Performance Monitoring of Rammed Aggregate Piers® (RAPs)

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ABSTRACT: *In this paper, settlement performance during water testing of structures at a waste water treatment facility in Turkey, constructed on soils improved by Rammed Aggregate Pier® (RAP) System, are presented. The soil profile is comprised of firm to stiff silty clay and medium dense silty sand of 10 m thickness overlain on thick soft to medium stiff silty clay with thin inclusions of sand lenses. The main goals of the in-situ soil improvement, to eliminate the risk of liquefaction and to form a homogeneous crust to reduce the total and differential of settlements was achieved by improving the soil with RAPs down to 15 m depth. In order to verify the design parameters, two kinds of field load tests, modulus load test and areal loading test were performed. Completed structures water tested and settlements are recorded, providing performance monitoring data under service loading conditions. With the implemented soil improvement, post construction settlements are reduced to 14 - 25 cm compared to initially estimated 20 - 80 cm long term settlements, and differential settlements are reduced to permissible limits.*

KEYWORDS: Impact rammed aggregate piers, stiffness, consolidation settlement, monitoring, ground improvement.

SITE LOCATION: 40°41'42.79"N 29°24'13.60"E

INTRODUCTION

The need of ground improvement methods has increased significantly in the recent years due to the need for construction of transportation, hydraulics and industrial structures at unfavorable soil conditions. Among existing alternatives, Rammed Aggregate Pier® (RAP) solution which was developed by Fox at USA in 1980's has been listed and served as an alternative to deep foundations or over excavation and replacement of compressible soils. RAPs are mainly used to reduce intolerable settlements, mitigate the liquefaction potential, reinforce slopes and improve the bearing capacity of footings, mat foundations, embankments, reinforced earth walls, transportation and port structures, etc. in Turkey as a cost-effective solution for construction on soft/compressible soil layers. Besides, it is expected that vibration and volumetric densification during construction of RAPs provide an additional benefit to increase strength and stiffness properties of cohesionless soils (sandy, gravelly and relatively non-plastic silty material).

Within the context of this manuscript, the settlement performance of structures at a waste water treatment facility where foundation soils were improved with 50 cm diameter RAP Impact elements are assessed. The settlement of structures are analyzed with Settle 3D, RocScience software program using the information from site soil investigation and compared with instrumentation data collected from a test embankment. The settlement behavior is further analyzed by comparing the estimated consolidation settlements and the recorded settlements during the water tests performed after the construction of structures. Before discussing the field load tests and their results, installation methodology of RAPs along with site soil profile will be explained.

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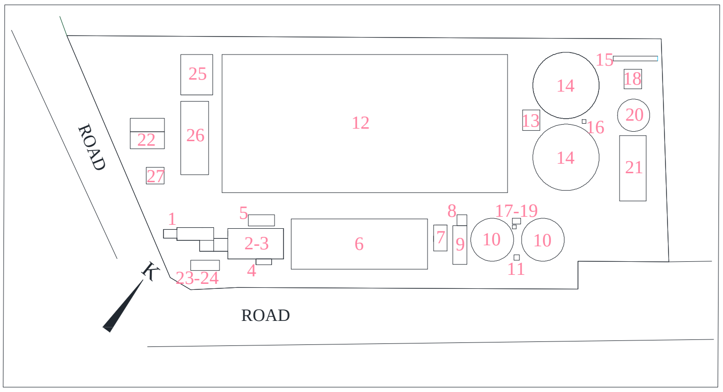
PROJECT DESCRIPTION and SUBSURFACE CONDITIONS

The project is located on a flat topography in the north of Yalova - Izmit Highway, between Yalova city center and Topcular pier. The north and north eastern sides, east and south eastern sides and south and west sides of the site are surrounded by the Marmara Sea, factories and some empty lots, respectively. The ground level is around at sea level and the maximum elevation is +1.0 m. The project site, planned to be used for the waste water treatment facility, is shown in Figure 1. The performance of main structures which cover large areas and seated at near surface layers are considered to be critical with respect to settlements. The foundation pressures from these main structures are around 80 - 110 kPa. In addition to these main structures, auxiliary structures such as distribution tanks and ducts, pumping stations, operational and administrative buildings all connected to each other have foundation pressures around 50 - 70 kPa. A schematic view of the facilities is shown in Figure 2.

Figure 1. Location of the site.



Main Structures:
No 12 : Aeration Tank
No 6   : Balancing Tank
No 14 : Biological Tanks

Figure 2. The layout plan of waste water treatment facility.

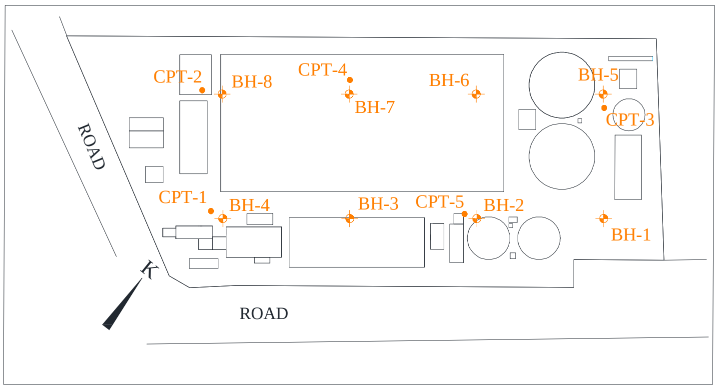


Figure 3. Boring and CPT location plan.

