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Differential Settlements of Cylindrical Steel Storage Tanks: Case of the Marine Terminal of Bejaia

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DIFFERENTIAL SETTLEMENTS OF CYLINDRICAL STEEL STORAGE TANKS: CASE OF THE MARINE TERMINAL OF BEJAIA

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ABSTRACT

The marine terminal of Bejaia is a zone of storage of hydrocarbon liquids. It consists of sixteen cylindrical floating roof steel tanks founded on a reconstituted and compacted granular fill. At the end of 1980s, after about 25 years of satisfactory service, the tanks were subjected to settlements, ovalization and tilting. Because of a distortion of the steel tank walls and jamming of the floating roof, a shear failure was evident and some 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. It was concluded that the soils underneath each tank to be improved. Micropiling has been chosen to strengthen the soil beneath the foundation. The proposed paper describes and discusses the case study, the method of treatment adopted in the field and the results of numerical modeling, and gives some lessons learnt.

INTRODUCTION

Béjaia is a coastal town located at about 250 kilometers east of the capital Algiers, Algeria. It is a part of the alluvial plain which covers an area of approximately 750 hectares. This 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 low bearing capacity of the soil, its high compressibility and the risk of liquefaction and flooding are among these risks and are a constraint on urbanization and require reasonable accommodation to limit the damage. Geotechnical surveys, carried out in the region to evaluate the resistance of soils and their degree of constructability, indicate that the surface layers of alluvial nature, predominantly sandy clayey and sometimes heterogeneous have not yet reached a sufficient degree of consolidation. These soil conditions require deep foundations or soil improvement for heavy civil engineering structures. Several cases of structures founded on shallow foundations in this alluvial plain have suffered various pathologies: collapse of oil tanks, loss of verticality of silos, settlement of Abutment Bridge (Sadaoui, 2006; Bahar et al., 2010; Bahar et al., 2011).

The marine terminal of Bejaia, located in the harbor area, is a zone of storage of hydrocarbon liquids. It consists of sixteen cylindrical floating roof steel tanks. The geological history indicates that this 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 40 to 50 m depth, likely marl – limestone of cretaceous age. At the end of 1980s, after about 25 years of satisfactory service, the tanks were subjected to settlements, ovalization and tilting. No site investigation was carried out prior to construction of the tank in order to obtain the soils information necessary for the design of the foundation. Instead, the initial design was based on the known performance of other structures in the area. 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 (Sonatrach, 1991; Sonatrach, 2004). The proposed paper describes and discusses the case study, the method of treatment adopted in the field for some tanks and the results of a numerical analysis performed to predict settlements of tanks, and give some lessons learnt.

SITE AND GEOLOGY

Bejaia is clinging to the slopes of Gouraya mountain, then spread southward across the alluvial plain. The regional geology materializes the plain of Bejaia in the synclinal postnape 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 Seghir rivers and interpenetrated in transgressive marine deposits (Fig. 1). It consists of sedimentary soil deposits of Quaternary age. The geologic formations found in the region are:

- Old alluvium: they are represented by marl gravel, pebble and sand enveloped in silt matrix.

- Swamp alluvium: they consist of fine elements represented by silt and mud with intercalations of fine sand.

- Recent Alluvium: The deposits are slightly muddy and cover the most of the plain.

- Fill: These embankments are not compact, except in the first meter, they are composed of heterogeneous soil represented by gravelly clay with a presence of few blocks.

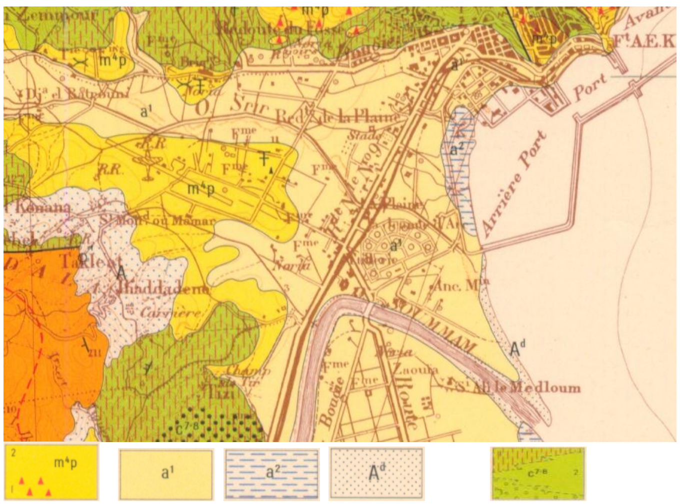


Fig. 1. Extracted of geological map of Bejaia n° 26 (1/50000).

MARINE TERMINAL SITE

Generally, foundation should be designed to provide an economical means of transmitting loadings from structures to the underlying soil stratum without causing soil failure or excessive settlement. Storage steel tanks are relatively flexible structures and they can tolerate greater settlements than other engineering structures. However, there is a limit to the settlements expected to be taken without distress. The most important undesirable effects of settlements to avoid in designing tank foundations are overall settlement of the tank, differential settlement across the diameter, which may overstress internal piping connections, differential settlement along the periphery, which may overstress the superstructure, and differential settlement between the tank and the external connection pipework. For economical design, flexible foundation commonly adopted in tank design consists of a granular overburden layer or a compacted soil pad, or a combination of both. Tank load is spread through granular overburden layer to the underlying soil. For the deposit of weak soils consisting of loose sand and marine clay, instability and settlement of pad foundation are of major concern.

The marine terminal of Bejaia is a zone of storage of hydrocarbon liquids. It consists of sixteen cylindrical floating roof steel tanks (Fig. 2). 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 (Fig. 3). 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 (Fig. 4). The main operating load for structures is the internal pressure of the stored petroleum product whose average density is 0.9. This pressure may change in the process of operation. 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.

Fig. 2. View of the marine terminal of Bejaia.

Fig. 3. Oil storage tank.

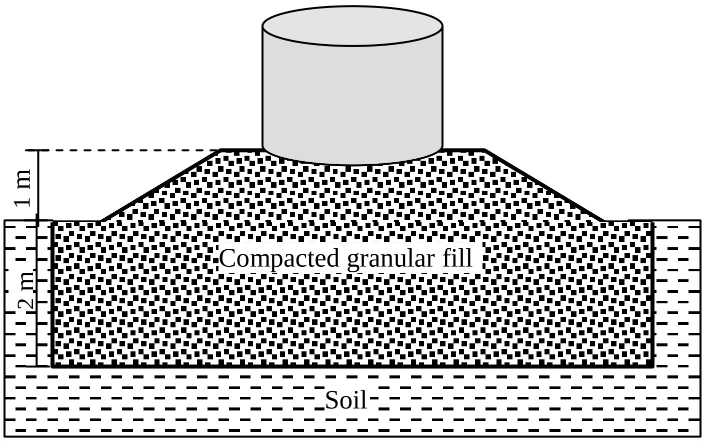


Fig. 4. Schematic tank foundation system.

