

Settlement observations of industrial structures founded on soft soils: case of the harbour zone of Bejaia city

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ABSTRACT: This paper presents and discusses the settlement observations of two industrial structures located on the harbor zone of Bejaia. It consists of a cylindrical steel tank and a battery of ten steel silos. The tanks were founded on a reconstituted and compacted granular fill. The silos were founded on a flexible raft foundation anchored on reinforced soil by stone columns. Due to the high compressibility of the soft soil deposits underlying the area, the foundations of these structures were subjected to excessive settlements, prejudicial to their stability. For the tanks, micropiles are used for underpinning and retrofit of existing foundations.

1 INTRODUCTION

Bejaia is located at about 250 km east of Algiers, Algeria. The harbor area had not experienced in the past urban development because of the different hazards identified by hydraulic and geotechnical studies conducted in the region. The subsoil of the area has unfavorable geotechnical characteristics. From the ground surface to approximately 30 to 40 m deep, it mainly consists of very soft to soft soil (Bahar et al. 2011; Sadaoui 2006). These kinds of soil are highly compressible and with lower strength. The unfavourable geological condition brings a lot of challenges to the heavy constructions. In this area, many foundation types, ranging from shallow foundations on the untreated or treated soils by various ground improvement methods to deep foundations, have been used. However, the shallow foundations usually suffer from the excessive settlements (storage tanks, silos, abutment of bridges), requiring additional reinforcement by a very expensive underpinning work.

This paper presents and discusses the settlement observations of two industrial structures located on the harbor zone of Bejaia city. It consists of a cylindrical floating roof steel tanks and a battery of ten steel silos. The tanks were founded on a reconstituted and compacted granular fill. The silos were founded on a large flexible raft foundation anchored on reinforced soil by stone columns.

2 GEOLOGICAL AND GEOTECHNIC CONTEXTS OF THE ALLUVIAL PLAIN

Bejaia is clinging to the slopes of Gouraya mountain, then spread southward across the alluvial plain. This plain covers an area of approximately 750 hectares. The geology materializes the plain of Bejaia in the synclinal post-nape basins of the Tell (Roth 1950). The depression between the mountains of Gouraya, to the north, and Sidi Boudraham to the southwest, has been filled by fine alluvium of the Soummam and the Seghir rivers and interpenetrated in transgressive marine deposits. It consists of sedimentary soil deposits of quaternary age. The geologic formations found in the

region are: the old alluvia represented by marl gravel, pebble and sand enveloped in silt matrix; the swamp alluvia consisted of fine elements represented by silt and mud with of fine sand intercalations; the recent alluvia which are deposits slightly muddy and cover the most of the plain and fill composed of heterogeneous soil represented by gravelly clay with a presence of few blocks. The geological history indicates that the harbor area extending the alluvial plain is composed of more or less muddy fine materials (silt, clay) and sand deposited on a bedrock encountered at approximately 30 to 40 m depth, likely marl and limestone of cretaceous age.

Many geotechnical surveys are carried out in the region to evaluate the resistance of soils and their degree of constructability. It appears that the surface layers of alluvial nature, predominantly sandy clayey and sometimes heterogeneous have not yet reached a sufficient degree of consolidation, therefore their bearing capacity is low and their compressibility is high. These soil conditions require deep foundations or soil improvement for heavy civil engineering structures.

3 CASE STUDIES

3.1 Settlements of steel tanks and underpinning work

The marine terminal of Bejaia is a zone of storage of hydrocarbon liquids. It consists of sixteen cylindrical floating roof steel tanks (Figure 1). The tanks range in capacity from 30 000 to 50 000 m³ with varying diameter ranging from 56 m to 67 m. All the tanks had a height of 16 m. The tanks were built in 1957. Their structure consists of an assemblage of metallic shells of varying thickness from 8 to 32 mm welded to a flexible foundation made of metallic sheets of 12 mm thick. Inside them slides a steel floating roof weighting approximately 430 tons (Sonatrach 1991). The tanks were founded on a reconstituted and compacted granular fill, raised from 2 to 3 m above the natural ground level (Figure 2). The main operating load for structures is the internal pressure of the stored petroleum product. The operating loads are cyclical. For the serviceability limit state (SLS), when the tank is filled, it transmits to the floor an average stress of approximately 120 kPa.