An extensive site investigation program, involving 25 to 35 m deep boreholes at 8 different locations and 20 to 26 m deep CPT soundings at 4 different locations were executed as shown in Figure 3. At various depths, standard penetration tests were performed along with the disturbed and undisturbed soil sampling. On the retrieved disturbed and undisturbed soil samples, soil classification, unconfined compression and consolidation tests were conducted.

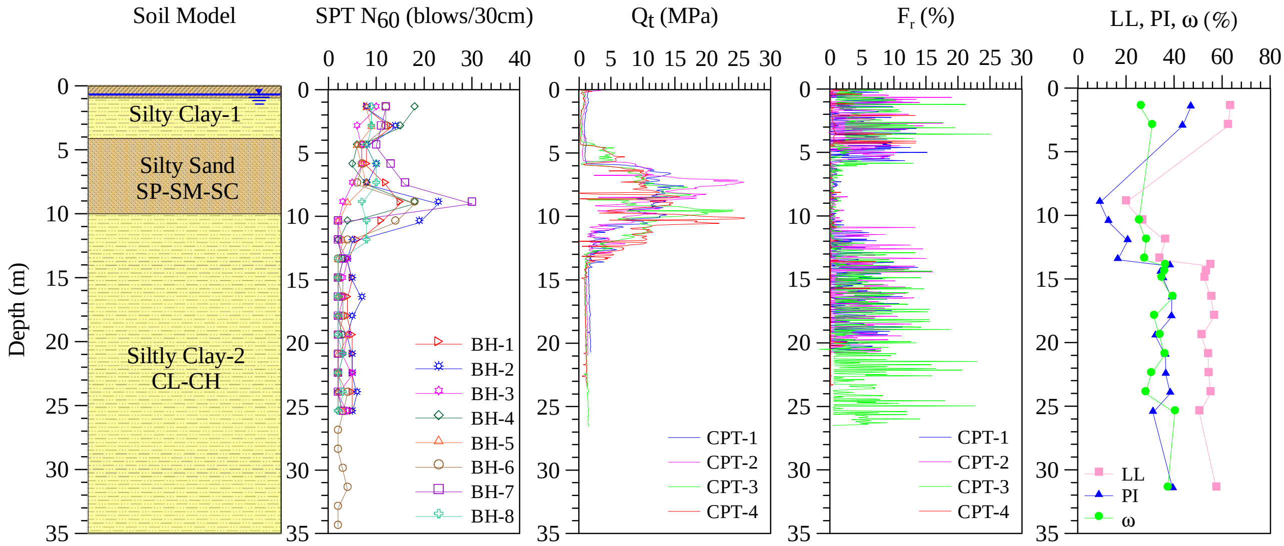


Figure 4. The representative soil profile, the variation of SPT N_{60}, Q_t, F_r, LL, PI and ω with depth.

The representative soil model, together with variations with depth of corrected cone tip resistance (Q_t) and friction ratio (F_r) obtained from cone penetration tests, SPT N_{60} values obtained from standard penetration tests, liquid limit (LL), plasticity index (PI) and natural water content (ω) are shown in Figure 4. The soil profile includes a 0.2 - 1.0 m thick top soil layer overlying a medium stiff to stiff silty clay layer down to a depth of 4.0 m. Below this clay layer, a 6.0 m thick loose to medium dense silty sand layer is encountered which overlies very thick soft to medium stiff silty clay layers with thin inclusions of loose to medium dense silty sand lenses. The ground water table is reported to be at 0.15 - 0.7 m below natural ground surface. Table 1 shows the summary of soil parameters.

Table 1. Summary of soil parameters.

Material	γ (kN/m^3)	ω (%)	LL (%)	PI (%)	c_u (kPa)	ϕ (°)	E_s (MPa)
Silty Clay	18.0	27	63	47	50	25	7.5
Silty Sand	18.0	-	-	NP	-	30	25
Silty Clay-2	18.0	44	48	31	50	25	7.5

γ : Unit weight (Robertson and Cabal, 2010)
ω : Natural water content, average values of laboratory test results
LL : Liquid limit, average values of laboratory test results
PI : Plasticity index, average values of laboratory test results
c_u : Undrained shear strenght, $c_u=(q_t-\sigma_v)/N_{kt} \rightarrow N_{kt}=14$ (Robertson and Cabal, 2010) (for Silty Clay-1 layer); $c_u=q_u/2$ (for Silty Clay-2 layer)
ϕ : Friction angle, relationship of PI and sinϕ' for clay layers; $\phi'=27.1+0.3N_{60}-0.00054(N_{60})^2$ for sand layer (Das, 2014)
E_s : Deformation modulus, $E_s=(3\sim8)q_c$ for soft clay, clayey silt; $E_s=(3\sim6)q_c$ for clayey sand (Bowles, 1996)

Design Consideration

At the preliminary design stage elastic and consolidation compression response of the site under loads to be imposed by the structures is assessed by using Settle 3D, RocScience software, which enabled 3D settlement analysis. Settle 3D model used for the analyses is shown in Figure 5. Assumptions used in the calculations were: i) a "flexible" foundation assumption is adopted to assess the differential settlement potential of the site; ii) Boussinesq stress distribution rule is adopted for the estimation of stress increase beneath loaded areas. In the consolidation settlement computation, for soft clay and medium stiff clay layers the compression index ratio is taken as of $C_c/1+e_0=0.12$ and $C_c/1+e_0=0.06$, respectively, and OCR=1.0. The consolidation settlements are estimated to vary in the range of 20 to 82 cm under the service loads, and their distribution is shown Figure 6.

