

SOIL IMPROVEMENT THROUGH RAMMED AGGREGATE PIERS TO MITIGATE LIQUEFACTION RISK IN STORAGE TANKS

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ABSTRACT

This paper presents the analysis, design, and construction of soil improvement for storage tanks foundation support at a site in Tabasco, Mexico. A total of 980 Rammed Aggregate Piers of 12.5 m in length and 0.5 m in diameter were designed and constructed to support four storage tanks. The soil conditions at the site generally consist of loose to medium dense silty/clayey sand to about 8m depth, underlain by very loose to medium dense silty/clayey sand with seams of very soft clay. The use of Rammed Aggregate Piers induced densification of the soil to increase its stiffness and reduce the liquefaction potential to control both static and dynamic induced settlements. Standard Penetration Test (SPT) and Cone Penetration Test (CPTu) explorations were performed prior to and post soil improvement to compare the liquefaction potential and the Factors of Safety associated with the two conditions. Finally, the soil improvement installation rate is presented. Soil improvement using Rammed Aggregate Piers represented a suitable alternative to support storage tanks foundation in potentially liquefiable areas.

Keywords: Soil improvement, Rammed Aggregate Piers, Liquefaction

INTRODUCTION

In the hydrocarbon sector, storage tanks represent an important geotechnical challenge due to the high bearing pressures resulting from the stored material. In addition, in areas with potentially liquefiable soils, this challenge is even greater. In these cases, deep foundations are typically used to support the service loads from the tank structure. An alternative to deep foundations is the use of Rammed Aggregate Piers to mitigate the liquefaction potential in sand-like behavior soils, thus increasing the resiliency of the structure. In cases where the soil or project conditions do not allow the use of shallow foundations, it is common to design deep foundations by evaluating the axial load capacity by analytical methods and in some cases, depending on the importance of the project, in-situ load tests are implemented. Also, the tolerance of the total and differential settlements is restrictive. Therefore, pile groups are frequently necessary. Due to the geometry of storage tanks, pile groups are placed following a circular array in order to distribute both static and accidental loads as uniformly as possible (Vega *et al.*, 2021).

This paper presents the analysis, design, and construction of soil improvement for storage tanks foundation support at a site in Tabasco, Mexico. Standard Penetration Test (SPT) and Cone Penetration Test (CPTu) explorations were performed prior to and post soil improvement to compare the liquefaction potential and the Factors of Safety associated with the two conditions.

CASE STUDY DESCRIPTION

The project consists in the construction of hydrocarbon storage tanks in Paraíso, Tabasco, Mexico. The tank diameters are 30.5 m for one tank, 18.3 m for two tanks, and 4.6 m for one tank with heights ranging from 7 to 12 m. Figure 1 shows a schematic of the tanks and the geotechnical conditions encountered at the site. A total of 980 Rammed Aggregate Piers (RAPs) of 12.5 m in length and 0.5 m in diameter were designed and constructed to support the four storage tanks.

The soil conditions found in the tank's footprint consists of 0.0 to 2.8 m, sandy material fill with silt (dredged); 2.8 to 6.4 m, very soft fat clay; 6.4 to 12.6 m (liquefiable layer), loose to medium dense sand/silty sand with seams of soft clay from 14.7 to 17.5 m, very soft clay with organics; 17.5 to 24.4 m, very loose to medium dense clayey sand with seams of clay; 24.4 to 31.5 m, medium dense to dense clayey sand; 31.5 to 42.0 m, medium to very stiff clay (See Figure 1). The groundwater table was found at approximately 3 m depth. Figure 2 shows the geometrical configuration of the RAPs proposed for each storage tank.