At the end of 1980s, after about 25 years of satisfactory service, three tanks C9, R13 and R21 were subjected to settlements, ovalization and tilting. The settlements observed along the perimeter of the three tanks R13, R21, and C9 are shown in Figure 3. Figure 4 shows the ovalization of the tank C9. The measured differential settlements reached maximum values of 28 cm, 22 cm and 18 cm for the tanks C9, R13 and R21 respectively (Sonatrach 2004). Because of distortion of the steel tank walls and jamming of the floating roof, a shear failure was evident and the tanks were considered unsafe for service. A comprehensive geotechnical investigation was conducted to evaluate the subsurface conditions of the site and to provide recommendations for foundation repair or retrofit of existing tanks as well as foundation design for new tanks and related facilities.



Figure 1. Tank locations.

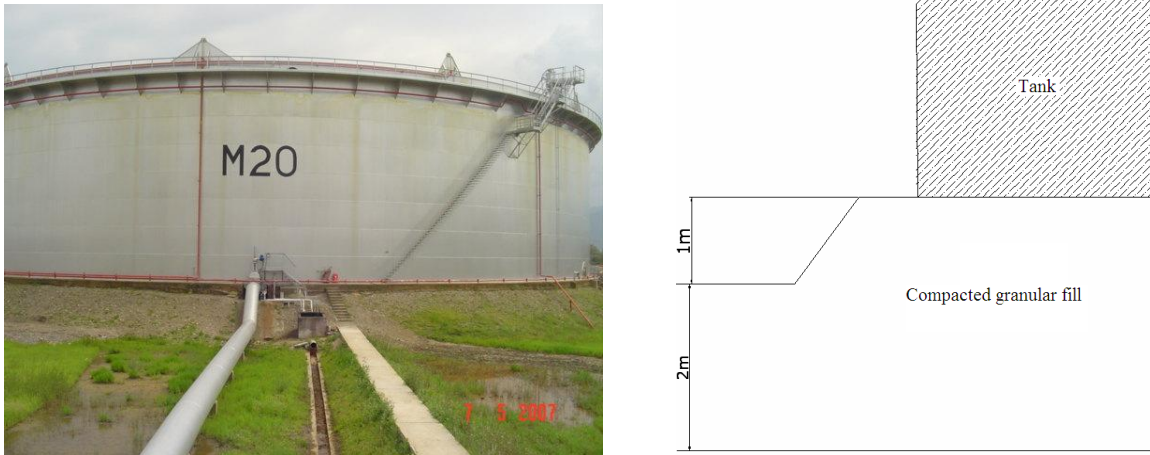


Figure 2. Oil storage tank.

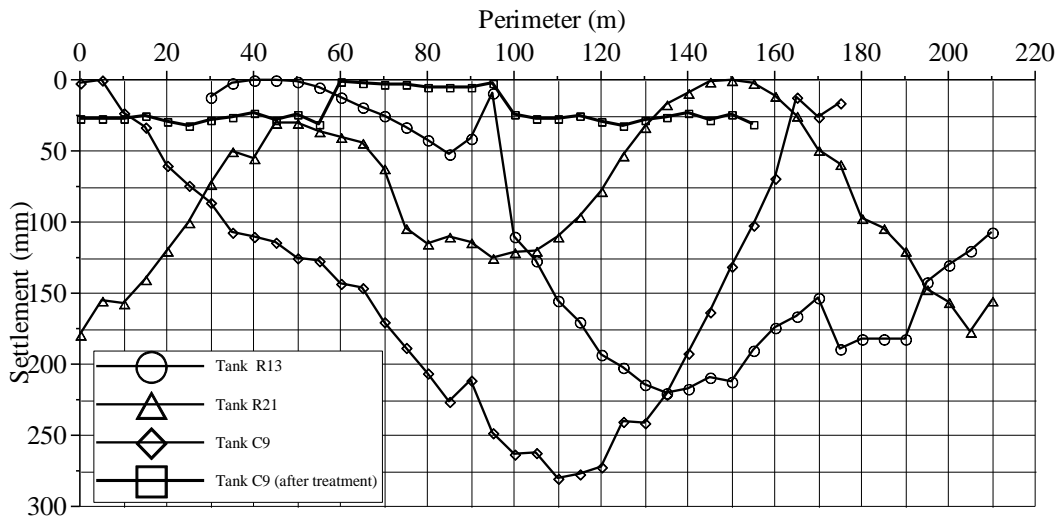


Figure 3. Settlements along the perimeter of the tanks R13, R21, and C9.

