INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

Settlement of steel tanks in soft clays: Caripito tank farm case history

Tassement de réservoirs en acier reposant sur des argiles molles: Cas historique du réseau de cuves de Caripito

Iván A.Contreras, Ph.D. P.E. – *Geotechnical Consultant, Caracas, Venezuela*
Juan F.Lupini, Ph.D., P.E. – *Geohidra Consultores, Caracas, Venezuela*
Juan M.Pestana, Sc.D., P.E. – *Geotechnical Consultant, Caracas, Venezuela*

ABSTRACT: The need for increased oil storage as a result of expanded petroleum exploration in eastern Venezuela required the assessment of existing cylindrical steel tanks and the construction of new large capacity tanks. This paper describes the geotechnical investigation conducted at the Caripito Tank Farm and includes a summary of the tank farm's history. The geotechnical investigation included performing field tests (SPT, CPTU, FV, Seismic Cone and Electrical Resistivity); obtaining representative soil samples from borings and CPT soundings for laboratory testing and evaluating liquefaction potential, bearing capacity, and settlement of the new tanks. The results of the geotechnical investigation and the recommended foundation solutions and repairs are also presented and discussed here.

RÉSUMÉ: Le développement de la recherche du pétrole à l'est du Venezuela nécessite l'augmentation des capacités de stockage en pétrole, ce qui impose l'évaluation des cuves en acier existantes et l'élaboration d'autres réservoirs de plus grande capacité. Cet article décrit l'exploration géotechnique menée au réseau de cuves de Caripito et contient un résumé de l'histoire de ce site. L'investigation géotechnique a compris des tests sur le site (SPT, CPTU, FV, Cône Sismique, et Résistivité Electrique), l'obtention d'échantillons de sols représentatifs à partir de forages et de sondages de CPT pour des tests en laboratoire, et l'évaluation du potentiel de liquéfaction, de la capacité portante et du tassement de ces nouvelles cuves. Les résultats de l'investigation géotechnique et les solutions recommandées pour les fondations et les réparations sont aussi présentés et discutés.

1 INTRODUCTION

The Caripito Tank Farm is located within a wide meander of the Caripe River which is tributary of the San Juan river approximately 3.2 km to the west, and consists of 30 cylindrical steel tanks with a total capacity of 2,762,000 bbl (barrels) (cf. Figure 1). The tanks range in capacity from 30,000 to 150,000 bbl with diameters between 21.3 and 45.7 m. The TAP (Deep Water Terminal) project includes storage, transportation, and oil shipment from Caripito to a new terminal in Güiria and required the upgrade of the tank farm in order to meet larger storage capacity as well as satisfy new construction and environmental standards.

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. This paper presents a brief summary of past tank performance, in terms of foundation settlement, results of the geotechnical investigation, and the recommended foundation solutions.

2 HISTORY OF CARIPITO TANK FARM

The Caripito Tank Farm construction was initiated in 1931 when commercial oil deposits were discovered in eastern Venezuela. During the past 70 years, most of the tanks have undergone large settlements, creating the need, in some instances, of relieving the tanks. The compression of soft clay layers underlying the site are responsible for the large settlements experienced by these tanks.

In the past, several geotechnical studies have been conducted in the area to evaluate the tank foundations and to propose remedial measurements for the problem of large foundation settlement (Carlson and Fricano 1961, Galavís 1982, Hidromet 1992, Geotécnica de Venezuela 1992, INTEVEP 1997). Most of these studies recommended deep foundations but the solutions were not implemented. Carlson and Fricano (1961) presented a summary of the activities at the tank farm from its construction until 1960, including detailed information of measured settlements during hydrostatic loading and measured long-term settlement between 1951-60 for tanks 37, 38, and 39. In addition, they presented a summary of average shell settlement of the rest of the tanks for the period 1940-1960.

Figure 2 shows the measured settlement of tanks 37, 38, and 39 interior during the hydrostatic test in 1949-50. These tanks are of the welded cone-roof type with a 45.73-m diameter and 14.63-m in height. Due to the soft consistency of the clay deposits underlying the area, settlements in excess of 1 m were measured in these tanks (cf., Figure 2). During the hydrostatic load test, Tank 39 tilted badly to the southeast and exhibited excessive deformation. The maximum differential settlement along the shell of the tank was measured to be 85 cm. A shear failure was evident and the tank was considered unsafe for service. The remedial measure consisted of floating the tank off the base by flooding the area within the firewalls and towing it to a new location for foundation repair. After repair, the tank was put in service and underwent additional consolidation settlement. Carlson and Fricano (1961) show the settlement vs. time curves for a nine-year period for tanks 37, 38, and 39. The measured shell consolidation settlement for tanks 37, 38, and 39 for the period 1951-60 was 42, 60, and 75 cm respectively.