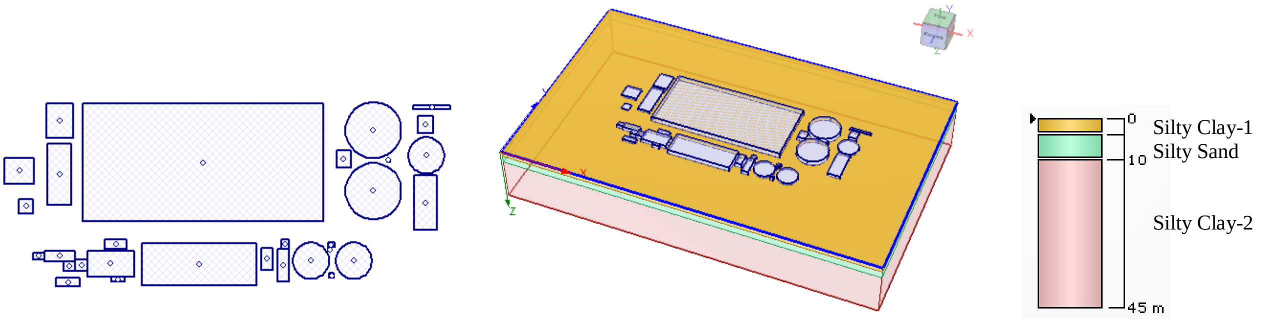


Figure 5. Settle 3D model.

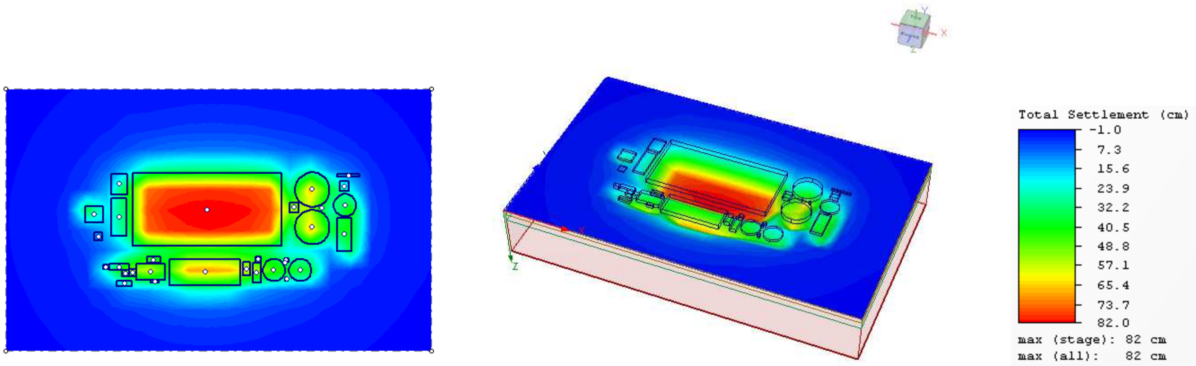


Figure 6. Estimated total settlement without improvement.

In order to eliminate liquefaction induced strength and rigidity losses of bearing layers under the foundations (the liquefaction triggering potential of silty sand layers with fines content varying between 10% to 40%, typically 20 %, were identified under a design earthquake motion of maximum acceleration $a_{max}=0.40$ g and moment magnitude $M_w=7.5$) and to limit the excessive surface settlements, it is decided to implement a soil improvement solution. The total elimination of the settlements is considered be a task not easily (or economically) achievable, and found to be not necessary for the proposed use of this site. Hence, the main goal of the in-situ soil improvement is defined as to form a thick homogeneous crust with improved soil properties under the foundations. The detrimental effects of the differential settlements reflected on the ground surface are expected to be minimized if a thick crust is located at the top of the soil profile. After a careful review of soil improvement methods which are available and can be implemented at the site to achieve the goals set, the use of Rammed Aggregate Pier® (RAPs) is preferred.

Construction of RAP Impact Elements

Installation steps for these stone columns with displacement technique are summarized below:

(1) a closed ended mandrel with a diameter of 36 cm is pushed into the design depth using hydraulically induced static force assisted with vertical dynamic energy,
(2) the mandrel and hopper are filled with aggregate (typically graded 13 to 38 mm particle size),
(3) the ramming action is applied with 100 cm up / 67 cm down compaction efforts, during which vertical dynamic energy is also introduced.

The ramming action expands the diameter from 36 cm to 50 cm if 100 cm up and 67 cm down compaction procedure is chosen. The significant increase in lateral stress combined with the high density of the stone created by this installation process provides the unique strength and stiffness of the RAP system (Handy 2001, Wissmann et. al., 2001). Figure 7 presents the construction methodology of RAP Impact elements and a view from the field construction.

Figure 7. Construction methodology of RAP Impact elements and a view from the filed construction.

Design Approach

In order to achieve the design goals, 50 cm diameter stiff RAP elements reaching to 15 m length from the ground surface with 1.4 to 1.7 m square pattern, corresponding to area replacement ratios of 10 % and 7.0 %, respectively, were installed beneath the main structures of the waste water treatment facility. The closest spacing was used beneath the aeration tank mat with the highest base pressure of 110 kPa.