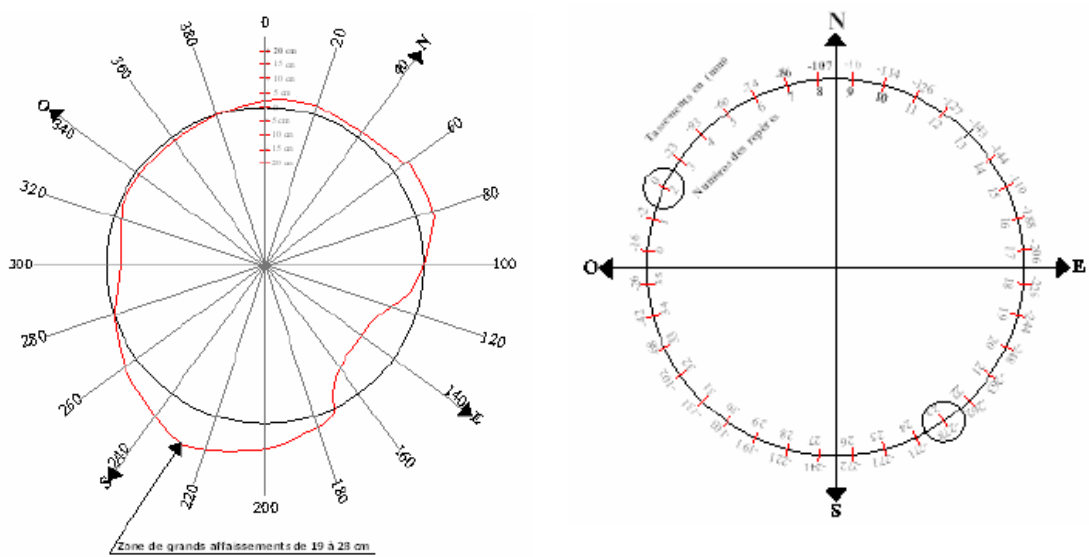


Figure 4. Ovalisation of the tank C9.

This investigation reveals that the subsoil conditions below the tank locations typically consists of 1.5 m fill overlying 24 to 28 m thick alluvial clay-sand dominated layers impregnated by mud at the northern marine terminal to sandy and gravelly with intercalation of layers of silty and muddy sand at the Southern Terminal, which is close to the marine environment. All these sedimentary layers are rest on a substratum of gray very stiff to hard marl found to a depth between 25 and 30 m. Groundwater is encountered at a depth of 2 m. Typical soil profile is shown in Figure 5. The engineering properties of the soil layers are summarized in Figure 6. This investigation indicates a low consolidation and a high compressibility of the soil layers, and shows a lateral heterogeneity of alluvial layers below the tank locations (Figure 5). This heterogeneity and the compressibility of the soft soil layers underlying the site seem to be responsible for the large differential settlements and tilting experienced by these tanks.

Based on the geotechnical investigation results, it was concluded that the subsurface soils underneath each tank to be improved. Micropiling has been chosen to strengthen the soil beneath the foundation (Figure 7). For some tanks, strengthening and underpinning work at site started in 1992, and ended in 2000. The superstructure was restored by jacking it up to the required elevations.