3 GEOTECHNICAL INVESTIGATION

The authors were involved in a comprehensive geotechnical investigation at the Caripito Tank Farm conducted by Geohidra Consultores C.A. in 1997-1998 (Geohidra, 1998). The objective of the investigation was to determine the subsurface conditions of the site and to upgrade the facilities for an increase in oil production. The fieldwork consisted of 46 Standard Penetration Test (SPT) borings, 44 Cone Pressure Test with pore pressure measurements (i.e., CPTU soundings), 2 Field Vane tests, 3 CPT seismic cone soundings, 20 electrical resisitivity tests, and a pumping test.

At 10 specified locations, cone penetration was stopped to measure the dissipation of excess pore-water pressure with time.

Dissipation tests allowed estimation of coefficient of consolidation and permeability of the clayey strata. To evaluate liquefaction potential, samples of silty sand and sandy silt materials susceptible to liquefaction were obtained to perform grain size analysis and determine fines content.

Thin-walled Shelby tubes were used to obtain samples in the borings from the clayey strata. Samples recovered were taken to the laboratory where index property tests were performed on recovered samples. In addition to index property tests, compressibility and strength tests including consolidation, triaxial unconfined compression (UC), unconsolidated undrained (UU), and consolidated undrained (CIU) tests were performed on undisturbed samples.

4 SITE CHARACTERIZATION

The results of the geotechnical investigation conducted by Geohidra were compiled and compared with previous investigations to develop representative profiles. This work is briefly summarized in the following paragraphs.

4.1 *Lithology*

The upper two and half to three meters consists of a desiccated crust composed of top soil, fill and a mixture of silty clay to clayey silt with sand. The consistency of the materials in this upper zone is stiff to very stiff. Below the desiccated crust, an erratic intercalation of cohesive and cohesionless soils was found.

Figure 1. Plan view of Caripito Tank Farm.

