proposed foundation solution; 5) lessons learned. This sequence can be considered as the application of the Scientific Method to a geotechnical problem, since it covers the observation of a phenomenon (in this case the failure), the analysis of existing information, the approach of possible hypotheses to analyze the causes, and the necessary research and additional experimentation to probe these hypotheses. Subsequently, once the causes of the failure

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are understood, an economically viable technical solution is proposed, and finally the lessons learned regarding the failure are recorded [1].

In this paper, a case of FGI occurred in industrial facilities located in the city of Puerto La Cruz, in the northeastern region of Venezuela, during 2017 is presented, in which the aforementioned methodology was applied.

2. Available information

The case analyzed in this article occurred on a parcel located in the city of Puerto La Cruz, in the northeast of Venezuela, on an industrial facility. This parcel covers approximately 20 hectares, and numerous equipment and pipe-racks structures were installed, all supported on isolated direct foundations. In the area in question, a site preparation was carried out between 2009 and 2015, after which approximately 30% of the parcel was in cut, and the remaining 70% presented a backfill conformed mostly between levels +39 mosl and +58 mosl.

In 2008, the first geotechnical research was carried out in the study area, before starting the site preparation. During this investigation, layers of cohesive and granular materials of variable thickness between 3 m and 10 m, of high rigidity ($N_{SPT} \geq 30$ blows/ft) were detected, located on a stratum of calcareous, decomposed, fractured and weathered shales and siltstones. The existence of backfill material was not reported. Groundwater level was not detected.

On the other hand, in 2015 a second geotechnical study was developed in the parcel, in which the boreholes were located in specific sites where the most important equipment and structures would be installed. According to this study, developed on the modified topography up to the project level, in the explored points the presence of a backfill material was detected, constituted mainly by materials A-2-6, A-4 and A-6 (according to the AASTHO Classification System), with variable thicknesses between 5 m and 15 m. In general, in the explored points in 2015, the backfill presented N_{SPT} values between 21 blows/ft and 62 blows/ft, with natural moisture contents varying between 9% and 28%. The internal friction angle of these backfill materials ranged between 32° and 36° between 2 m and 6.5 m of depth. No significantly weak strata were detected in the surveys conducted. Groundwater level was not observed.

According to the geotechnical study carried out in 2015, it is generally observed that the material used as a backfill does not comply with what is specified in section 2.1 of the Standard PDVSA AK-11 *Earthwork - Excavation & Backfill*, which is mandatory for oil industrial projects in Venezuela. This standard specifies that the backfill materials must be A-1, A-2 or A-3, i.e. draining granular materials. However, due to the limited availability of such materials in areas close to the project, those responsible for it agreed to make an exception to the aforementioned regulations, allowing the backlfill to be made with materials A-2-6, A-4 and A-6, coming from the hills near the project.

The soil survey performed in 2015 confirmed, based on field and laboratory data, that the backfill material had adequate support capacity for foundations, as long as the conditions of the subsoil studied were maintained. The final report included a special recommendation regarding the prevention of rainwater infiltration during foundation construction, given the clayey nature of the materials used as backfill.

3. Fault description

At the end of 2016, settlements up to 180 mm were registered in Unit 66B, in some sectors of a pipe rack supported on isolated direct foundations, with an embedment depth of 3 m below the current ground level. Later, in other units, several structures showed settlements of similar magnitudes to those registered in Unit 66B. In all the mentioned cases, the structures were in the construction stage, so the soil deformations did not occur under service loads.

In Figure 1, measurements corresponding to the topographic control of the project are observed. These measurements correspond to the extreme settlements (minimum and maximum) in the most affected units. Also, photographs illustrating the effects of registered settlements are shown.