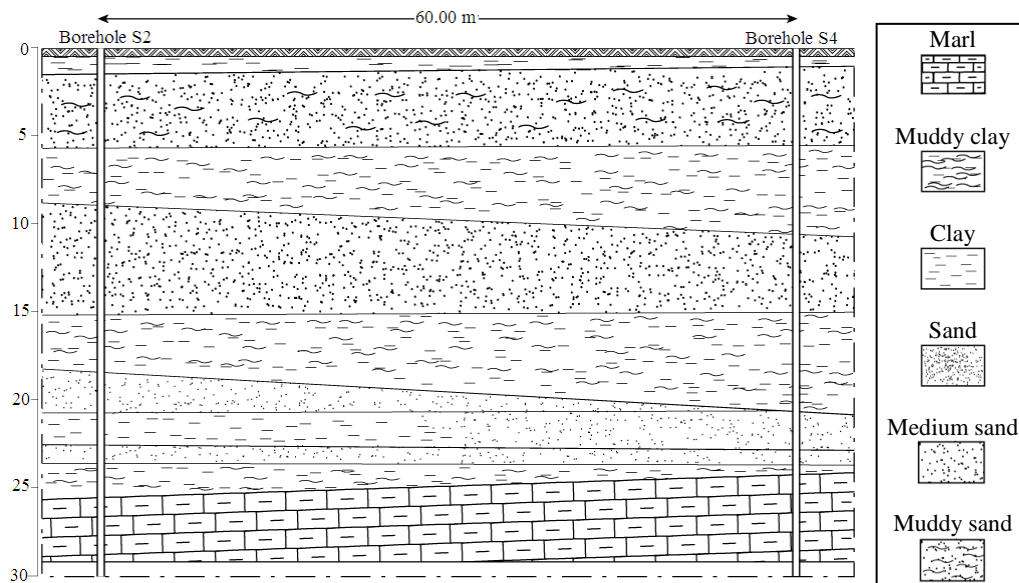


Figure 5 : Typical soil profile.

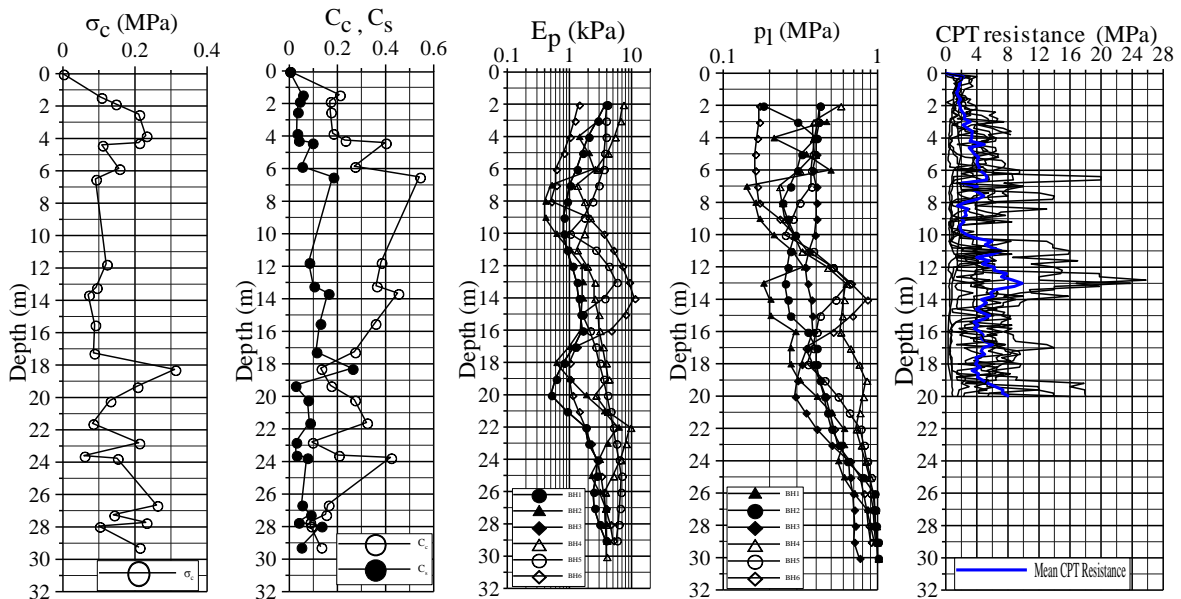


Figure 6. Soil characteristics.