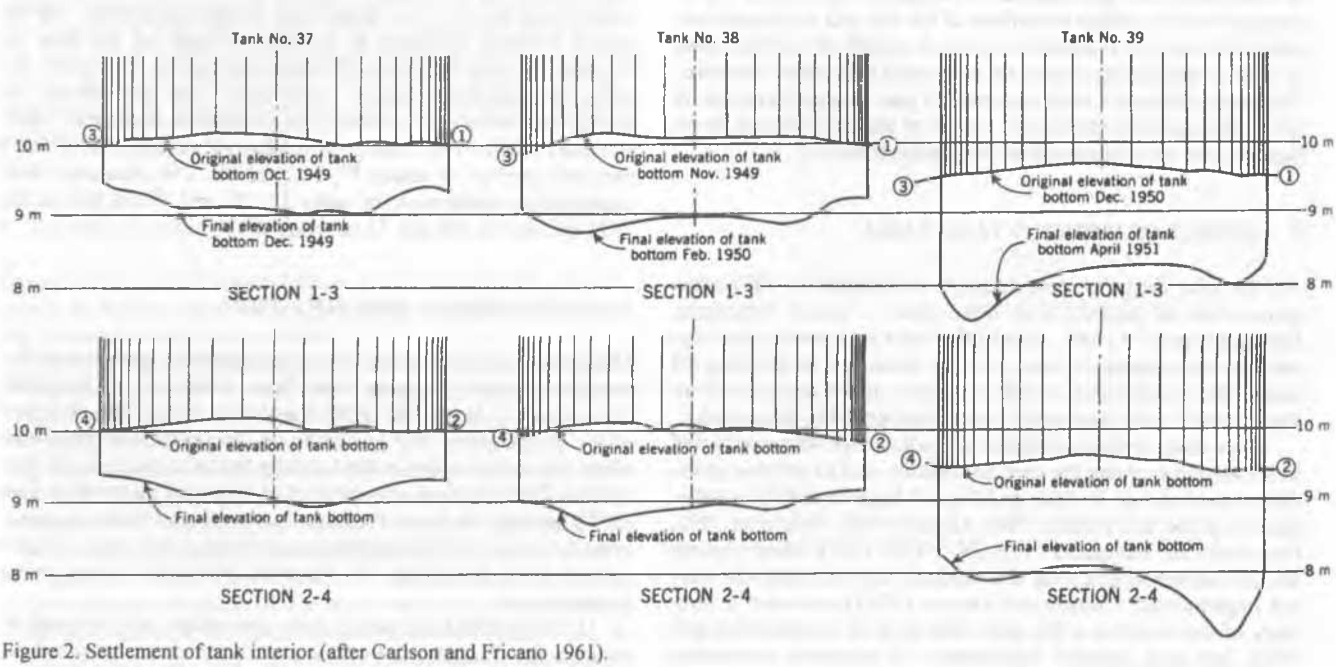

Figure 2. Settlement of tank interior (after Carlson and Fricano 1961).

The cohesionless strata consist of discontinuous silty sands and clayey sands occasionally mixed with gravel. These materials vary in thickness with relative densities ranging from loose to dense, N_{60} values in the range of 8 to 60 blows, and q_t in the range of 5 to 15 MPa. The cohesive materials mainly consist of clayey silt and silty clay with variable thickness and very soft to very stiff consistency. The plasticity index is generally in the range 40% to 60%. However, the clay becomes thicker and softer toward the east side of the tank farm where tanks 37, 38, and 39 are located. In this zone the clay is very soft and exhibits a thickness of approximately 25 to 27 m.

Underlying the erratic intercalation of cohesive and cohesionless soils at a depth of about 28 to 30 m, a firm stratum of very dense sand with some gravel is found. The ground water table is relatively shallow due to the proximity to the Caripe and San Juan rivers and the low ground elevation. The ground water is at an average depth of 3.6 m with daily variations due to tidal fluctuations.

4.2 *Compressibility*

Figure 3 shows the void ratio effective stress-relationship for specimens of Caripito clay. The best estimate of the in-situ effective stress is 84 kPa with an approximate preconsolidation pressure of 127 kPa obtained from standard consolidation tests. This corresponds to an overconsolidation ratio of 1.5. Caripito clay is highly compressible with values of compression index C_c and recompression index C_s of 0.93 and 0.17 respectively. The coefficient of consolidation c_v was also obtained from consolidation tests. In the recompression range the average coefficient of consolidation is 6.3 m^2/year, while in the virgin compression range the average coefficient of consolidation is 1 m^2/year.

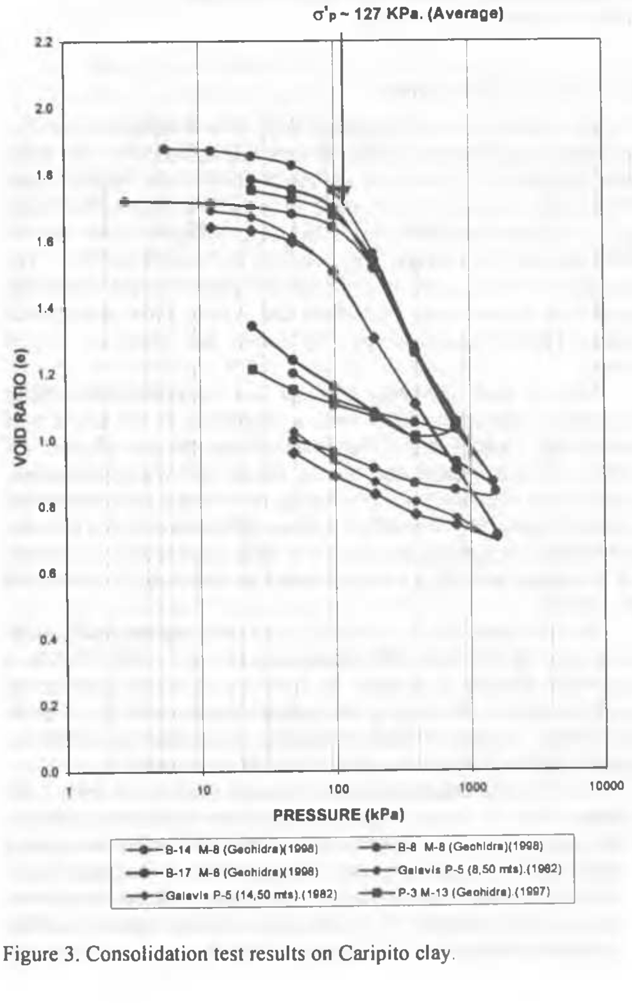

Figure 3. Consolidation test results on Caripito clay.