SETTLEMENTS IN STEEL TANKS

Many studies on oil storage tank foundation systems show that stability and settlement are two main factors which may lead to the rupture or even the complete failure of oil tanks (Bell and Iwakiri, 1980; Green and Height, 1975; Marr et al., 1982; D'Orazio and Duncan, 1987). In comparison with the absolute magnitude of maximum settlement, differential settlement and the shape of the settlement dish are of more importance in engineering. Based upon 31 case histories of tank settlement and damage, D'Orazio and Duncan (1987) concluded that allowable bottom settlement of steel tanks depends on the shape of the deformation. They classified the shape of settlement into 3 profiles (Fig. 5). The maximum settlement is located at the center of the tank (profile A), the settlement is relatively flat at interior and decreases rapidly toward the tank edge (profile B), and the maximum settlement is located about two third of the radius from the center of the tank (profile C). The settlement profile A is the least severe with respect to distortion and profile C is the most severe.

At the end of 1980s, after about 25 years of satisfactory service, the tanks of marine terminal Béjaia were subjected to settlements, ovalization and tilting. Figures 5a and 5b show the ovalisation of some tanks with diameter of 66.91 m and 56.16 m respectively measured in December, 1994. The settlements observed along the perimeter of the three tanks R13, R21, and C9 are shown in Figures 6, 7 and 8. 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).

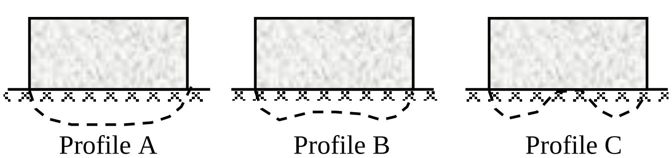


Fig. 5. Differential settlement profiles of bottom plate of steel tank.

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. No site investigation was available in order to obtain the soils information necessary for the design of the foundation. We think that the initial design was based on the known performance of other structures in the area.

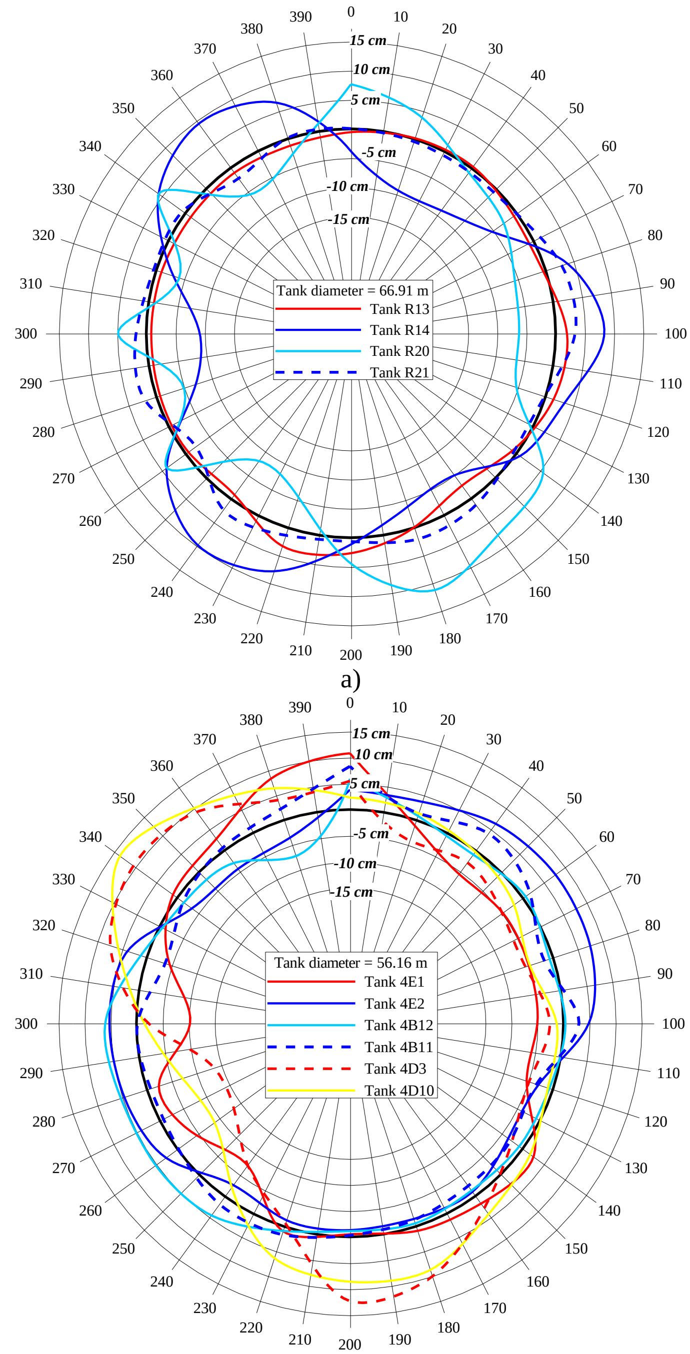


Fig. 6. Ovalisation of tanks (december 1994).

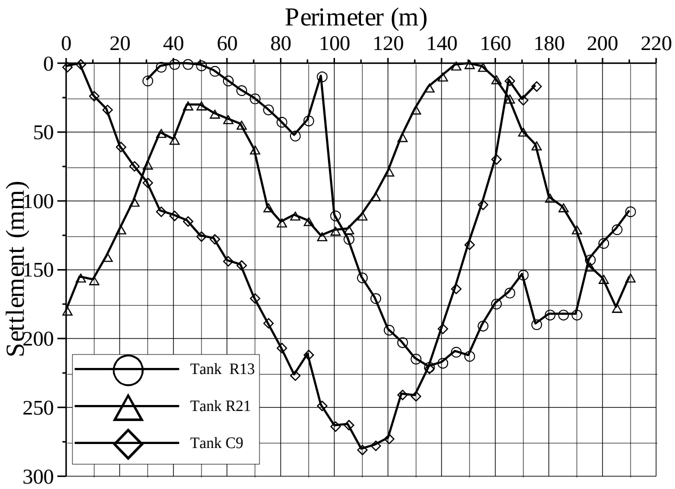


Fig. 6. Differential settlements observed along the perimeter of the tanks R21, R13 and C9.

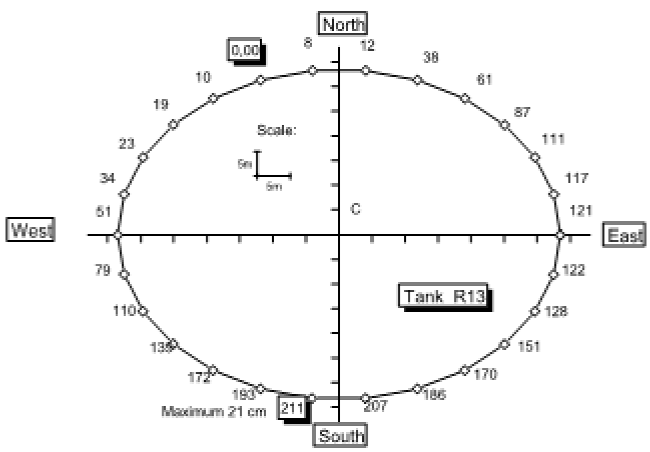


Fig. 7. Differential settlements observed along the perimeter of the tank C13.