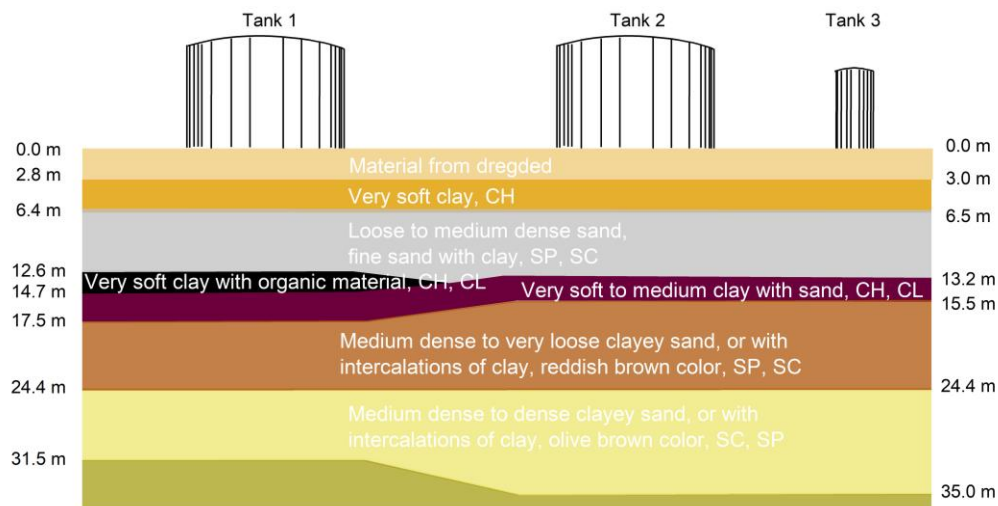


Fig. 1. Diagram of the storage tanks and stratigraphic cross section

GEOTECHNICAL SOLUTION

According to the geotechnical analysis and design, the post liquefaction induced settlement was reduced from 40 cm in free-field conditions, to 5 cm in improved conditions with the RAPs solution. The RAPs array was reviewed to meet the failure and serviceability criteria, such as, edge instability failure (static and dynamic condition) and static settlement. RAPs have been subjected to rigorous studies on their effectiveness in mitigating the risk of liquefaction of silty sands. For details of the study see Rollins *et al.* (2021).

Rammed Aggregate Pier (RAP) displacement installation process

The RAPs displacement construction process consists in:

The mandrel is driven into the ground using a strong static crowd force provided by the mast and augmented by dynamic vertical impact energy imposed by the vibro-hammer. In each installation, clean aggregate of nominal size from 12.7 to 38.1 mm ($\frac{1}{2}$ to $1\frac{1}{2}$ inch) is deposited through the hopper and mandrel. The equipment will rise 1.2 meters at a constant rate. The equipment is then driven back down providing static vertical thrust and high vertical impact energy provided by a vibratory -driving hammer, producing a rammed aggregate layer approximately 0.30m (1 ft) thick. Hydraulic thrust pressure will be recorded to

obtain a measure of vertical densification force. Additional vertical impact energy can be used depending on the specific site and according to installation procedures. This process is repeated until it reaches the surface, resulting in a high-density Rammed Aggregate Pier, and in the process densifying the surrounding soil by expanding the cavity by the displacement produced during the process (Figure 3).