4.3 *Strength*

Figure 4 summarizes the undrained shear strength, s_u, profile of Caripito clay. The graph includes results from UC and UU tests as well as those estimated from CPT. In spite of the expected scatter (due to vertical and horizontal soil variability), a general trend of increasing strength with depth below the desiccated crust is observed. The undrained shear strength ratio (s_u/σ'_p) corresponding to the mean observed values is estimated to be 0.26. For interpretation of CPT data in terms of s_u, a cone factor N_{kt} of 15 was used and provides good agreement with the measured data. This cone factor value is typical for soft clay deposits of this plasticity as shown by Aas et al. (1987).

The shear strength of Caripito clay was also measured in a series of consolidated undrained triaxial (CIU) tests. Caripito clay specimens were subjected to consolidation pressures varying from 50 kPa to 480 kPa. The undrained shear strength ratio s_u/σ'_p was determined to be 0.33. For the cohesionless strata, results from the CPT were used to estimate soil parameters. The internal friction angle (ϕ') was estimated to be in the range of 29° to 36° using the correlation proposed by Robertson and Campanella (1983).

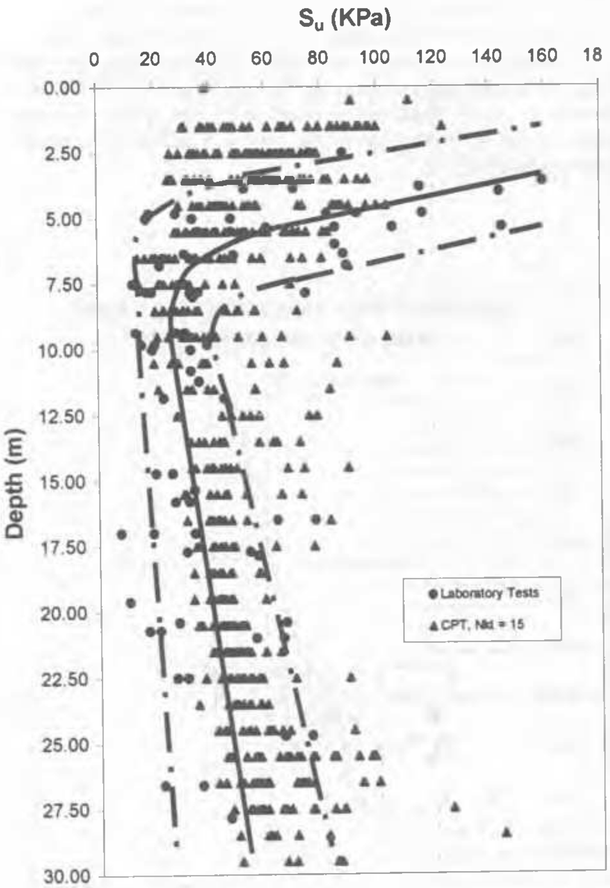

Figure 4. Undrained shear strength profile of Caripito clay.

5 ASSESSMENT OF SEISMIC PERFORMANCE

A site specific seismic hazard assessment and one-dimensional ground response analysis of the Caripito area was conducted. It indicated a peak ground surface acceleration of 0.29g for a 6.5 magnitude earthquake corresponding to a 500-year return period. These ground motion parameters were used in the seismic evaluation.

5.1 *Liquefaction Assessment*

The liquefaction potential was evaluated using the methods proposed by Seed and Idriss (1982), Stark and Olson (1995), and Tokimatsu et al. (1991), which utilize SPT, CPT, and shear wave velocity (V_s) data, respectively. The methods rely on the comparison of the seismic Shear Stress Ratio (SSR) with liquefaction resistance estimated from SPT, CPT, and v_s, depending on the method.

In all cases, the SSR for "level" ground sites was evaluated using the simplified procedure proposed by Seed and Idriss (1971). In the SPT method, the liquefaction resistance was estimated from the corrected penetration resistance $(N_1)_{60}$. The corrections for overburden and fines proposed by Liao and Whitman (1986) and Seed and De Alba (1986) were used, respectively. In the CPT method, the liquefaction resistance was estimated from the corrected normalized CPT tip resistance, q_{c1}. The equation proposed by Kayen et al. (1992) was used to compute the effective overburden stress correction factor. In the v_s method, the liquefaction resistance was estimated from the normalized shear wave velocity v_{s1}.