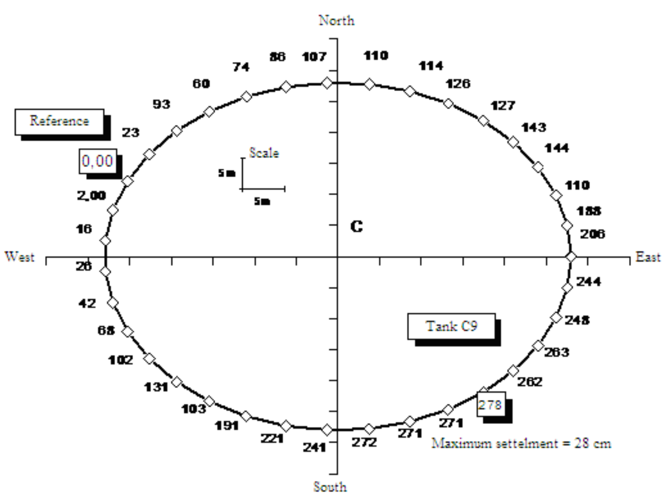


Fig. 8. Differential settlements observed along the perimeter of the tank C9.

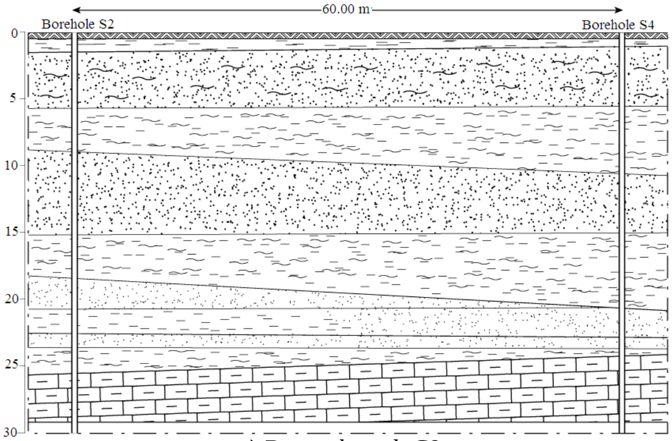
SITE INVESTIGATION

This study is focused on storage tanks A8, C9, R13 and R21. No site investigation was available in order to obtain the soils information necessary to provide recommendations for foundation repair or retrofit of the existing tanks. Geotechnical investigations were conducted beneath each tank. The soil investigation consisted of two boreholes put down through the marl stratum , three cone static penetrometer (CPT), and three Menard prebored pressuremeter (MPT).

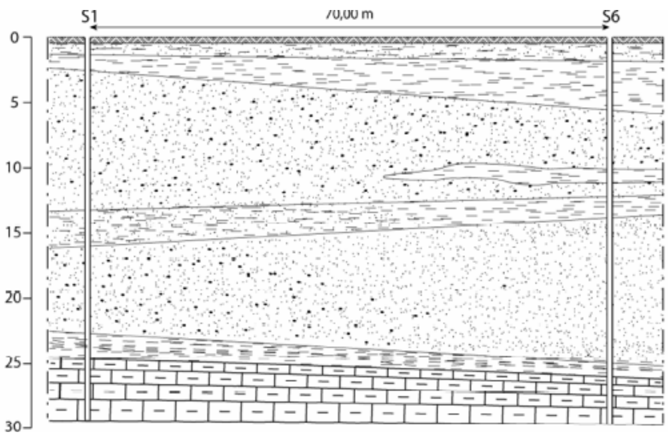
Stratigraphy over the sites of the fourth tanks was typically composed of fill about 1.5 m thick, overlying 24 to 28 m thick alluvium clay-sand dominated layers impregnated by muds at the northern marine terminal to sandy and gravelly with intercalation of layers of silty and mudy sand at the Southern 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. At the time of the geotechnical investigation, groundwater was encountered at a depth of about 2 m. The ground water level is tidally influenced and at certain times of the year, the groundwater level was just below the ground surface. Typical soil profiles are shown in Figures 9a and 9b. The engineering properties of the soil layers are summarized in table 1 and Figures 10, 11 and 12.

Grain size distributions (Fig. 10) show that 93 to 100% of the elements have a diameter lower than 0.2 mm and 77 to 100% elements have a diameter lower than 80µm (fine sand, silt and clay). The water levels are high; they vary between 21 and 48%. The wet and dry weight volume (γ_d and γ_h) are respectively variables from 11.8 to 17 kN/m^3 and 17.5 to 20.8 respectively. The degree of saturation generally varies from 95 to 100%. The clay layers have high plasticity, the plasticity index varies between 27 and 42%, and the liquid limit varies between 53 and 88%. This investigation indicates a low soil consolidation and a high compressibility of the soil layers, the index compression varies between 0.10 to 0.55 (Fig. 12). The results of direct shear and triaxial tests (UU) show dispersion due to the heterogeneity of soils. The friction angles, φ, and cohesions, c, obtained by shear tests are respectively variable from 3 to 25 ° and 10 to 100 kPa. Cohesions are low up to 25 m depth, corresponding to the alluvial and marine deposit and then an increase in the layer of gray marl. From triaxial tests, these values are generally low, ranging between 15 and 75 kPa for the cohesion and 0 to 5° for the friction angle. The tangent modulus E_i and the secant modulus E_{50}, corresponding to a level of 50% of the deviator of rupture most commonly used in geotechnical behavior laws, obtained from triaxial tests are given in table 1.

It shows also a lateral heterogeneity of alluvium layers below the tank locations (Fig. 9). This heterogeneity and the compression of soft soil layers underlying the site seem to be responsible for the large differential settlements and tilting experienced by these tanks. The tanks were founded on difficult soil conditions.



a) Beneath tank C9



b) Beneath tank R21.

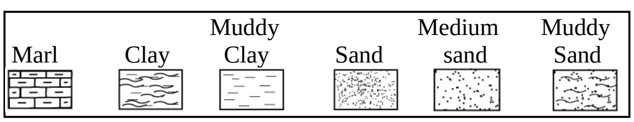


Fig. 9. Typical soil profiles.