Foundation settlements were calculated using a two-step procedure: the compression of the zone of matrix soil reinforced by the piers (upper zone) is the first estimated and then the compression of the zone of soil that is located below the tip of the piers (lower zone) is computed, the sum of the two yielded the total settlement.

The compression of the RAP-reinforced zone beneath the mat is estimated by using composite constrained modulus of improved soil, $E_{comp.}$. The representative values of E_{comp} for the upper zone layers are computed by considering the area ratios of RAP and, properties of natural soil and piers. The composite constrained modulus for a RAP improved soil zone is computed using following relationship:

$$E_{comp} = E_{RAP} R_a + E_s (1 - R_a) \qquad (1)$$

where, E_{RAP} is the compression modulus of RAPs, E_s is the constrained modulus of matrix soil and R_a is the area replacement ratio. The area replacement ratio for a square pattern of RAPs, R_a, can be expressed in terms of the diameter and spacing of the piers as follows:

$$R_a = \frac{A_{RAP}}{(S_p)^2} \tag{2}$$

where, A_{RAP} is the area of the compacted piers (0.2 m^2 for diameter of 50 cm). Estimates of settlement in the lower zone materials, below the bottom of the pier bulbs, are computed using conventional geotechnical settlement analysis procedures.

Settle 3D analyses are carried out for the improved soil conditions, considering the presence of RAPs of 15 m length under the foundation base. Elastic settlements are calculated for the upper zone (i.e. in the improved upper 15 m) with an assumed RAP stiffness modulus value of 25 MN/m^3 and consolidation settlements of lower zone were calculated using properties of underlying layers.

Table 2. Soil parameters used in upper zone settlement analyses.

Material	γ (kN/m^3)	E_{RAP} (MPa)	E_{comp} (MPa)
RAP Zone-1	18.4	50	12
RAP Zone-2	18.4	100	32
RAP Zone-3	18.4	50	12

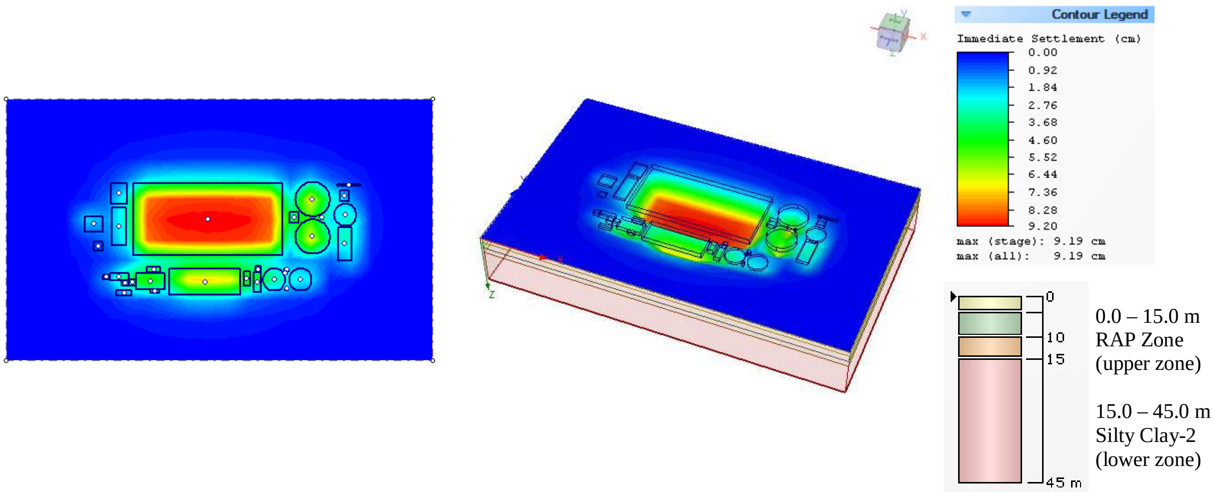


Figure 8. Upper zone settlement.

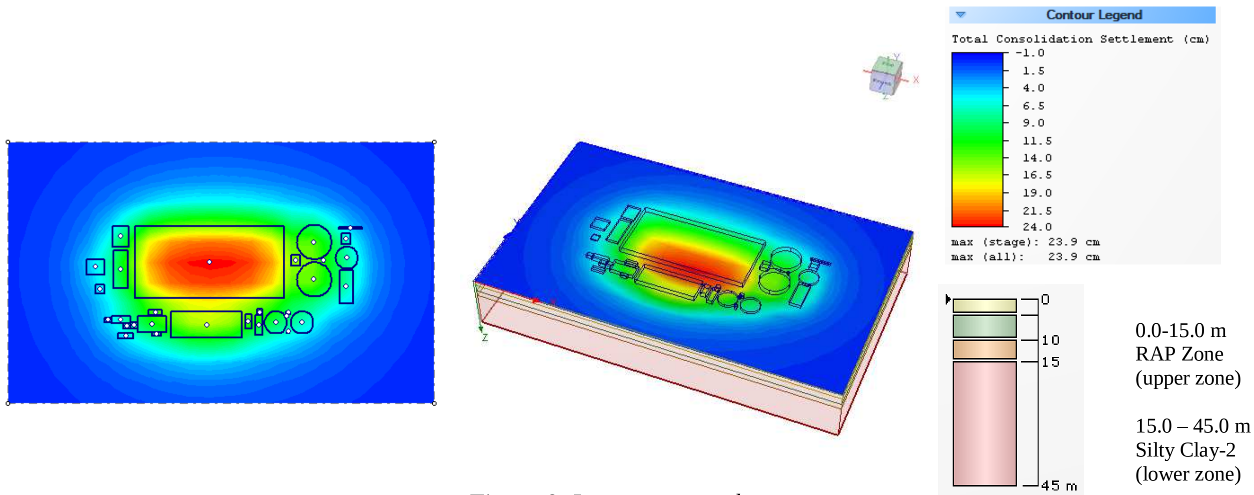


Figure 9. Lower zone settlement.