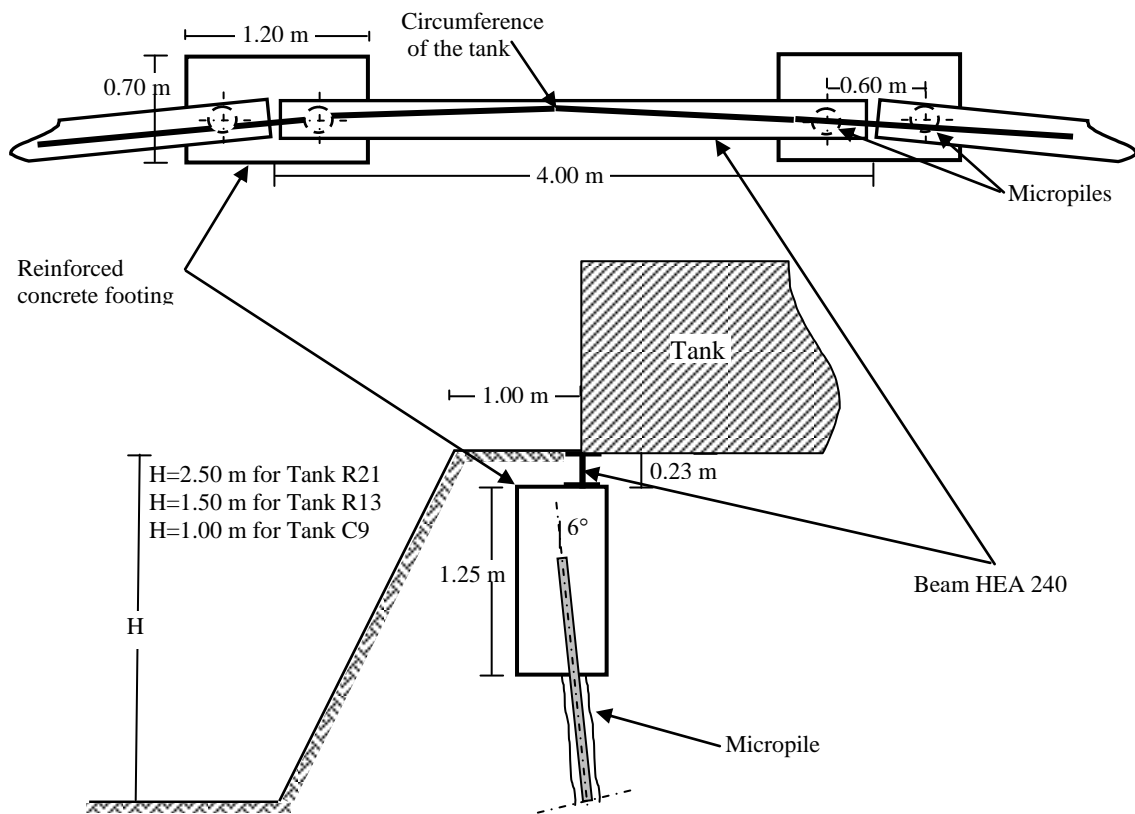


Figure 7. Strengthening and underpinning work.

3.2 Settlements of a battery of ten steel silos for grain storage

The structure consists of a battery of ten cylindrical steel silos for grain storage with a total capacity of 76000 tons (Figure 8). The height of each silo is 27 m and the diameter 23 m. The silos are based on a large flexible raft foundation of reinforced concrete of rectangular shape of 75 cm thick and 120x50 m² area. The geotechnical investigation shows a succession of sedimentary layers of silt and muddy silty sand up to 30 m depth, and then comes below a layer of plastic marl sometimes muddy (Figure 9). The level of the water table is detected from 2 m in depth. The superficial layer of site through the first twelve meters is characterized by a void ratio variable 1.06 to 1.30 and a compressibility index C_c varying between 0.37 and 0.54. These results reflect the very loose nature of the soil and high compressibility. The dynamic penetrometer tests showed a horizontal homogeneity of alluvial formation (Figure 10). The minimum value of resistance is recorded from 0.8 to 1.0 MPa on the first twelve meters. The dynamic resistance of the layer of fine dense sand is found higher than 10 MPa at 12 m depth. For a large raft foundation, the oedometer method was used to estimate the settlement. For a service stress of 120 kPa, compaction consolidation of alluvial layers of 40 m depth is estimated at 160 cm. These results are not allowable. Therefore, there is need to move into the deep foundation system or to proceed to reinforce the soil.