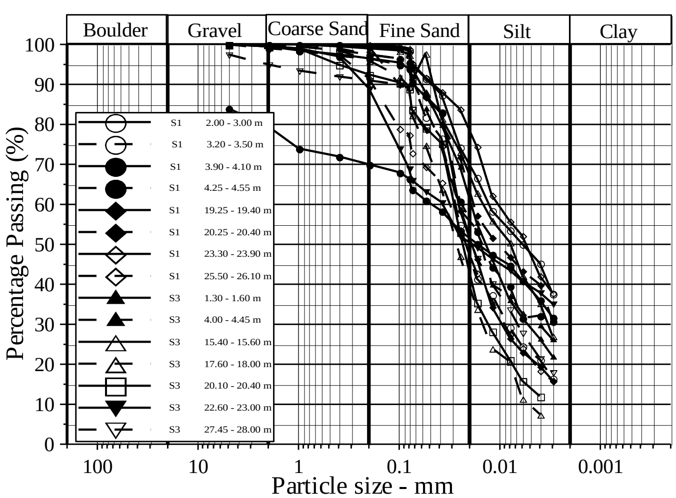
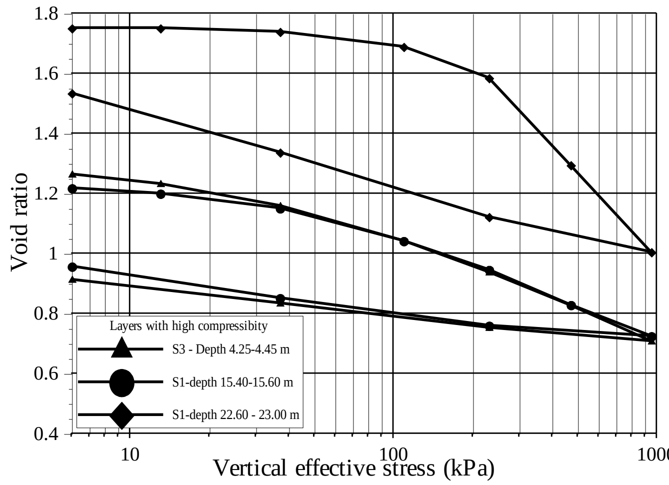
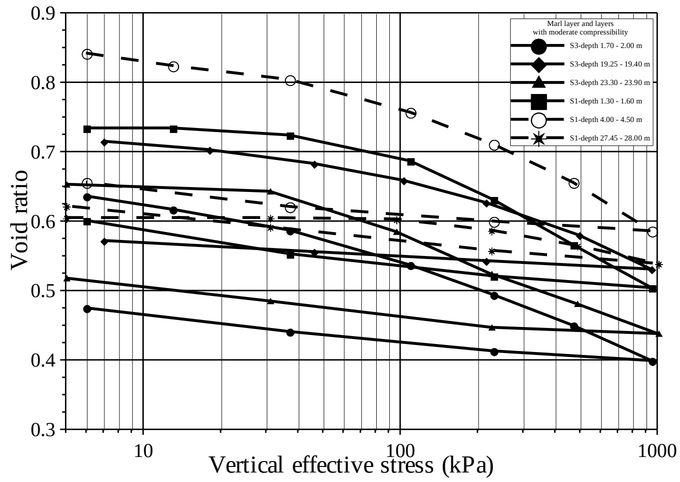


Fig. 10. Size grain distribution.



a) High compressibility soils.



b) Moderate compressibility soils.

Fig. 11. Consolidation test results.

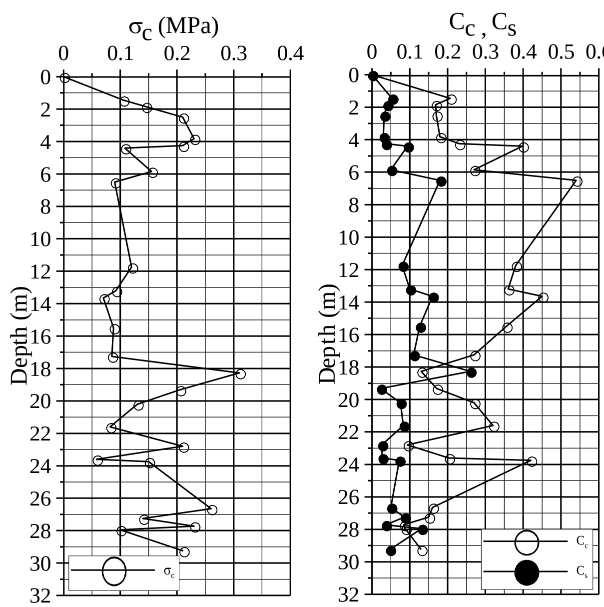


Fig. 12. Soil characteristics.

The results of the static penetrometer tests (CPT) show a large variation in tip resistance q_c. Peak strength of 10 to 30 MPa were measured in layers of coarse alluvium (sand and gravel), and between 1 to 2 MPa in layers of silty sands and soft clays (Fig. 13). According to the pressuremeter tests, the limit pressure (p_l) and pressuremeter modulus (E_p) varies between 0.15 and 0.8 MPa and 0.5 to 10 MPa in the first twenty meters (Fig.13). The relative density (Dr) of the layers obtained by different means is given in table 2.

Table 1. Soil characteristics.

Soil	c (kPa)	φ (°)	E_i (MPa)	E_{50} (MPa)
Plastic brown clay	62	0	19.5	10
Soft brown clay	15	4	4.9	2.6
Silty muddy clay	40	3	25	12.8
Plastic Gray clay	50-65	2	20-30	10-15
Highly plastic clay	27.50	0	5.8	3.2
Hard marl	75	3	53	26.8

Table 2. In situ relative density.

Layer depth (m)	In situ relative density Dr (%)					
	Electrical method		SPT		CPT	
	Mean	Low	Mean	Low	Mean	Low
0 – 8 to 11	> 60	-	50 to 55	42	42	30
8 to 11 – 19	44 to 65	32	55 to 60	45	35	25

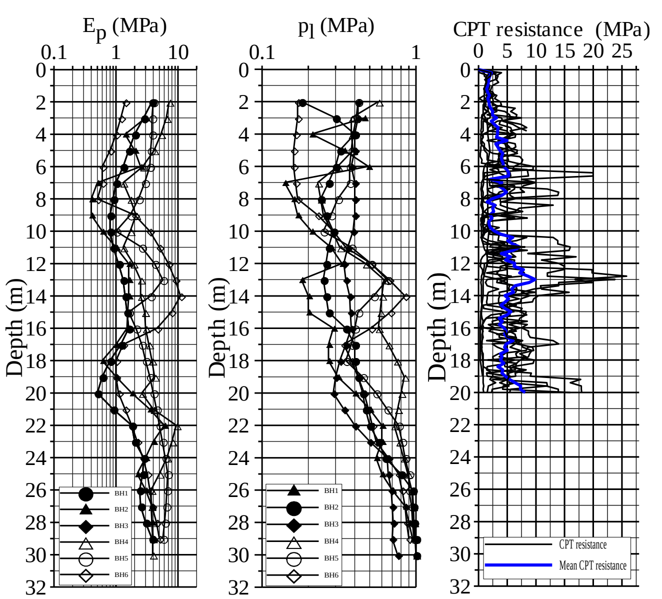


Fig. 13. In situ soil characteristics.