The boundary lines separating liquefaction and no liquefaction sites for 7.5 magnitude earthquakes were corrected to an earthquake of magnitude 6.5, using the magnitude correction factor c_m proposed by Seed et al. (1985). Figure 5 shows the results of liquefaction potential evaluation using CPT data. The liquefaction assessment, using the three methods, consistently shows many values lying to the left of the boundary line indicating potential liquefaction sites. Therefore, several of the tanks founded on sandy strata were found to be susceptible to liquefaction under an earthquake magnitude 6.5 and surface ground acceleration of 0.29 g.

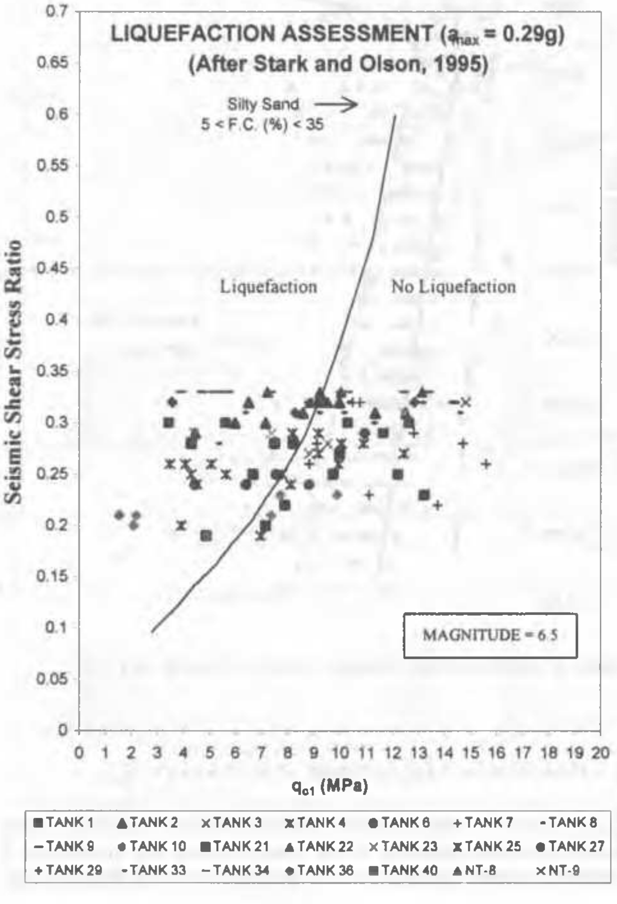

Figure 5. Liquefaction assessment at Caripito Tank Farm using CPT measurements

5.2 *Ground Surface Damage Susceptibility*

Liquefaction may occur at some depth below the ground surface and its effect may not be evidenced at the ground surface unless the liquefied layer is sufficiently extensive and the mantle of the non-liquefiable soil is relatively thin. The approach developed by Ishihara (1985), based on the relative thickness of the liquefied layer and non-liquefied soil cover, was used to discriminate between occurrence and non-occurrence of surface expression of ground rupture due to liquefaction. The results of the surface ground damage assessment indicated several locations where surface ground rupture is expected under such conditions. This information was used to define the area susceptible to liquefaction surface induced damage that was used to define the final layout. Tanks located in this area will require ground improvement.

Post liquefaction stability of the tanks that are not susceptible to ground ruptures but exhibit sand layers susceptible to liquefaction underneath them was also evaluated. In this analysis, the liquefied shear strength was used for the sands within the liquefied zone and estimated according to Olson and Stark (2000). The results of the analyses indicate that those tanks are stable and that no damage is expected under the design seismic event. Similarly, settlements induced by earthquakes were estimated. The procedure proposed by Terzaghi et al. (1996) was used in the analysis and the results indicate settlements in the order of about 0.15 m or less and can be tolerated by those tanks.

5.3 *Settlement*

A settlement analysis was conducted to estimate the deformations and identify tanks susceptible to excessive settlement. The analysis included immediate and consolidation settlements. To assess tank settlements, profiles were developed for individual tanks based on CPT results around the tank. In some cases significant variability was found in the soundings around a tank and required numerical modeling.