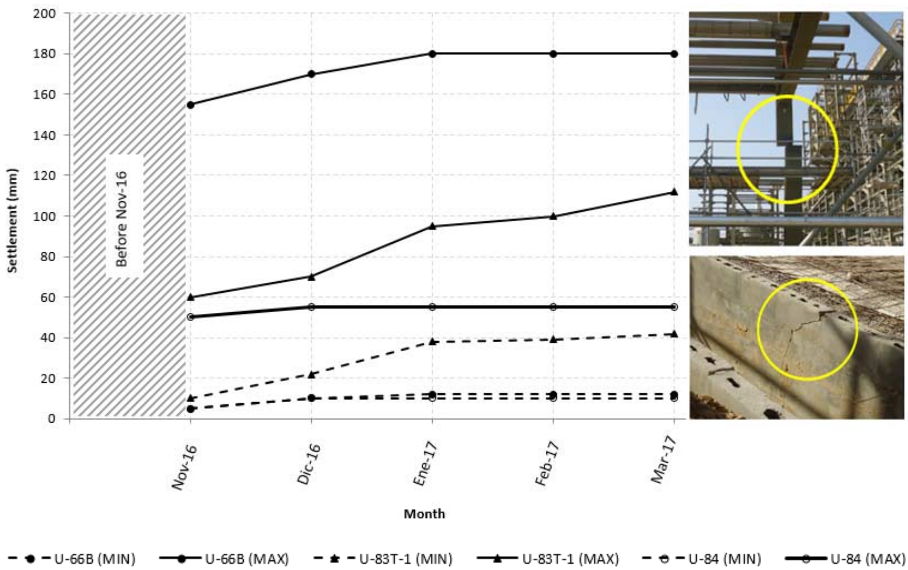

Figure 1. Settlement measurements and field deformations evidences.

Topographic deformation measurements before November 2016 (when the settlements of the structures reached, in some cases, more than 40 mm) were not reported, which suggests that a progressive topographical control of the foundations' coordinates was not carried out during the construction stage. Also, based on the values shown in Figure 1, it is evident the occurrence of variable differential settlements between 45 mm and 168 mm, inadmissible for the structures installed in the project.

Finally, it should be noted that foundations of all the structures that suffered deformations were isolated direct foundations, combined foundations or slab foundations (that is, shallow foundations), which confirmed the existence of an important compressibility problem in the subsoil layers affected by the stresses due to the application of external loads, in some sectors of the parcel.

4. Analysis of failure causes

4.1. Hypotheses to analyze the causes

Based on the evidence observed in the field and the analysis of available geotechnical and environmental information, the settlements observed in the parcel could be associated to two (2) main causes:

1. *Constructive causes*. Existence of a localized area of the parcel, in which the backfill material was poorly compacted below the embedment level of the foundations (3 m depth, measured from the current ground level).
2. *Environmental causes*. Weakening of the backfill due to the rainwater infiltration, due to the occurrence of heavy rains in the area during construction works in 2016.

In order to analyze these causes, two hypotheses were considered:

A. In the study area there is an adequately compacted backfill, constituted by predominantly clayey materials, particularly sensitive to changes in moisture content, which was dampened by the infiltration of rainwater. Due to this, the backfill softened, generating settlements of some structures.

B. In the study area there is a backfill made up of predominantly clayey materials, particularly sensitive to changes in moisture content, conformed under the application of a compaction energy lower than that required to reach 95% of the MDD determined according to the ASTM standard D-1557 (modified Proctor). Due to this, the backfill underwent excessive deformations, generating settlements of some structures.

The validation process of these hypotheses included: I) the completion of a detailed geotechnical exploration in the sites where the largest settlements of structures were observed; II) the analysis of the backfill compaction level; III) the analysis of the possible softening of the backfill due to the rainwater infiltration.

4.2. Geotechnical investigation in 2017

In this new investigation, 15 boreholes were performed, identified as PBH-01 to PBH-15, and 5 test pits, identified as C-1 to C-5. These surveys were carried out in the sectors where the greatest settlements of the installed structures were observed, at distances greater than 50 m from the boreholes realized in 2015; and they focused mainly on the compressible stratum characterization, below the foundation level.

In this exploration, the presence of a backfill formed with materials A-2-4, A-2-6, A-4 and A-6 according to the AASHTO Classification System, located between the surface and up to 11.5 m depth, was observed. Below the backfill material and up to the maximum exploration depth, similar materials as those found in the 2008 and 2015 studies were observed.