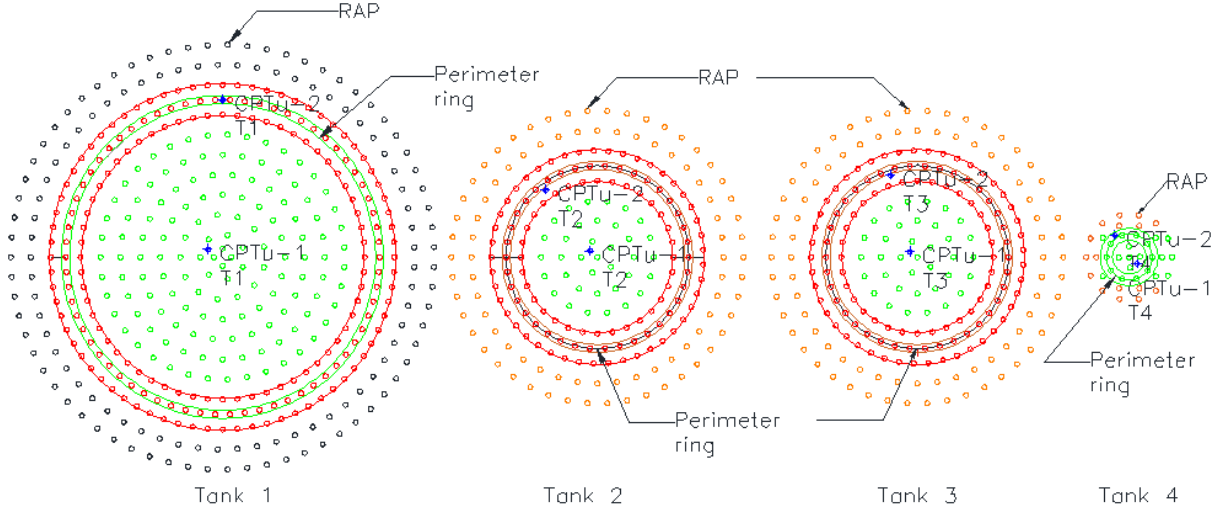


Fig. 2. Distribution of Rammed Aggregate Piers (RAPs) in each storage tank

Settlement control and design methodology

RAP soil reinforcing elements are designed to control foundations settlements to meet the project design criteria. Foundation settlements are estimated by summing the estimated settlement in the RAP-reinforced zone (the “upper zone”) and the estimated settlement in the zone of soil below the bottoms of the RAP elements (the “lower zone”) in accordance with the methodology described by Lawton *et al.*, 1994. These zones are depicted in Figure 4.

Upper zone settlement – spring analogy

Upper zone settlement calculations implement a spring analogy as shown in Figure 5. The RAP elements act as stiff spring; the matrix soil between the pier acts as softer springs. The stiff RAP elements attract a larger percentage of foundations-bottom stress than the soft springs. By assuming that the foundation is perfectly rigid, the top-of-RAP stress (q_g) may be computed using Equation 1:

$$q_g = q \frac{R_s}{R_a R_s - R_a + 1} \quad [1]$$

where q is the average foundation-bottom stress, R_s is the ratio of the stiffness of the RAP elements and the matrix soil, and R_a is the ratio of the area coverage of the RAP elements to the gross footprint area.

The stiffness ratio, R_s , may be defined as the ratio of the spring constants of the RAP (k_g) and the matrix soil (k_m), equation 2. The RAP spring constant is defined as the relation between of the stress applied at top of pier element and the resulting vertical strain. The RAP spring constant is estimated using a database of spring constant established from past RAP modulus test and then verified with a modulus test performed at the project site.