5.4 *Modeling Technique*

Tank deformations were modeled using a finite element code due to complex subsurface conditions and loading history. The tanks were modeled as structures having a completely flexible base. Subsurface conditions were used from individual profiles based on CPT results as previously described. In general the incompressible foundation layer was assumed to be at 28 to 30 m. The constitutive soil models used in the analyses were the Cam-Clay model for the soft clays (Schofield and Wroth, 1968) and elastic-plastic (Mohr-Coulomb-type) for sands and medium to stiff clays.

With the goal of having a model that represents the existing subsurface conditions fairly well, a calibration of the model was performed. Calibration of the model allows the use of more realistic soil parameters and thus a better settlement prediction. Calibration consisted of reproducing the interior tank settlement profile of tank 37 shown by Carlson and Fricano (1961). In the calibration the general shape of the tank interior and settlement of the center and edge were achieved as shown by Contreras et al. (1997).

The tool used for the modeling was the program SIGMA/W, developed by GEO-SLOPE International Ltd. SIGMA/W uses the finite element technique to simulate stress/deformation of earth structures allowing up to eight different constitutive models. Other features include modeling axisymmetric problems, drained and undrained conditions and different material types.

The analysis considered new floating roof tanks with a diameter of 48.33 m and 16.66 m high. Two tank configurations were simulated in the analysis: new tanks concentric to existing tanks and 2 tanks in new areas. Placing a new tank concentric to an existing tank intends to take advantage of the competent ground, which resulted from the consolidation process underneath the existing tank. This arrangement is very interesting and challenging to model because it requires simulation of very unique loading histories and material properties to predict tank settlements.

The results of the analysis are shown graphically for four tanks that exhibit shape C in Figure 6. D'Orazio and Duncan (1986) established three tolerable settlement criteria based on the profile shape. The settlement criteria utilized in this project were slightly modified from the values proposed by D'Orazio and Duncan (1986), as follows:

profile shape A, $\Delta\rho/D=0.013$ and maximum $\rho_{center} = 0.033$ D
profile shape B, $\Delta\rho/D=0.008$ and maximum $\rho_{center} = 0.025$ D
profile shape C, $\Delta\rho/D= 0.004$ and maximum $\rho_{center} = 0.01$ D.

The ratio $\Delta\rho/D$ is defined as follows:

$$\Delta\rho/D = (\rho_{center} - \rho_{edge}) / D$$

In which $\Delta\rho$ = differential settlement between the center and the edge of the tank, D= tank diameter, ρ_{center} =center settlement, and ρ_{edge} =edge settlement. A summary of computed settlements is presented in table 1.

Table 1. Computed Total Settlements

	Total Settlement			
Tank	Center (m)	Edge (m)	$\Delta\rho/D$	Shape
150001	0.790	0.736	0.0031	C
150002	0.792	0.376	0.0086	C
150003	1.550	0.441	0.0229	B
150004	0.335	0.352	0.0002	C
150006	0.338	0.336	0.00004	C
120007	0.995	0.456	0.0122	B
120008	0.842	0.343	0.0137	B

Even though some tanks meet the $\Delta\rho/D$ criteria, they exceed the maximum tolerable settlement and distortion limits shown in Figure 6. Therefore, it was concluded that foundation improvement was necessary to reduce settlement to tolerable values. Additional analyses were performed to evaluate the effectiveness of foundation improvement on the expected settlement. Three techniques were evaluated and found appropriate for ground improvement: ground modification, construction of a barrier, and surcharge load with vertical strip drains.

6 RECOMMENDATIONS FOR FOUNDATION IMPROVEMENT

To mitigate the liquefaction hazard, ground improvement consisting of stone columns (i.e. vibroflotation) was recommended for tanks located in the area susceptible to ground surface rupture. A preliminary design of the foundation improvement consisted of a triangular pattern of stone columns with a diameter of 0.8 m and a separation of approximately 2-m. The improvement technique was required to guarantee a minimum factor of safety against liquefaction of 1.2.