The liquefaction potential was also evaluated using the procedures proposed by Seed and Idriss (1971) and Youd and Idriss (2001) which utilize Standard penetration tests (SPT), and using CPT tests. The measured count all tanks founded on silty sand strata were found to be susceptible to liquefaction under an earthquake magnitude of 6.2 and surface ground acceleration of 0.20 g.

UNDERPINNING WORK

Based on the geotechnical investigation results and since the differential settlements are not tolerable, it was concluded that the subsurface soils underneath each tank to be improved. In the first step the underpinning was restricted to the three tanks C9, C13 and A8. Various techniques were considered. In situations where the tank has settled severely on soils that have very low shear and bearing capacity, it may generally be necessary to install added support by pressing piles or micropiles around the perimeter of the tank. For cost efficiency, reduced disturbance, supposed minor risk during installation injected micropiles were finally selected to strengthen the soil beneath the foundation around the circumference of the tank and to transfer the base loads of 120 kPa to denser strata, thus controlling settlements and improving the compressibility characteristics of the soil.

The foundation scheme proposed by the engineers is shown in figure 14. It was decided to support the tanks by micropiles. The micropiles have a length of 12 m and a diameter of 140 mm. They were designed to ensure working loads of 179 kN. A total of 104 micropiles were installed through the crown, with an inclination of 6°. They were arranged through the perimeter of tank into a couple of micropiles capped by massive reinforced concrete ribs, spacing of 4 m. To connect the couple of micropiles to the tank a steel beam (HEA 240), supported by the cap of the couple of micropiles was placed around the periphery. The micropiles were cast in place with a diameter between 135 and 140 mm. The boreholes were made by drilling method. As reinforcement, tube of 89 mm diameter and 11.50 m long, was used. The primary injection of the micropile is effected through the top of the pile, whereas the secondary injection is made through injection pipes at different horizon along the depth of the pile.

Then, the tanks were restored by jacking it up to the required elevations to correct the additional settlement. R21 and R13 tanks of 50000m^3 capacity were strengthened by this process between 1991 and 1992. The strengthening and underpinning works that has been carried out were considered successful. For the 10 years following completion, no visual instability was perceptible on these structures, level survey indicates an additional settlement less than 32 mm, as shown in figures 15 and 16. Foundation designs of new structures have used micropiles as an economical alternative to other foundation systems. The contractors have designed foundations for several new structures using micropiles.

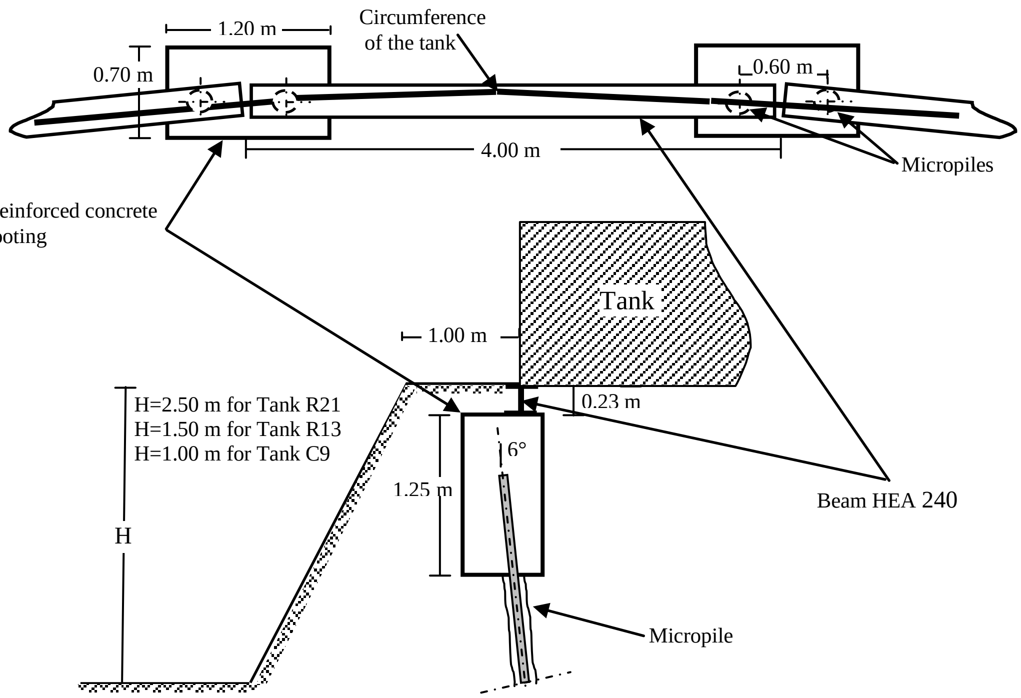


Fig. 14. Strengthening and underpinning work.

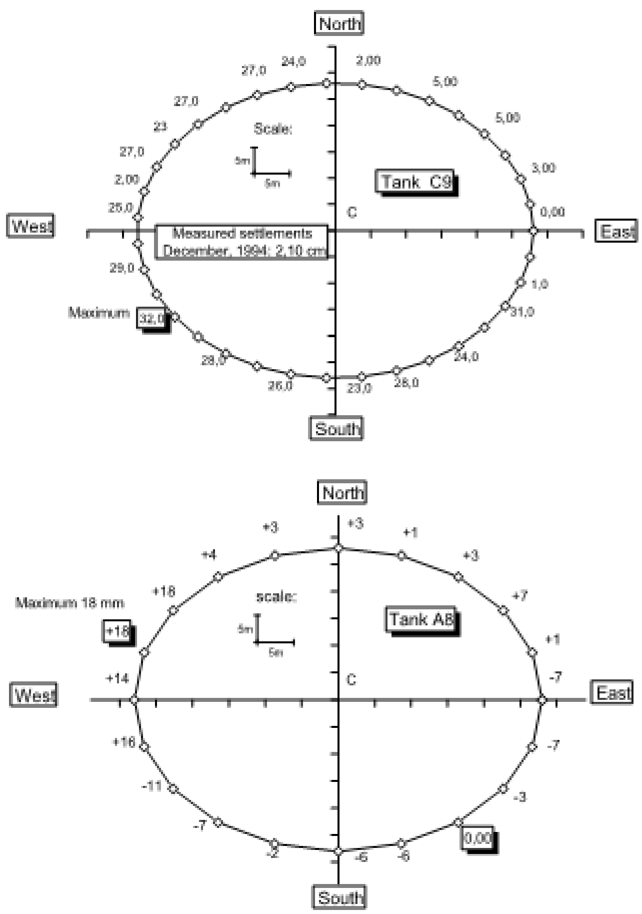


Fig. 15. Differential settlements observed along the perimeter of the tanks C9 and A8 after treatment.