$$R_s = \frac{k_g}{k_m} \quad [2]$$

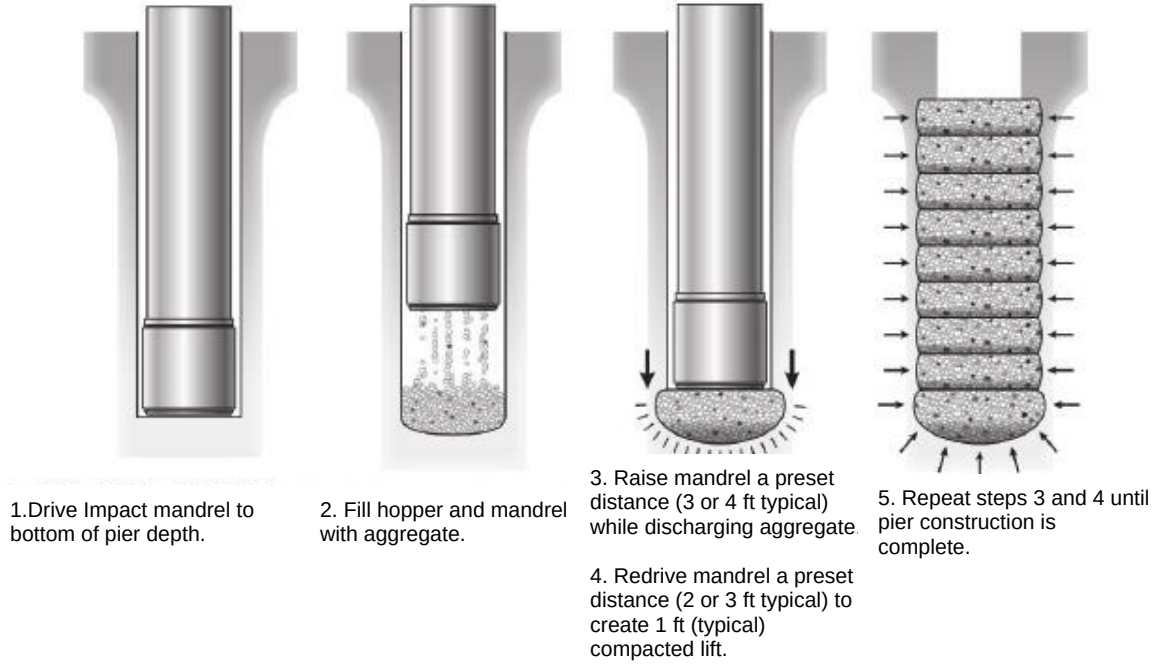


Fig. 3. a) Construction Method of the Rammed Aggregate Pier System and b) photograph of RAP installation at the site

The settlement in the Upper Zone (s_{uz}) is simply computed as the of q_g to the RAP spring constant (k_g), Equation 3.

$$S_{uz} = \frac{q_g}{k_g} \quad [3]$$

Upper zone settlement – composite modulus

Settlements for mat foundations and large footings, or large areas associated with embankments, tanks, silos, or other structures may also be evaluated using a composite modulus approach where settlement within the upper zone of soil (zone of soil reinforced with the RAP system) is estimated using conventional settlement calculations as shown in the following equation (adapted from Terzaghi et al., 1996):

$$S_{uz} = \frac{q I_{\sigma} H}{E_{comp}} \quad [4]$$

Settlements within the “lower zone” (zone of soils beneath the upper zone which receives lower intensity foundation stress) are computed using conventional geotechnical settlement methods that involve: estimating the depth of stress influence below the foundation bottom (typically taken as twice the foundation width for square foundations), estimating the foundation-induced stress in the lower zone (established using conventional influence factor charts), and estimating the compressibility of the lower zone soils.

Lower zone settlements (s_{lz}) in granular soils are estimated using elastic settlement methodology with the equation:

$$S_{lz} = \frac{q H_{lz} I_\sigma}{E_s} \quad [6]$$

where q is the average foundation-bottom stress, I_σ is the influence factor in the lower zone, H_{lz} is the thickness of the lower zone, and E_s is the secant modulus of the soil in the lower zone.

Lower zone settlements (s_{lz}) in cohesive soils are often estimated using consolidation approaches as shown in the following equation:

$$S_{lz} = m_v * \sigma'_{v0} * H_0 \quad [7]$$

where σ'_{v0} is the vertical effective stress at the midpoint of the layer (H_0) and m_v is coefficient of volumetric variation. Alternatively, lower zone settlements in cohesive soils can be estimated using elastic settlement methodology.

Total settlements

The estimated settlement of RAP-supported foundations (s) is determined by summing the upper zone and lower zone settlement values:

$$S = S_{uz} + S_{lz} \quad [8]$$

Liquefaction susceptibility analysis

The liquefaction susceptibility analyses conducted for free-field conditions were performed with the semi-empirical methods proposed by Idriss and Boulanger in 2008 and Boulanger and Idriss, 2014. Therefore, a detailed explanation of the methods is not provided, this can be consulted in the cited references. For the case of liquefaction analysis of improved soils, the criteria adopted is detailed below.

Liquefaction susceptibility analysis with soil improvement based on Rammed Aggregate Piers

The installation of Rammed aggregate piers in liquefiable soils replaces loose soils with dense and rigid reinforcement elements, increasing the density of the surrounding soil significantly. The use Rammed aggregate Piers reduces the liquefaction potential by the following conditions:

The degree of densification generated by the installation of RAPs can be seen by an increase in the number of blow counts in the SPT test of the matrix soil in sands. This increase depends mainly on the percentage of fines content in sand.

$$N'_{SPT} = N_{SPT} + \Delta N_s \quad [9]$$

where N'_{spt} is the number of SPT blow counts improved when considering the densification of the matrix soil, ΔN_s is the increment of SPT blow counts due to densification and N_{spt} is the number of SPT blow counts on the soil during initial conditions (no improvement).