At the preliminary design stage, the immediate settlement of improved upper layers with RAP elements and the long term consolidation settlement of non-improved cohesive layers underlying RAP elements were computed to be in the range of 10 cm and 14 - 25 cm, respectively, indicating a considerable decrease in settlements compared to the settlements without soil improvement. The distribution of computed settlements shown in Figures 8 and 9 indicates a considerable decrease in differential settlements can be also expected with the planned soil improvement.

Modulus Load Test Procedures and Results

An assumed pier modulus value is selected using methods described in literature (Fox and Lawton, 1994) based on known pier properties and the properties of the surrounding soil, and then confirmed with site specific modulus tests. The modulus test set up is similar to a pile load test configuration and the test is performed in general accordance with ASTM D-1143. The tests are also used to observe how the RAP behaves in the soil matrix, by monitoring the deflection of tell-tales installed at the tip of the test piers. The modulus load test of RAPs may also incorporate tell-tales at different elevations within the pier (Brain et al., 2006). The tell-tale elements consist of a horizontal steel plate that is attached to two sleeved vertical bars extending to the top of the pier. When a RAP is equipped with a tell-tale reference plate, the deformation mode of the pier can be recognized from the shape of the tell-tale load settlement curve in comparison with the top of pier settlement. Typical modes of deformation for RAPs installed in soft soil include bulging and tip movement (Wissmann et. al., 2001).

In this project, 7 modulus load tests were performed on RAPs installed with lengths of 14 - 16 m to assess the load bearing capacity and stiffness response of individual RAPs. As the axial compressive load is directly applied on the pier, the magnitude of stress is controlled by a hydraulic jack with a calibrated manometer. The construction machine was used as a counter weight for the modulus load tests performed in this project. The vertical displacements of the piers were monitored using five comparators which were connected to the transverse beam and two of these comparators were placed on the tell-tale in order to measure the deformation at the bottom of piers. The distance of the bearing points of the transverse beam from the test pier was provided to be at 5 pier diameters. A concrete cap with a diameter of 60 cm was placed top of the pier in order to transfer the load. A schematic drawing of the test set up and the photos taken from the test area are shown in Figure 10.

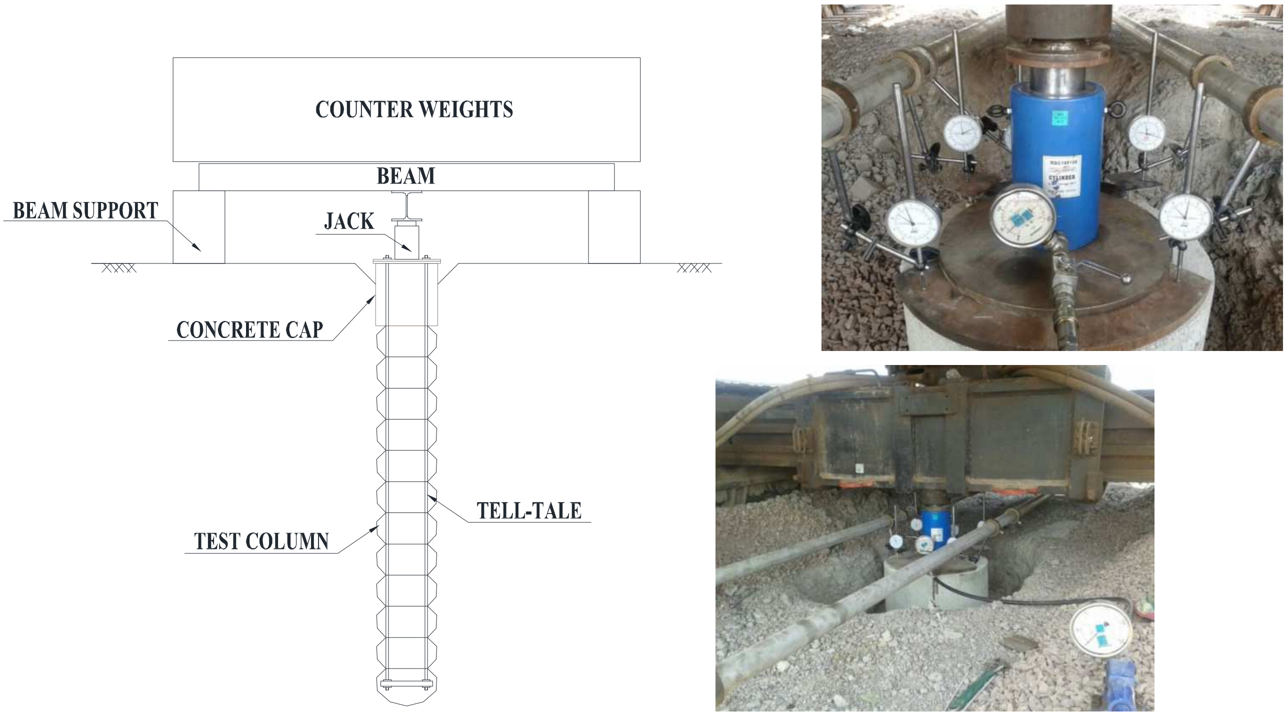


Figure 10. The test set-up and field photos.