Based on the results of dynamic penetrometer test and the experience already gained in previous projects made at the harbor zone of Bejaia makes the contracting authority and the owner of the work to strengthen soil by stone columns (Coprec 2004). This choice is also justified by the cost of achieving in comparison to deep foundations by piles, the local availability of substitute material (ballast) and timeliness of the process. The final depth of treatment was selected on the basis of the control tests without any theoretical justification. The large raft is anchored on 12 m reinforced soil by stone columns (Figure 11). The stone columns have an average diameter of 1 m and a length of 12 m. The diameter of 1 m was verified after stripping of the column test conducted near the site. The mesh really made in the field is regular and rectangular, 2.39m x 2.43m. These dimensions represent respectively the space between the stone columns in the transverse and longitudinal

directions. The total number of columns made on the area of 5871 m² is 1100 columns and the approximate volume of rolled ballast used is estimated to 10400 m³.

For verification of the design and certification of stability and serviceability, a load test on a single stone column was performed according to standards by the company responsible for carrying out the work. The load settlement behaviour of the improved ground as observed for single column tests is presented in Figure 12. The results give a settlement of about 7 mm for 1.5 time ultimate state limit, corresponding to 478 kPa. Twenty dynamic penetrometer tests were also carried out on stone columns in order to determine their mechanical properties according to the requirements of the DTU 13-2 (1978) and COPREC (2004). Typical results are given in Figure 13. These results showed that only 25% of tests give a dynamic resistance greater than 15 MPa from 1 m depth. 75% of tests give dynamic resistance between 3 and 9 MPa from 2 to 4 m depth. The work began in 2004 and completed in early 2005. A monitoring system of settlement was placed on the silos.

Because the absence of a calculation, calculations of bearing capacity and settlements are performed by the method of Priebe (1995), taking into account the actual data of the project. For the performed strengthening, the total settlement obtained for a service stress of 130 kPa is about 58 cm, 14.60 cm of which are within the depth of the soil treatment. The calculated settlements are unacceptable for a soil treatment within 12 meters deep. Based on these pessimistic results, a monitoring of settlement was recommended before any loading. This follow up began immediately after completion of construction.



Figure 8. View of the silos.

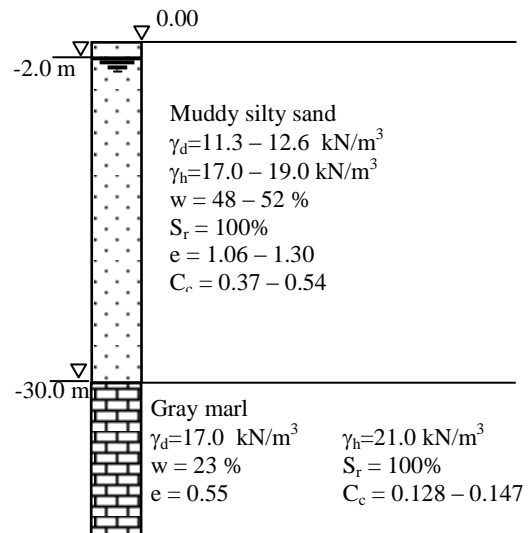


Figure 9. Geotechnical profile.

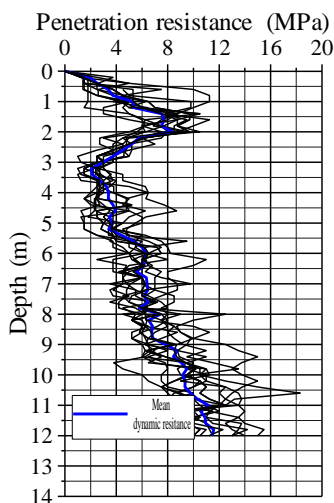


Figure 10. Dynamic penetration test results

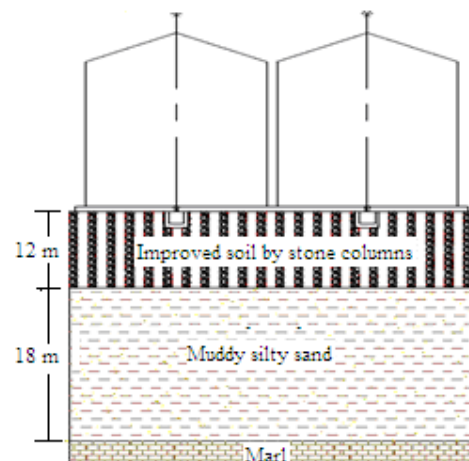


Figure 11. Schematic section.