The vertical compaction-ramming of the RAP increases the lateral stress of the adjacent soil. Hence, an increase in lateral stress results in an increase in the CRR cyclic resistance ratio (Rollins *et al.*, 2021).

POST-IMPROVEMENT REVIEW

As a result of the liquefaction susceptibility analysis for free field conditions, the amine water zone presents high liquefaction potential in the foundation soils. The potentially liquefiable sand layer is found from a depth of 6.5 m to 20 m, and a post-liquefaction induced settlement of 37 to 45 cm were estimated. With the results obtained in a punctual manner with the data from the borings performed in the area and according to their coordinates, isovalue curves were generated using the Kriging interpolation method, which assumes that the distance or direction between the sample points reflecting a spatial correlation can be used to explain the variation of the post-liquefaction induced settlements at ground elevation (Figure 6a). The soil improvement proposal with RAPs for the tank foundations consists of the following: installation of piers with a diameter of 0.50 m (installed) to a depth of 12.5 m from the existing ground elevation after site grading; on-center (o-c) spacing of 2 m (6.6 ft), in a concentric array for both, the tank foundation footprint and the confinement rows extending beyond the tanks footprint; o-c spacing of 1.5 m (5 ft) for the footing support or perimeter ring foundation. With the results obtained from the borehole data with the proposed improvement system and similar to the case without improvement, isovalue curves were generated at surface level (Figure 6b). This figure shows quantitatively the effect of the improvement on the decrease of post-liquefaction settlements.

The proposed solution using RAPs was constructed in a period of 50 days; under favorable working conditions, an efficiency of up to 575 ml/day was obtained. Subsequently, to validate its effectiveness, complementary geotechnical explorations were performed through CPTu and SPT tests. Comparison of the pre-improvement and post-improvement explorations showed the RAPs effectiveness in reducing the liquefaction potential of the site. To quantify the degree of soil improvement, pre- and post-improvement borings were performed at each tank location. The SPT results are shown in Figure 7, where a significant increase in the SPT blow numbers obtained can be observed in the four sites after applying the RAP improvement. In addition, CPTu tests were performed.

Figure 8 shows the increase in the cone tip resistance at the site where tank 1 is located. The same figure shows the Safety Factors calculated before and after soil improvement.

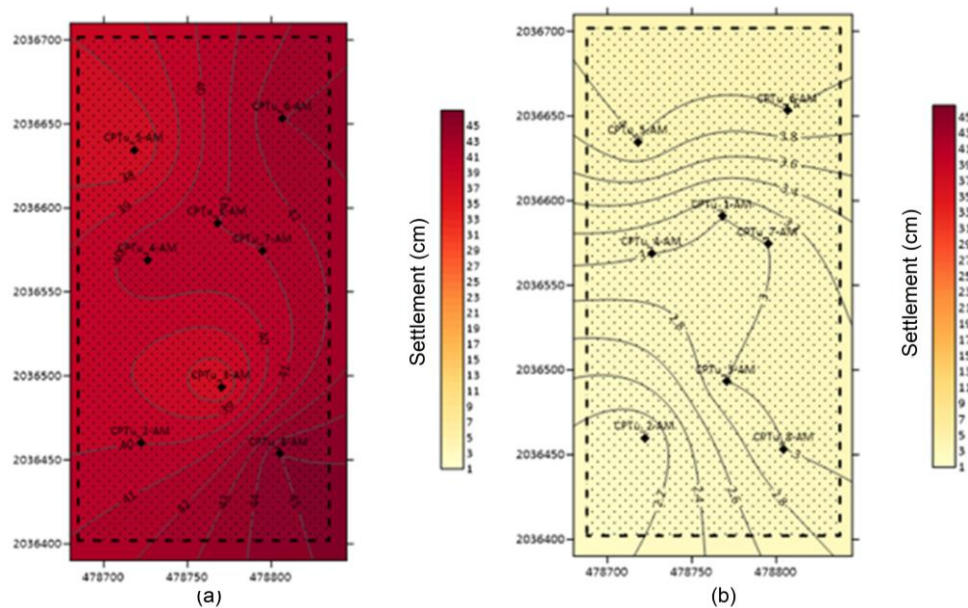


Fig. 6. Isovalue Curves of Post-Liquidation Induced Settlements at Surface Level for a) the Free Field Condition and b) for the Improved soil condition.