Loading, starting with 5 % of service load is increased until the pier is tested up to 150 % of its service load. Then, an unloading procedure is followed. All load increments is held for a minimum of 15 minutes and until the rate of deflection reduces to 0.254 mm per hour or less, or for a maximum duration of 1 hour. Field load tests were performed by closely following the loading scheme summarized in Table 3**.**

Table 3. The loading scheme.

Load No	% Service Load	Load (ton)	Load No	% Service Load	Load (ton)
0	5	0.68	8	133	17.96
1	16	2.16	9	150	20.25
2	33	4.46	10	100	13.50
3	50	6.75	11	66	8.91
4	66	8.91	12	33	4.46
5	83	11.21	13	0	0.00
6	100	13.50	14	100	13.50
7	116	15.66	15	0	0.00

Two representative load vs. settlement curves are shown in Figure 11. RAPs undergoing primarily elastic deformation with little tell-tale movement indicate sufficient mobilization of shaft friction, without bulging, to resist the applied stress. The results indicated that a RAP stiffness of 35 - 70 MN/m^3 can be adopted and showed that the stiffness value used in the preliminary design was on the safe side.

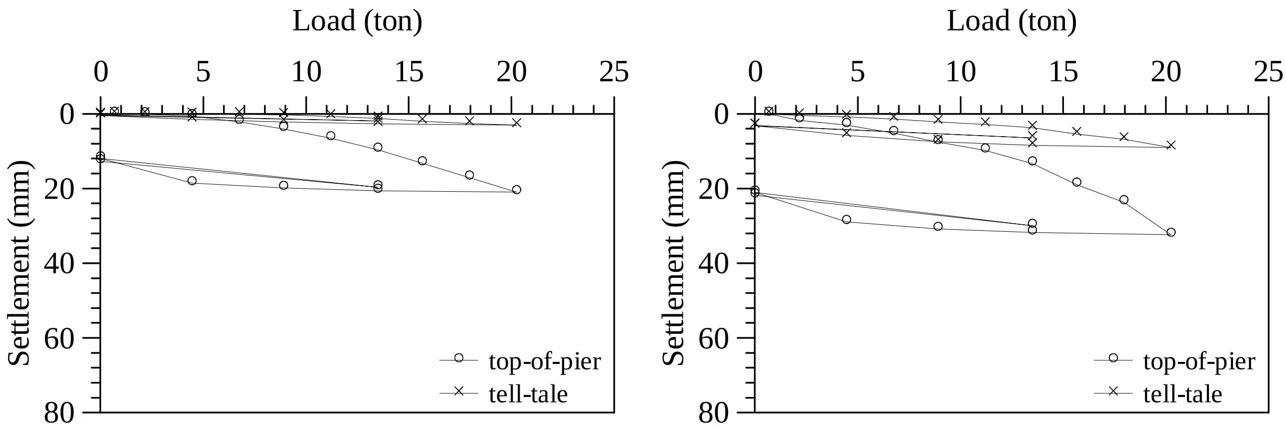


Figure 11. Modulus load tests, load-settlement graphs.

TEST EMBANKMENT AND ASSESSMENT OF MONITORING RESULTS

To verify the general design of soil improvement with the use of RAP elements and to determine the likelihood of differential settlements, a full scale area load test was performed by using a test embankment with 1V:1.5H side slopes, 6.1 m height and 36.8 m x 36.8 m base in plan. Test embankment was located at an area where soil conditions were relatively unfavorable as suggested by the boreholes. The area test contained 729 RAP elements of 15 m length and installed in a square grid with 1.4 m center to center spacing. The height and dimensions of the embankment were chosen to simulate the loading conditions expected in the project. A general view of test embankment is shown in Figure 12. Vertical deformations of the test embankment were monitored by geodetic measurements at 9 points (SP) for a period of 52 days. Also, a vibrating wire piezometer (transducers at 22.5 m and 27.5m depths) and four inclinometers installed to the depth of 40 m were located outside of the test embankment for measuring excess pore pressures and lateral deformations, respectively. Pore pressure transducers and inclinometers were monitored during the construction of test embankment, and then measurements were continued on weekly basis. The layout plan and the section of the load test embankment, and field photos are shown in Figures 13 - 14, respectively.

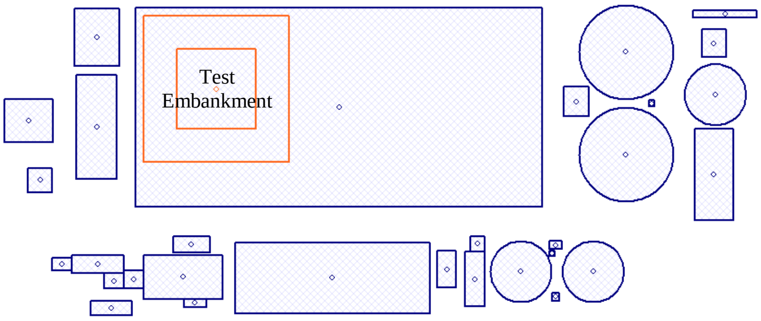


Figure 12. General view of test embankment.