The N_{SPT} values recorded and corrected by energy between 3 m and 8.5 m of depth, are lower than 15 blows/ft, and between 8.5 m and 11.5 m of depth, they exceed on average 30 blows/ft. In the underlying natural ground, the resistance to penetration increases gradually with depth. The moisture content of the recovered samples varied between 5% and 35%, with high values from 20% to 33% between 3 m and 8.5 m of depth.

Figure 2 shows the location of the low soil rigidity are in the studied parcel, considering the explorations made in 2008, 2015 and 2017.

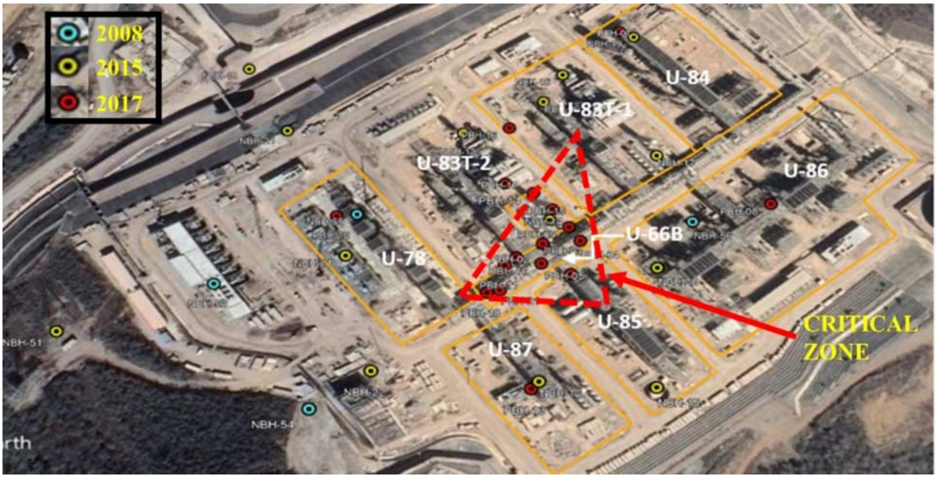

Figure 2. Critical zone of the parcel, considering explorations performed on 2008, 2015 and 2017.

According to the above, the presence of a layer of low rigidity materials between the surface and 11.5 m depth is evident, in which a critical sector of very low soil rigidity between 3 m and 8.5 m of depth is observed.

4.3. Compaction conditions of backfill material

According to section 3.7 of the PDVSA´s standard AK-11 *Earthwork - Excavation & Backfill*, the degree of compaction of a backfill around or under foundations must be at least 95% of the MDD of the modified Proctor test. Thus, in order to verify that this condition has been fulfilled in the studied area, *in situ* density measurements were made at depths of {3; 3.5 and 4} m in the trial pits C-1 to C-5.

Additionally, in order to analyze the compaction energy level used in the studied backfill, compaction tests were carried out in the laboratory at different levels of compaction energy, using samples recovered in trial pits C-1 to C-5. Table 1 summarizes the information associated with these tests.

On the other hand, Figure 3 shows the information obtained from laboratory compaction tests for different compaction energy levels, as well as *in situ* measured density and moisture content values. In this figure were included the average curves for each compaction energy level, as well as the curves corresponding to 20%, 40%, 60%, 80% and 100% saturation.

In Figure 3 it can be seen that, in general, *in situ* dry density values are lower than 1,600 kg/m^3, which is equivalent to compaction energy levels below 275 kN-m/m^3. The *in situ* moisture content values present a great dispersion, with a minimum of 7% and a maximum of 26%. The degree of saturation of the backfill presents a variable range between 40% and 80%, without reaching the degree of total saturation. Also, it can be observed that the void ratios determined for *in situ* conditions are considerably higher than the void ratios corresponding to compacted materials.

Table 1. Compaction tests on samples from trial pits C-1 to C-5.