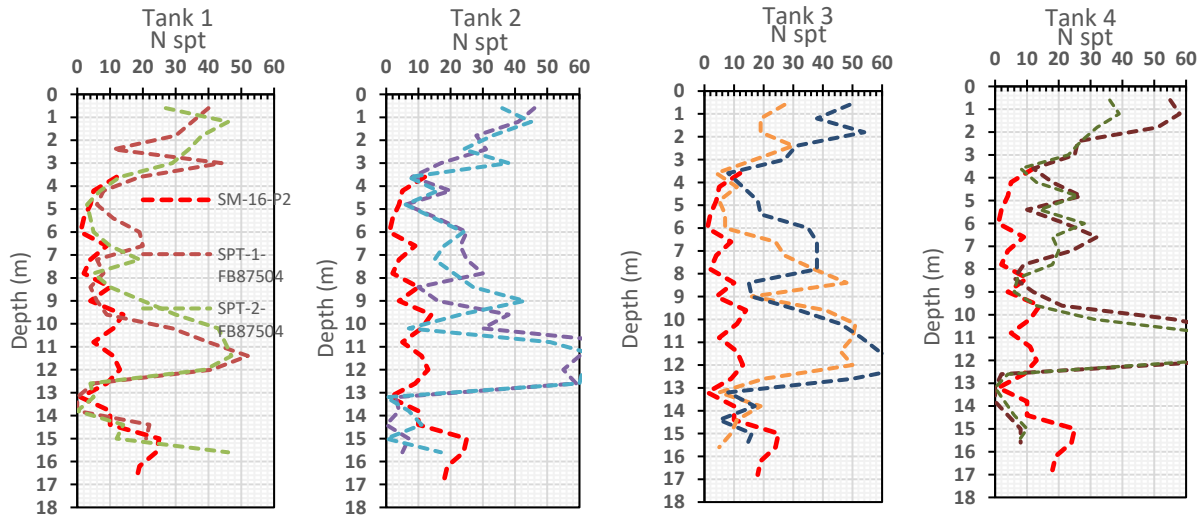


Fig. 7. SPT records in the four tanks before and after soil improvement with RAP

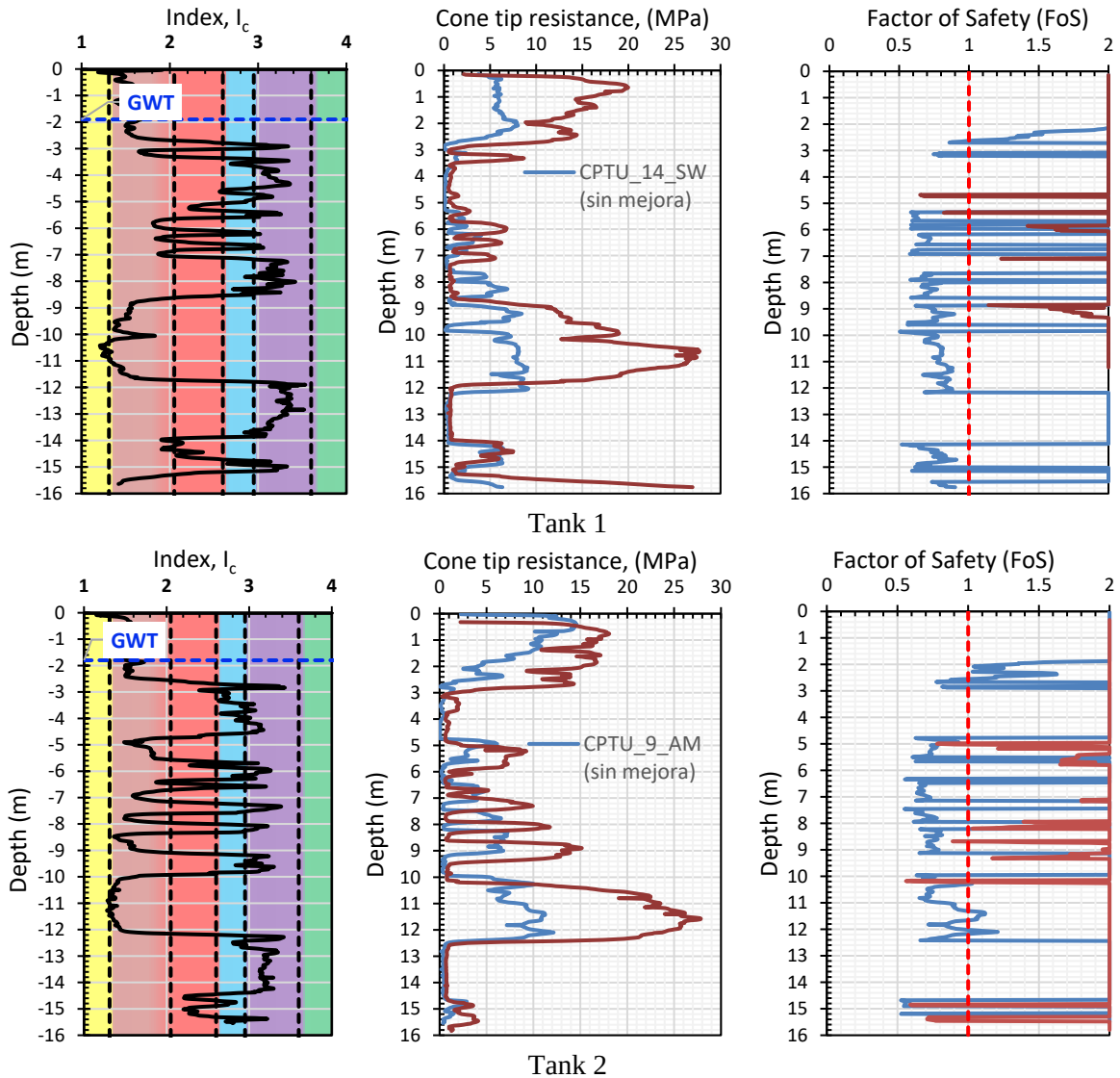


Fig. 8. CPT records in two of the four tanks before and after soil improvement with RAP

Figure 9 presents the daily installation rate obtained when constructing the 980 RAPs in less than 7 weeks of work. The average performance was 35 piles of 12.5 m in length per day.