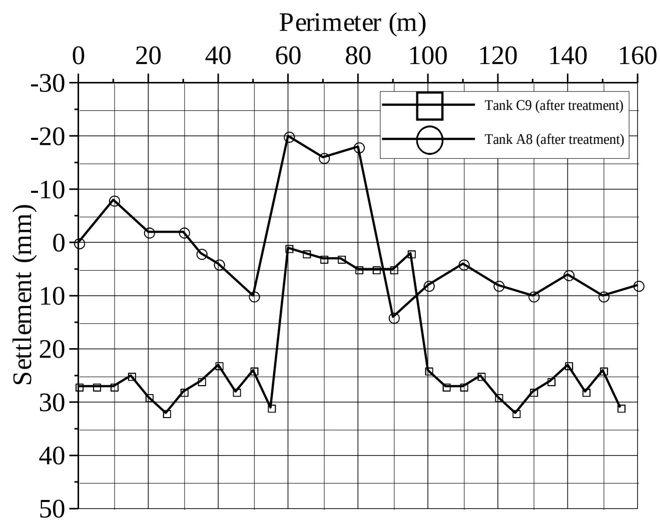
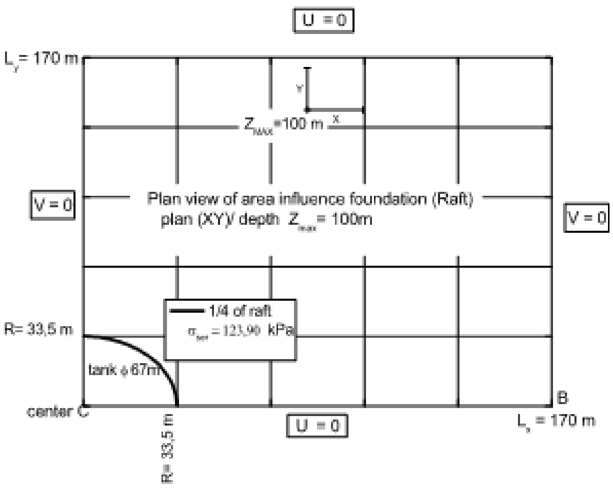


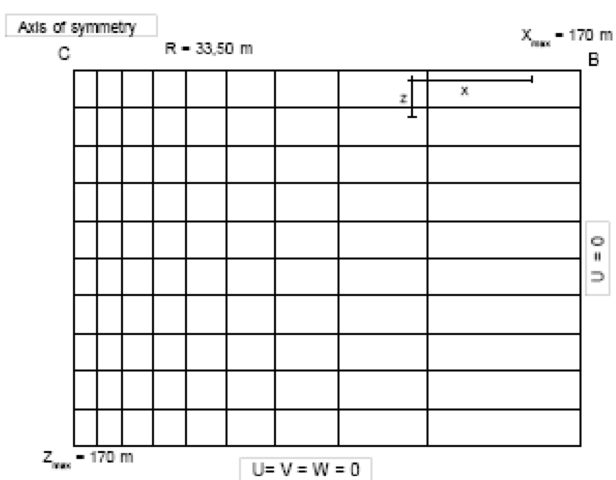
Fig. 16. Differential settlements observed along the perimeter of the tank C9 before and after treatment.

NUMERICAL ANALYSIS

In order to choose the foundation of new storage tank projects in the marine terminal area, finite element analyses were performed using the CESAR-LCPC software (ITECH-LCPC, 2004). A settlement analysis was conducted to estimate the deformations of tanks of 50000 m^3 capacity. Because the representation of the mechanical behaviour of the soil is one of the most important parts of a soil structure interaction analysis, four models were considered: linear elasticity, elastoplastic model with Mohr-Coulomb criterion, elastoplastic model with Drucker-Prager criterion and elastoplastic modified Cam-Clay model. Axisymmetric deformation (2D problem) calculations with reasonable assumptions were performed and will be compared with the three dimensional (3D) analysis. It can be expected that the settlements obtained in 2D cross sections would overestimate the settlements. To reduce the complexity and size of the 3D models symmetry axes were defined. Then, only one quarter of the construction is modelled. This is possible because the loaded area is almost symmetric and the influence of the non symmetric outer part of the tank is expected to be negligible. Figures 17a and 17b show the geometry and boundary conditions of 2D and 3D models. Considering the influence of tank of 67 m diameter, the model radius and depth are 170 m and 100 m respectively. The tanks were modeled as structures having a completely flexible base.



a) (2D) axisymmetric problem



b) (3D) problem.

Fig. 17. Geometry and boundary conditions of 2D and 3D models.

The soil profile for the finite element simulation is based on soundings with depths down to 40 m from the surface, and on in situ test results performed around the tanks. In some cases significant variability was found in the soundings around tank (Fig. 9). This lateral heterogeneity of compressible alluvial layers is not considered in the all numerical analyses performed, the thickness of each soil layer is assumed to be constant in the first approximation analysis (thickness of layers invariable in vertical direction).

For the axisymmetric model, 8-node quadrilateral elements were used to model the soil domain and the flexible base of the tank. The 2D model consists of 1795 nodes and 576 iso-parametric elements. The 3D model mesh comprised 17002 elements and 46299 nodes. Figures 18a and 18b show a typical finite-element mesh used in the analyses. The serviceability limit state (SLS) is about 120 kPa; it represents the weight of dress, floating roof and the stored crude oil. The input model parameters of each layers used to perform the calculations are summarized in tables 3 and 4.

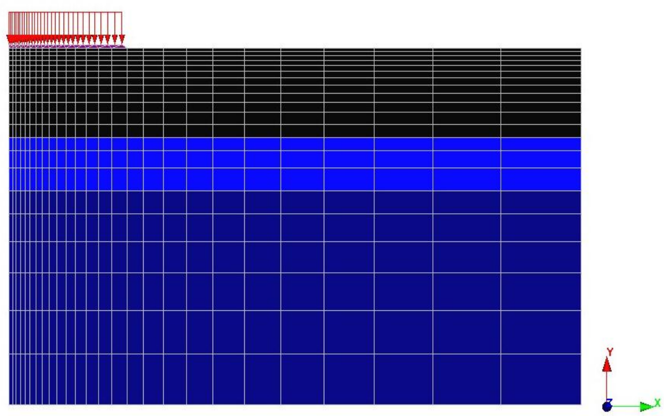
The calculated settlements under serviceability loads are shown in Figures 19 and 20 for 2D and 3D models. Figures 21a and 21b show the deformed mesh for the two models. Considering the modified Cam Clay model and 3D model, the vertical settlements calculated are about 480 mm at point C (center) and 155 mm at point B (edge). Between point B and C about 325 mm and 332 mm of differential settlements are expected for 3D and 2D model respectively. The predicted differential settlements are excessive. These results are in fairly good agreement with the measured differential settlements, which reached maximum values of 280 mm, 220 mm and 180 mm for the tanks C9, R13 and R21 respectively (Sonatrach, 2004). Taking into account the linear elastic model and the elastoplastic models with Mohr-Coulomb and Drucker-Prager criteria, the predicted differential settlement ai approximately about 140 to 200 mm between the edge and the center of the tank for the two considering problems.