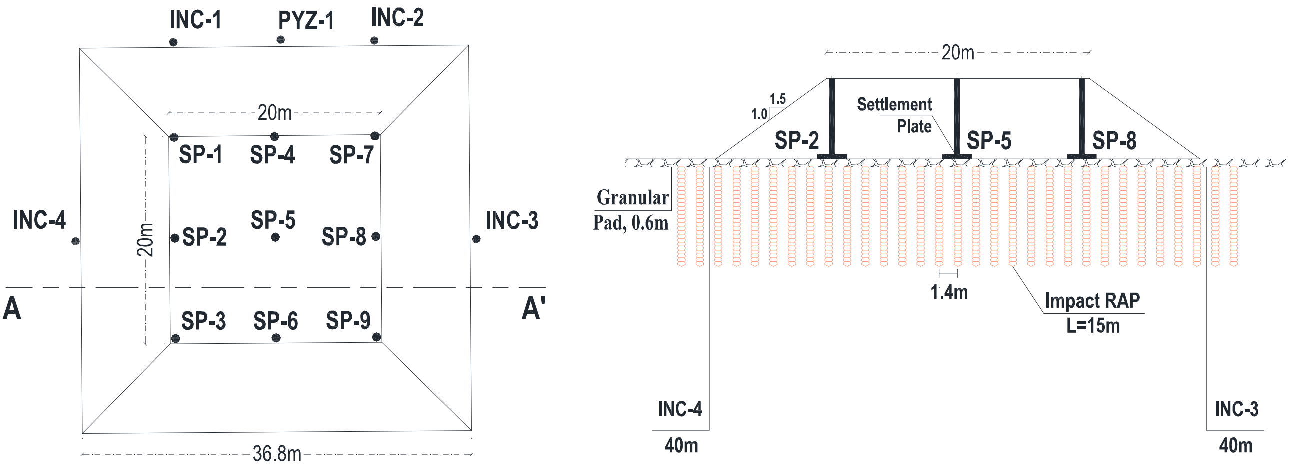


Figure 13. The layout plan and the section of test embankment.

Figure 14. The test embankment field photos.

Figure 15 shows settlement and pore water pressure - time response under the test embankment constructed on RAP improved ground. The geodetic measurements at 9 points (SP) for a period of 52 days indicate that the maximum settlements reached are around 28 cm or less, large portion of it occurring during construction. The piezometer measurements at depths of 22.5 m and 27.5 m indicate that the initial readings of the pore water pressures of 236 kPa and 273 kPa, respectively, increased minimally during the embankment placement (probably due to radial drainage provided by the closely spaced RAP elements) and then decreased at a low rate to their initial values. The four inclinometer readings installed around the test embankment showed the development of lateral displacement up to 90 mm during construction due to the rapid rate of fill placement (reaching maximum height in 7 days) and then movements slowed down after the full load of the embankment is imposed.

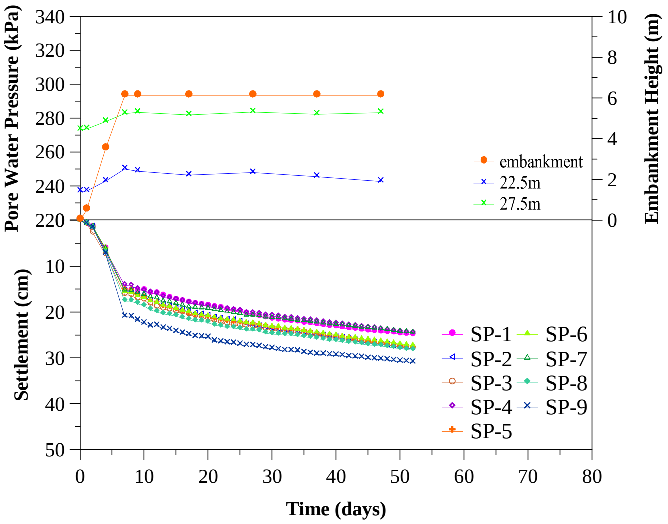


Figure 15. The measured results for the test embankment and instrumentetion.

It can be assumed that the immediate settlements are completed during the construction of the embankment to its full height. The lower zone consolidation settlements expected to occur in all structures under service loads are computed using Settle 3D software and utilizing the estimated soil parameters from back analysis of observed behavior at the field load test. The estimated soil input parameters are given in Table 4. The lower unimproved layer thickness is increased to 75 m and divided into two sublayers.

Table 4. Soil parameters use in Settle 3D assessments.

Material	Depth (m)	γ (kN/m^3)	$E_{comp.}$ (MPa)	C_c	C_r	e_0	OCR	c_v (m^2/day)
RAP Zone-1	0.0 – 4.0	18.4	12	-	-	-	-	-
RAP Zone-2	4.0 – 10	18.4	32	-	-	-	-	-
RAP Zone-3	10 – 15	18.4	12	-	-	-	-	-
Silty Clay-1/2	15 – 45	18.0	-	0.270	0.054	1.10	1	0.03
Silty Clay-2/2	45 – 75	18.0	-	0.125	0.025	0.85	2	0.03

In Figure 16, the distribution of the estimated settlements arising from the consolidation of the unimproved lower zone is shown. It is observed that in the range of 7- 45 cm settlements can be expected when all structures are constructed and fully loaded. When the results of piezometer measurements and Settle 3D analysis are evaluated together, continuing consolidation settlement is observed to extend down to 22.5 m depth. In addition, it is expected that the settlements which will take place under rigid mat foundations will be less than calculated since the flexible mat behavior is taken into account in the analyses.