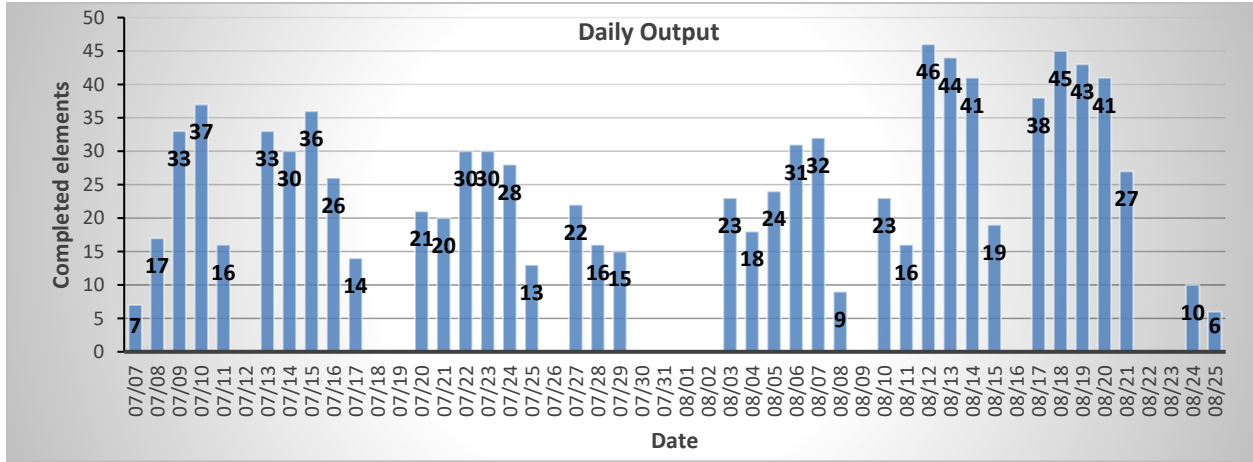


Fig. 9. Plot of daily installation rate in the construction of RAP's

SUMMARY AND CONCLUSIONS

In the reinforced zone, the RAPs provided lateral compaction of the matrix soil, which induces an increase in lateral and confining stresses and in stiffness. In addition, they provide radial drainage of the matrix soil which allows for rapid excess pore pressure dissipation. Radial drainage provided by the RAPs also accelerates the consolidation process of clay layers. This causes settlements to occur relatively immediate in the reinforced zone. The results of the settlement analysis for tank loading conditions satisfy the allowable settlement criteria adopted in the design baselines. To obtain these settlements, a three-dimensional analysis was conducted using Rocscience Settle 3 software. The depth of RAPs reinforcement is 12.5 m from the finished floor elevation of the platform and with on-center element spacings of 2 m for the flexible bottom foundation of the tanks. For the ring footings, a 1.5m o-c spacing between elements and rings was used. The proposed RAP arrangement will be able to reduce the post-liquefaction induced settlements to 5 cm relatively uniformly for the improved area. The soil improvement effectiveness was verified by CPTu and SPT tests performed before and after soil improvement.

A Limit Equilibrium (LE) analysis was also performed to evaluate the edge stability under static loading condition considering the improved shear strength parameters due to the reinforcement provided by the RAPs. The composite shearing strength of RAP-reinforced soils is computed using the conventional method of calculating the weighted average of the shear strength components of the RAPs and matrix soil materials (FHWA 1999). The composite shear strength is expressed in the following equation (FitzPatrick and Wissmann, 2002):

$$\tau_{comp} = \sigma'_v \tan \phi'_{comp} + c'_{comp} \quad [10]$$

The composite cohesion intercept (c'_{comp}) is computed with the expression:

$$c'_{comp} = c'_g R_a + c'_m (1 - R_a) \quad [11]$$

Where c'_g is the cohesion intercept of the RAP aggregate, c'_m is the cohesion intercept of the matrix soils, and R_a is the ratio of the RAP area to the gross footprint area of the reinforced soil zone. Because the cohesion intercept of the RAP aggregate is zero, Equation (11) reduces to:

$$c'_{comp} = c'_m (1 - R_a) \quad [12]$$

The composite friction angle (ϕ'_{comp}) is computed with the expression:

$$\phi'_{comp} = \arctan [R_a \tan \phi'_g + (1 - R_a) \tan \phi'_m] \quad [13]$$

Where ϕ'_g is the friction angle of the RAP aggregate and ϕ'_m is the friction angle of the matrix soils. RAP friction angles ranging from 48 to 52 degrees have been measured in the laboratory and in the field, but for design purposes, slightly lower friction angles (typically 45 degrees) are often used. As a result of the LE analysis, the pressures produced by the tank at the edges are within the allowable range and do not generate edge instability, which is reflected in a Factor of Safety $F.S > 1.5$.

For the dynamic condition, a pseudo-static limit equilibrium analysis was performed considering the maximum and minimum pressures corresponding to the hydrodynamic condition. In addition, the hydrodynamic model was subjected to a horizontal seismic coefficient taken as 50% of the Peak Ground Acceleration, PGA, taken from the seismic design spectrum ($PGA=0.25g$). As a result, it was obtained that the pressures produced by the tanks at the edges are within the permissible range and do not generate an edge instability as is reflected in a Factor of Safety of $F.S > 1.1$.

Soil improvement using Rammed Aggregate Piers represented a suitable alternative for foundations of storage tanks in potentially liquefiable areas.

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