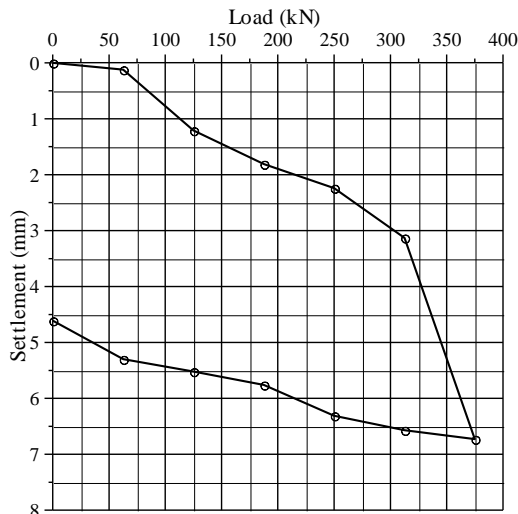


Figure 12. Results of a single stone column load test.

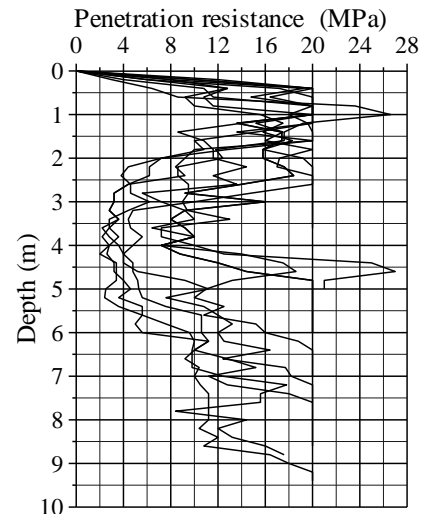


Figure 13. Typical dynamic penetration resistance results carried out on stone column

For verification of the design and the stability of the silos during the cereal storage, the structure was monitored to evaluate vertical settlements. The settlements were made by tachometric aiming on twenty fixed reflective targets (points A1, B1,..., Figure 14). Four stations were installed sufficiently far from the silos to avoid the influence of the loading of the silos and vibration devices. They were fixed on reinforced concrete caissons isolated by polystyrene to avoid possible relative movements of the stations. The reflectors were fixed on both outer edges and central part of the silos to the same initial side elevation. The measures are conducted in accordance with the general loading and unloading of the silos. The observations were made from 01/09/2005 to 01/03/2007. Two independent stations, Z_0 and Z_1 are also located respectively outside and on the center of the raft.

Figure 15 shows a typical loading and unloading of the silos in time recorded on 545 days of assessment. The distribution of foundation settlements in time for some measuring points is shown in Figure 15. Figure 16 shows the settlements against the silo load on the center of the raft. The analysis of the different results obtained has the following observations:

- During the first five months of loading and unloading of the silos, the foundation settlements developed rapidly to about 15 to 20 cm and 7 to 13 cm on the center and the edges of the raft respectively. The maximum differential settlement is 10.80 cm which is measured in the silo C23. The differential settlement occurred is prejudicial to the stability of the structure that the first signs of tilting and deformations at the base of the silos began to appear at the central area.

- The loading evolution to a value of 130 kPa and maintained constant during one month generated a total settlement of 28 cm in the central area and 17 cm at the edges. The differential settlement is about 11 to 14 cm between center and periphery of the raft. The differential settlement was accentuated by the non compliance with recommendations for loading and unloading silos.

- The indices of excessive settlements appear on site by observed deflection of the raft, the change of the slope of the cover raft toward the central part, important deflection of the road, cracks and tilting of the silos particularly the silos C13, C14, C23 and C24.

- The soil improvement has done only under the raft; it was arrested at the edge of the raft. The difference in behavior between the two areas, virgin soil (outwards of the raft) and improvement soil (under of the raft) supported the development of the differential settlements. The measured settlement at point Z_0 located at 4 m from the edge of the raft is about 7 cm.