Table 3. Modified Cam Clay model parameters.

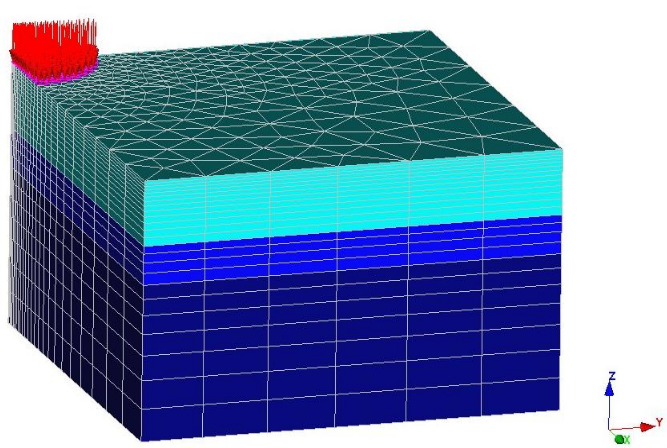
	Compressible alluvial layer	Plastic marl	Hard marl
E (kPa)	97	300	100
ν	0.35	0.30	0.25
G (kPa)	35.40	115	400
M	0.80	0.60	1.20
λ	0.117	0.055	0.02
κ	0.035	0.028	0.004
e_o	0.60	0.50	0.30
P_{co} (kPa)	1.55	3.00	7.50

Table 4. Model parameters.

Thickness of soil layers (m)	Nature of soil layers	Soil parameters
0.012	Flexible steel base	γ = 78.5 kN/m^3, E = 21 10^7 kPa, ν= 0.30
0.0 to 1.00	Compacted granular fill	γ = 19 kN/m^3 E= 4000 MPa, ν= 0.33 c=1 kPa, φ = 35°, ψ=5° α=0.273, β=0.034, k≈0
1 - 25	muddy sand and soft clay (high compressibility layers)	γ =18,8 kN/m^3, E= 9700 KPa, ν= 0,35 c=27 kPa, φ=14°, ψ=0° α=0.101, β=0, k=33 kPa
25 – 40	Plastic marl	γ = 20.5 kN/m^3, E= 300 bars, ν =0.30, c= 90 kPa, φ=15°, ψ=0° α=0.109, β=0, k=110 kPa
40 - 100	Hard marl	γ = 21 kN/m^3, E= 100 MPa, ν= 0.25 c=100 kPa, φ=30°, ψ=0° α=0.231, β=0, k=120 kPa

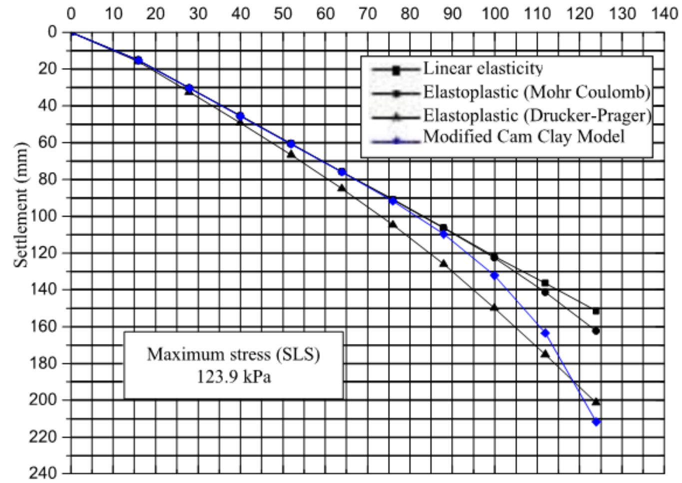


a) Axisymmetric 2D model

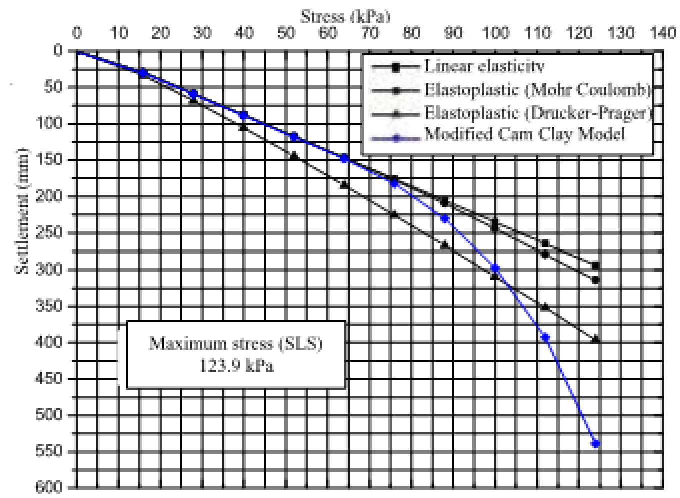


b) Quarter 3D model

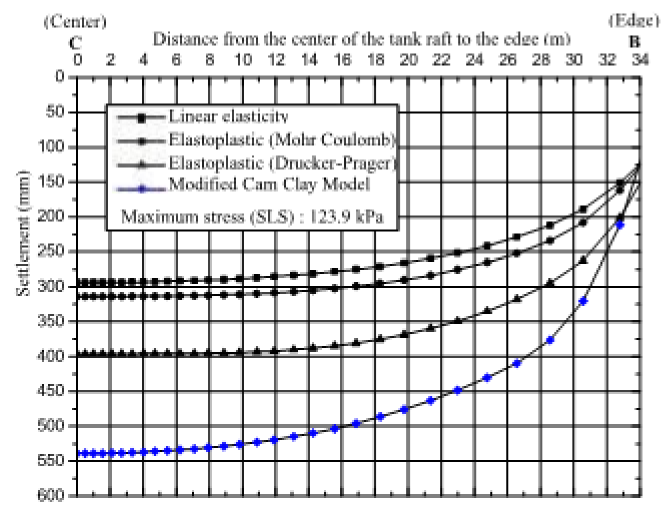
Fig. 19. Finite element mesh.



a) Edge (Point B)