Compaction test	Number of layers	Weight of hammer (lb)	Blows by layer	Energy (kN-m/m^3)
Modified Proctor	5	10	56	2700
Standard Proctor	3	5	56	600
	3		45	475
	3		23	242

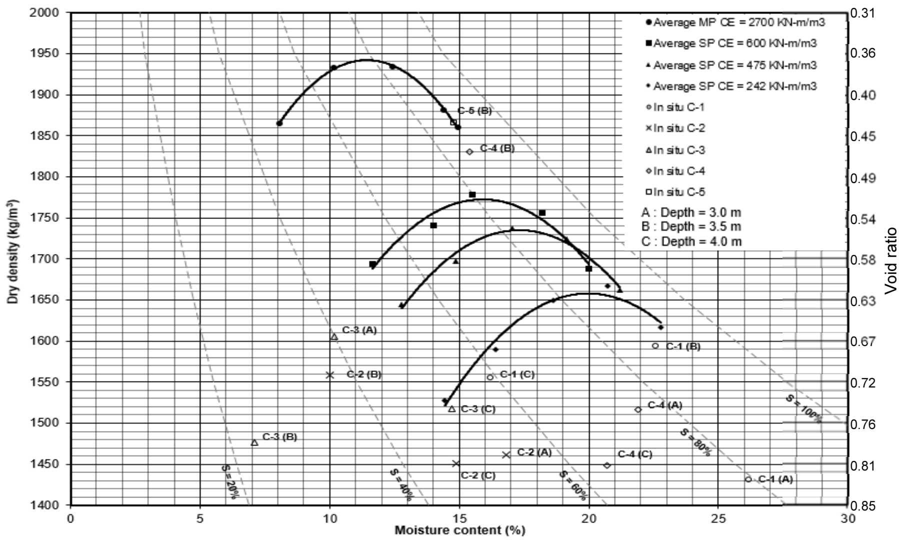

Figure 3. Compaction lab tests and *in situ* compaction measurements in trial pits C-1 to C-5.

Considering the previous analyzes, it is evident the existence of around 5 m of an uncontrolled backfill (even conformed without any type of compaction, or simply compacted by the passage of trucks) below the foundation levels (between 3 m 8.5 m deep), in those sectors of the parcel where significant settlements were observed in the installed structures. Likewise, it is clear that the degree of compaction of backfill conformed in the critical zone of the parcel, does not comply with the conditions stipulated in section 3.7 of the PDVSA AK-11 standard.

4.4. Saturation due to rainwater infiltration

The main effect of a saturation process due to rainwater infiltration into predominantly cohesive backfill, is the increase in moisture content and the degree of saturation of the material. Another important aspect to consider is that a backfill (at any level of compaction energy) has a maximum capacity to store water until saturation is reached, which depends directly on the void ratio [2].

In order to analyze the above-mentioned aspects, Figure 4 was prepared. Figure 4-a) shows the variation of moisture content (ω) as a function of the void ratio (e) for samples of compacted soils at different levels of energy, obtained from the trial pits C-1 to C-5. Figure 4-b), on the other hand, includes information on the degree of saturation (S) and the void ratio obtained from *in situ* measurements carried out in the aforementioned trial pits.