- The maximum total settlement measured at the center of the raft is 28 cm. This result is obtained for loading varying from 24.60 to 130 kPa. The settlement tends to stabilize because it is stationary for a month of observation with a constant loading. However, regarding to floating stone columns, it is necessary to understand the consolidation settlement of compressible layer that lies between the base of the stone columns and the marl bedrock (18 to 23 m thick). The settlement obtained by the method of Priebe for a stress of 130 kPa is 58.30 cm, with 14.60 cm obtained in the treated layer. This result seems reasonable owing to the fact that the in-depth influence of the raft foundation and a settlement of untreated layer will appear in the long term.

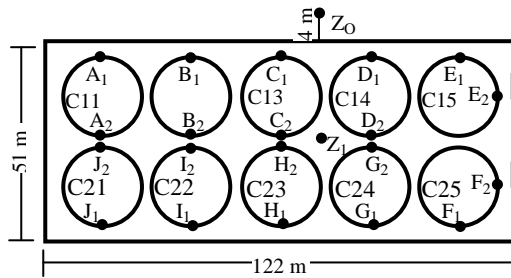


Figure 14. Layout of the silos.

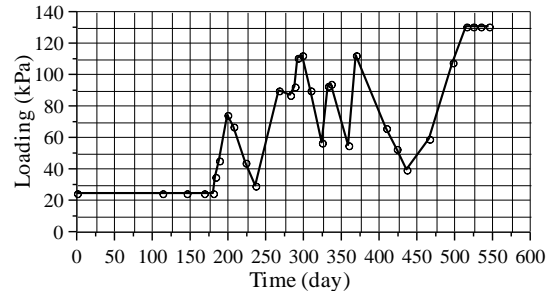


Figure 15. Loading of the silos since 01/09/2005.

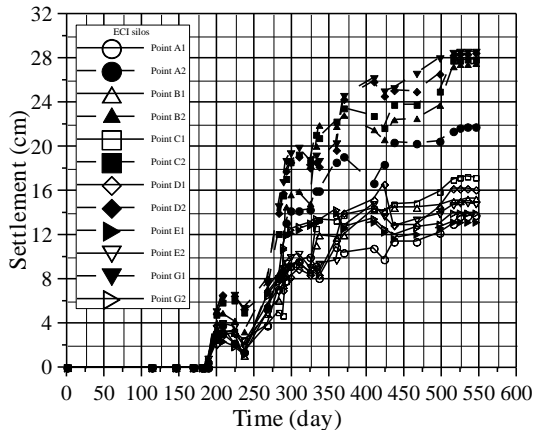


Figure 16. Settlements from 01/09/2005 to 01/03/2007

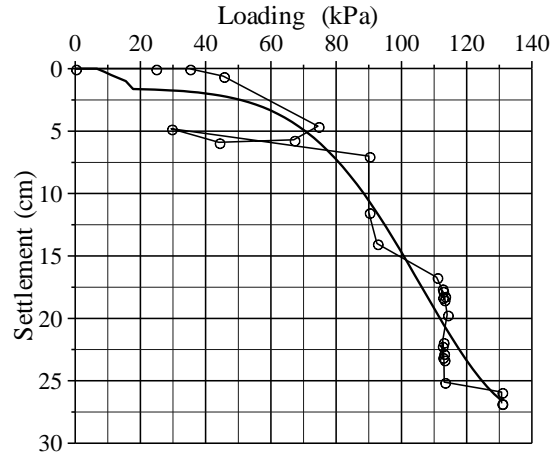


Figure 17. Settlements vs. loading at the raft center.

4 CONCLUSIONS

The shallow foundations of the industrial structures located in the harbor area of Béjaia were subjected to excessive settlements prejudicial to their stability. The construction of heavy industrial structures in this area requires deep foundations or soil improvement to reduce soil settlements. Micropiles were successfully adopted to solve the foundation problems of the tanks. The large flexible raft foundation of the silos, anchored on reinforced soil by stone columns, was also subjected to serious stability problems due to the excessive differential settlements. Cracks, tilting, deflection were observed, started from a differential settlement of about 7 cm, corresponding to about 1/400 of the silo diameter. These observations show a lack of the geotechnical survey carried out at the design stage leading to underestimate the thickness of the soil reinforcement, and to a non respect of the operating load recommendations.

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