To mitigate potential damage to tanks caused by excessive settlements, three alternatives for settlement control were recommended. Settlement analyses indicate that when a new tank is installed concentric to an old tank and subsurface conditions consist of relatively thin (i.e. 3 to 5 m) soft clay overlying a thick sand, the soil between the edge of the old tank wall and the new tank wall would be squeezed out. This is caused by the additional shear deformations of the clayey soil induced by local overstressing, which undergoes local yielding. For those tanks the recommended solution consisted of creating a barrier that minimizes the tendency of the soil to be squeezed out. A sheet pile wall, a slurry wall or an equivalent system stiff enough to reduce the tendency for the soil to displace laterally was recommended for three tanks.

Ground modification techniques such as stone columns (Franki), vibro-replacement, or vibro-displacement to improve the strength and stiffness of the existing soil was recommended. These techniques were recommended at locations where the use of a barrier was not appropriate or was impractical. The configuration consisted of a triangular pattern of improvement points of approximately a 0.6 m diameter at a spacing of 1.2 m in concentric strips of 2 to 4 m in thickness.

Preloading with vertical strip drains was recommended for two tanks located in a virgin area. The design consisted of vertical strip prefabricated drains placed in triangular pattern at 1.2 m spacing and 12 m length. The limitation of this solution is the time associated with it, requiring 11 months to complete 85% of predicted consolidation.

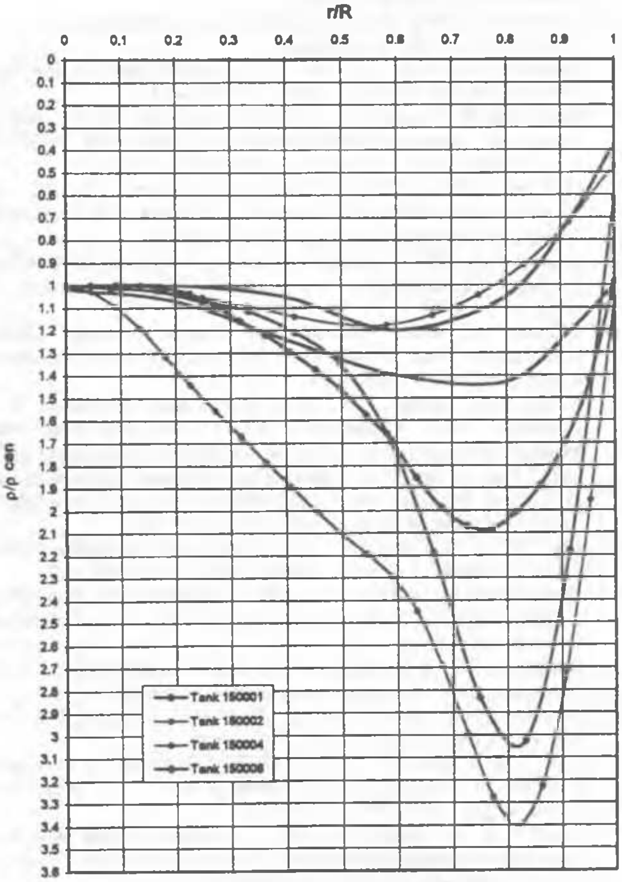

Figure 6. Normalized computed settlement profile for tanks with shape C

7 CONCLUSIONS

A brief history of the Caripito Tank Farm has been presented. The most comprehensive geotechnical investigation at Caripito Tank Farm has been described and relevant results on compressibility and strength of Caripito Clay using a variety of field and laboratory testing techniques have been presented. The key (and most challenging) element of this project was the local (vertical and horizontal) variability of problematic foundation soils, ranging from very soft and compressible clays to liquefaction prone cohesionless deposits.

Increased reservoir capacity should be achieved by the construction of new tanks and the use of a larger concentric steel ring on old tanks designed to take advantage of improved foundation conditions as a result of previous loading history. Numerical modeling was used to predict the performance of the proposed system and required the calibration of the model through modeling previous settlement profiles. Different ground remediation techniques were recommended on a case-by-case basis.