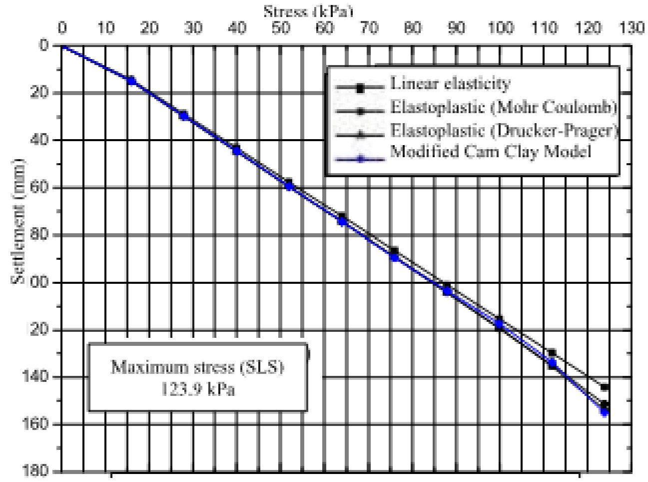


b) Center (point C)

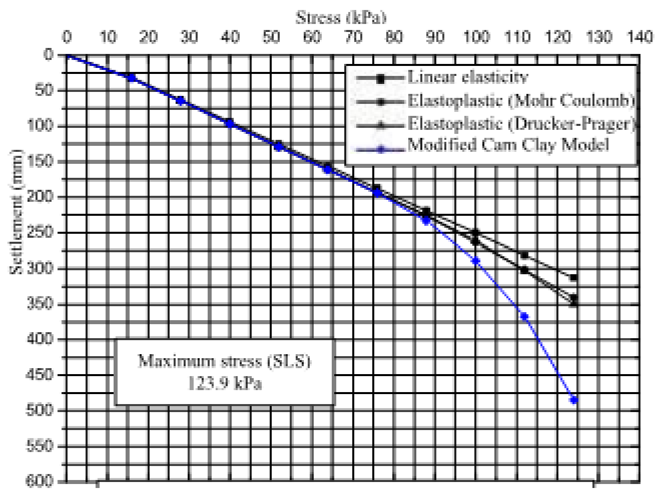


c) Half settlement profile.

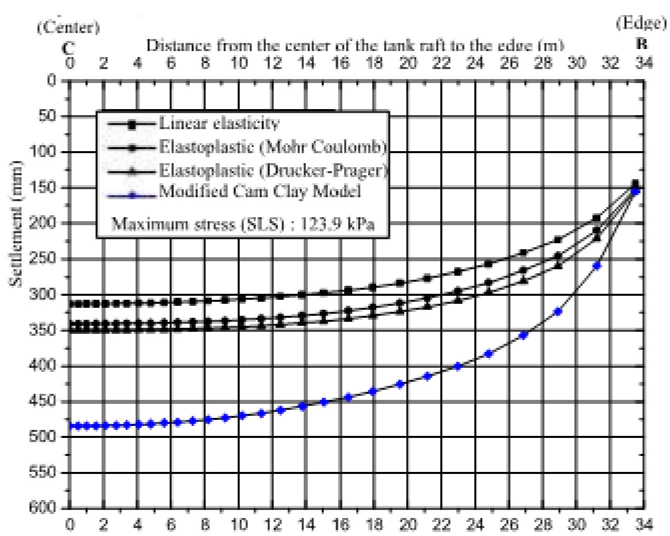
Fig. 19. 2D (axisymmetric) computed settlement.



a) Edge (Point B)

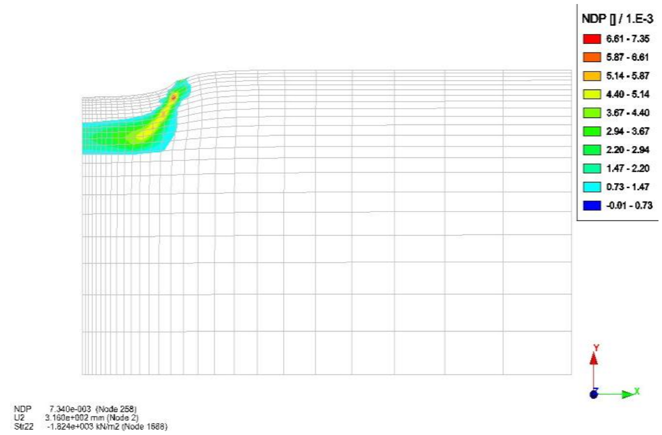


b) Center (point C)

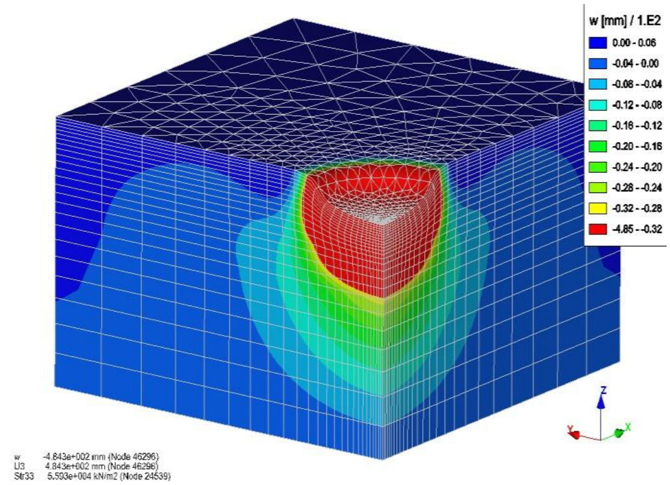


c) Half settlement profile.

Fig. 20. 3D model computed settlement of flexible base.



a) 2D model.



b) 3D model

Fig. 21. Deformed meshs.

CONCLUSIONS

A brief history of the marine terminal tanks of Bejaia has been presented. After about 25 years of satisfactory service, some cylindrical floating roof steel tanks were subjected to excessive differential settlements prejudicial to their stability. Because a distortion of the steel tank walls and jamming of the floating roof, some of them were considered unsafe for service.

The most comprehensive geotechnical investigation performed around the tanks, to evaluate the subsurface conditions of the site and to provide recommendations for foundation repair or retrofit of existing tanks, has been described. The key elements of this investigation were the local variability of foundation soils, the low consolidation and the high compressibility of the soil layers. The tanks were founded on difficult soil conditions. The heterogeneity and the compression of soft soil layers underlying the site seem to be responsible for the excessive differential settlements experienced by these tanks.

Based on the geotechnical investigation results and since the differential settlements are not tolerable, it was concluded that the subsurface soils underneath each tank to be improved. Micropiles were successfully adopted to solve the foundation problems of the tanks.

In order to choose the foundation of new storage tank projects in the marine terminal area, finite element analyses were performed to estimate the deformations of tanks of 50000 m^3 capacity. The analyses were conducted in axisymmetric deformation (2D) and three-dimensional (3D) problem. The predicted differential settlements under serviceability loads are excessive. These obtained results using modified Cam Clay model are in fairly good agreement with the measured differential settlements. The differential settlements obtained considering 2D and 3D models are very close.

This study shows that the soil settlement is a common problem in the harbor area of Bejaia. The constructions of heavy industrial structures in this area require deep foundations or soil improvement to reduce soil settlements. For the tank foundations, consolidation by micropiles is best suited to the site. The results of numerical analysis can help the designing engineers in his decision for different improvement techniques.

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