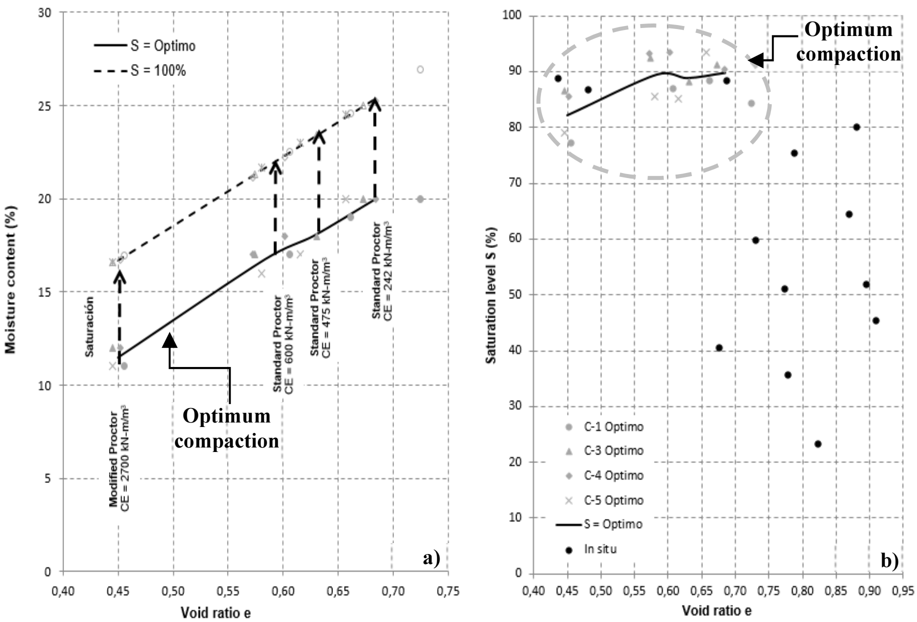

Figure 4. Moisture content vs. void ratio for different compaction energy levels (CE) and saturation (S), and *in situ* conditions according to measurements in trial pits C-1 to C-5.

According to the information shown in Figure 4, the materials compacted according to the pattern derived from a modified Proctor test, and subsequently saturated due to some external reason, would not reach moisture content values higher than 18%. For uncontrolled backfills conformed with the same material, in which the void ratio could be greater (and therefore have greater capacity to store water), moisture content values higher than 30% would be reached in case of saturation of the material.

Considering the analyzes carried out, it is clear that, in general, the effect of softening by saturation of the backfill is less significant than the low resistance of the material due to the low compaction energy. This can be explained because the *in situ* saturation levels present a great variability (between 20% and 90%), and also due to the fact that the greater settlements recorded, do not necessarily correspond to the sectors with higher saturation levels.

4.5. Hypothesis analysis

According to the analysis carried out, the validation of hypothesis B is evident. It should be noted that, even taking into account the predominantly clayey nature of the material used as backfill, the effect of the rainwater infiltration as a factor of saturation and softening, it is less important than the fact that the backfill was conformed under uncontrolled conditions. This can be explained because the levels of *in situ* saturation present a great variability (between 20% and 90%), and also due to the fact that the greater settlements registered, do not necessarily correspond to the sectors of the parcel with higher saturation levels.

5. Foundation solution

The proposed foundation solution, basically consists of installing a number of micropiles, which will cross the existing foundation, and build a cap head that serves as a connection between these deep elements and the existing foundation. In this way, the loads transmitted by the superstructure will be completely absorbed by the micropiles.

The design of the micropiles was carried out as established by the FHWA [3]. More than 400 type B micropiles were installed, 15 m deep and 200 mm in diameter.

In order to ensure the functionality of the proposed foundation solution, a load testing program on micropiles was carried out, following the guidelines established by the FHWA [3]. The program included the test of five (5) micropiles (equivalent to 1% of the total elements to be installed) located in different sectors of the affected area, of which one (1) showed failure in the micropile.

According to what is established in section 7.4.3.3 of FHWA [3], in case of failure of the micropile, the contractor must review the design of the element, the construction procedure, or both. After reviewing the design (in which no deficiencies were found), when analyzing the constructive method, it was concluded that the use of bentonite mud during the drilling prior to the placement of the grout, affected the ground-micropile adhesion. Based on the above, the contractor made significant changes in the construction process, which allowed to reach the design capacity of the micropiles.

6. Learned lessons

Based on the analysis carried out, they derived the following learned lessons:

- *Material used as backfill*. The use of materials with high clay content, susceptible to modify their conditions due to changes in soil moisture content, is not recommended for backfills in industrial facilities, since during the useful life of the facility, situations could arise that would constitute a high risk of soil wetting (excavations, broken pipes, faults in drains). It is imperative to use draining granular materials.
- *Quality control of backfill*. Any backfill formed for a civil projects, must have an adequate quality control, carried out by certified companies that ensure the quality and functionality of the backfill.
- *Load tests on micropiles*. It is necessary to carry out load tests in micropiles, since they allow to verify the admissible capacity of the micropiles and identify deficiencies in the construction process, and modify them to meet the design premises.

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