REFERENCES

- Aas, G., Lacasse, S., Lunne, T. & Hoeg. K. 1987. "Use of In-situ Test for Foundation Design on Clay." Use of In Situ Testing in Geotechnical Engineering, ASCE, *Geotechnical Special Publication, No. 6*, 1-30.
- Contreras, I. A., Furiol, A. & Rolo R. 1997. "Estudio Geotécnico de un Patio de Tanques en el Oriente del Pals" *Memorias de la 3 Conferencia Latinoamerica de Ingenieros Geotécnicos Jóvenes*, Caracas, 33-48.
- Carlson, E.D. & Fricano, S.F. 1961. "Tank Foundation in Eastern Venezuela." *J. Soil Mech. and Found. Div.*, ASCE, 87 (SM5), 69-90.
- D'Orazio, T. B. & Duncan, J.M. 1986. "Differential Settlements in Steel Tanks." J. Geotech. Engng, ASCE, 113 (9), 967-983.
- Galavís, M.L. 1982. "Informe de Fundaciones Tanques 37, 38, y 39" Terminal de Caripito, Edo. Monagas
- Geohidra Consultores C.A. 1998. "Geotechnical Study for New Tank Area and Pipeline Route in Caripito, TAP-Project."
- Geotécnica de Venezuela C.A. 1992. "Solución de Estabilización de Recalce de Tanques de Almacenamiento de Petróleo No. 26, 37, 38, y 39 Ubicados en el Terminal de Caripito, Edo. Monagas"
- Hidromet S.A. 1992. "Estudio de Suelos Tanques de Almacenamiento de Petróleo Crudo No. 26, 37, 38, y 39 Terminal de Tanques de Lagoven S.A. Caripito Edo. Monagas."
- Intevep, S.A. 1997. "Documento Técnico, Evaluación Integral de la Confiabilidad Geotécnico-Estructural del Patio de Tanques de Caripito." p. 129
- Ishihara, K. 1985. "Stability of Natural Deposits During Earthquakes." Proc. 11th Int. Conf. Soil Mechanics and Foundation Engineering, San Francisco, Vol. 1, 57-153.
- Kayen, R.E., Mitchell, J.K., Seed. R.B. Lodge, A., Nishio, S. & Cotinho, R. 1992. "Evaluation of SPT-CPT and Shear Wave-Bared Methods for Liquefaction Potential Assessment Using Loma Prieta Data." Proc. IV Japan-US Workshop of Earthquake Resistant Design of Lifeline Facilities and Countermeasure for Soil Liquefaction, Technical Report NCEER-24-0019, Vol. 1, 177-204.
- Liao, S.S. & Whitman R.V. 1986. "Overburden Correction Factors for SPT in Sand." *J. Geotech. Engrg.*, ASCE 112 (3), 373-377.
- Olson, S.M. & Stark, T.D. 2000. "Liquefied Strength From Liquefaction Flow Failure Case Histories." (in review) Canadian Geotechnical Journal.
- Robertson, P.K. & Campanella, R.G. 1983. "Interpretation of Cone Penetration Test." *Canadian Geotechnical Journal* 20 (4)
- Schofield, A.N. and Wroth, C. P. (1968) Critical State Soil Mechanics, McGraw-Hill Book Co., London.
- Seed, H.B. & Idriss I.M. 1971. "Simplified Procedure for Evaluating Liquefaction Potential During Earthquakes" *J. Soil Mech. and Found. Engng.*, ASCE 97 (9), 1249-1273.
- Seed, H.B. & Idriss I.M. 1982. "Ground Motions and Soil Liquefaction during Earthquakes." Earthquake Research Institute Center, University of California at Berkeley, 134.
- Seed, H.B. & De Alba, P. 1986. "Use of SPT and CPT Tests for Evaluating the Liquefaction Resistance of Sands." Proc. Of In-situ 86, ASCE Special Conf. On Use of In-situ Testing in Geotechncial Engineering, *Special Publication No. 6*, Blacksburg, 281-302.
- Stark, T.D. & Olson, S.M. 1985. "Liquefaction Resistance Using CPT and Field Case Histories." *J. Geotech. Eng. Div.*, ASCE 121 (GT12), 856-869.
- Seed, H.B., Tokimatsu, K., Harder, L.F., & Chung, R.M. 1985. "Influence of SPT Procedure in Soil Liquefaction Resistance Evaluations." *J. Geotech. Engrg.* , ASCE 111 (12), 1425-1445.
- Terzaghi, K., Peck, R., and Mesri, G. 1996. "Soil Mechanics in Engineering Practice." John Wiley & Sons, Third Edition, 675.
- Tokimatsu, K., Kuwzyama, S. and Tamura, S. 1991. "Liquefaction Potential Evaluation Based on Raileigh Wave Investigation and its Comparison with Field Behavior." *Proc. 2nd Int. Conf. On Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, ST. Louis, Vol. 1, 357-364. University of Missouri Rolla.