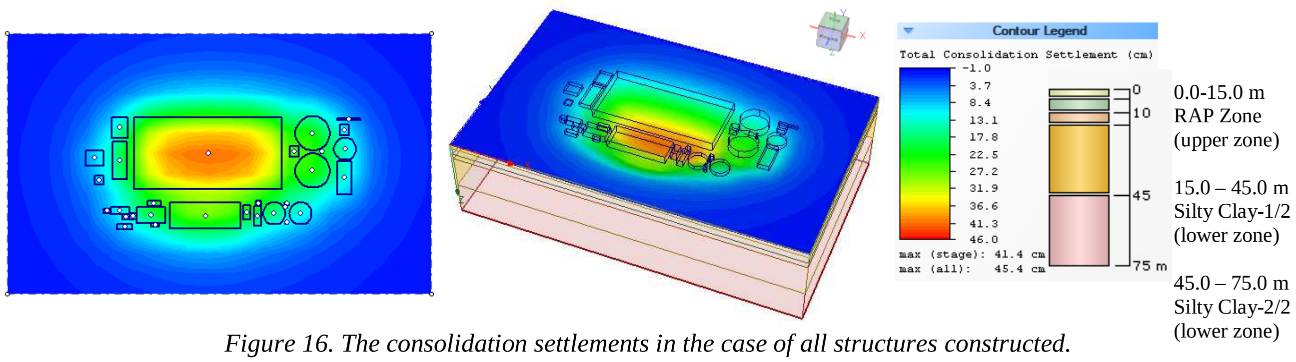


Figure 16. The consolidation settlements in the case of all structures constructed.

ASSESSMENT OF WATER TEST RESULTS

The construction of the waste water treatment facility structures supported by RAP elements was completed within approximately 6 months. Water loading tests are carried out to check the construction performance in such facilities. During the water loading tests within the scope of this project, the settlement measurements were taken for 302 days. The water height - time relationship applied in water loading tests is shown in Figure 17.

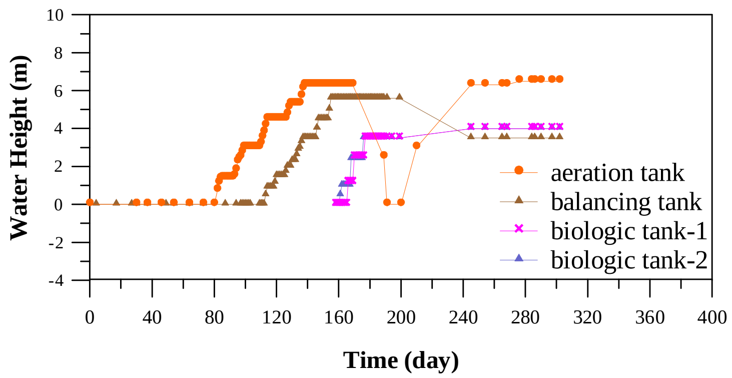


Figure 17. Water height – time relation applied in water loading test.

In Figure 18, the measurement points are shown for aeration tank, balancing tank and biologic tanks. In Figure 19 the estimated settlement - time response using Settle 3D software is shown together with the settlement readings taken during the water loadings tests.

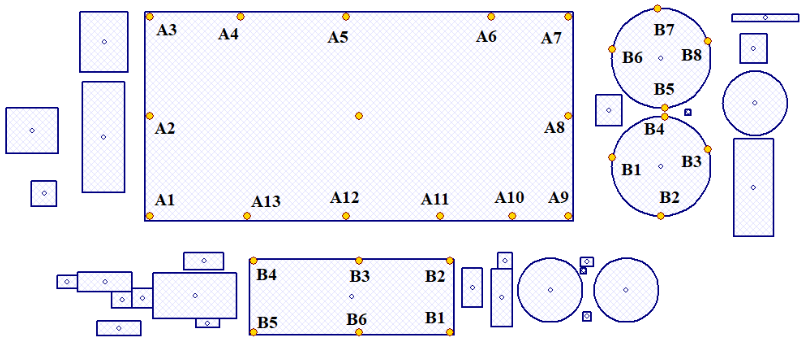


Figure 18. The measurement points for water loading test.

It is observed that under service loads of 96 - 108 kPa the measured settlements reached values varying between 14.5 cm and 24 cm, in 5.0 - 5.5 months after reaching the maximum of water level for the aeration tank and the balancing tank. It was observed that the settlements reached 16 cm and 16.5 cm within 4.0 months under pressures of about 75 kPa for the biologic tanks. The total settlements recorded in the monitoring period were about 50 % smaller than those predicted by 3D settlement calculations.

It was also observed from settlement-time curves that the settlements under the aeration tank and balancing tank was almost completed during the monitoring period, and about 70 % of the final settlements were reached under the biological tanks. It is thought that the calculated settlement amounts are higher than the field results due to vagueness in the thickness of the compressible layer which could not be precisely determined during the drilling operations and the compressible units are assumed to continue down to depths of 75 m in the analysis. The distribution of measured final settlements on the plan is shown in Figure 20 and it is seen that the differential settlements are controlled to remain between 0.015 % and 0.25 % in accordance with the limiting values adopted as among the project targets.

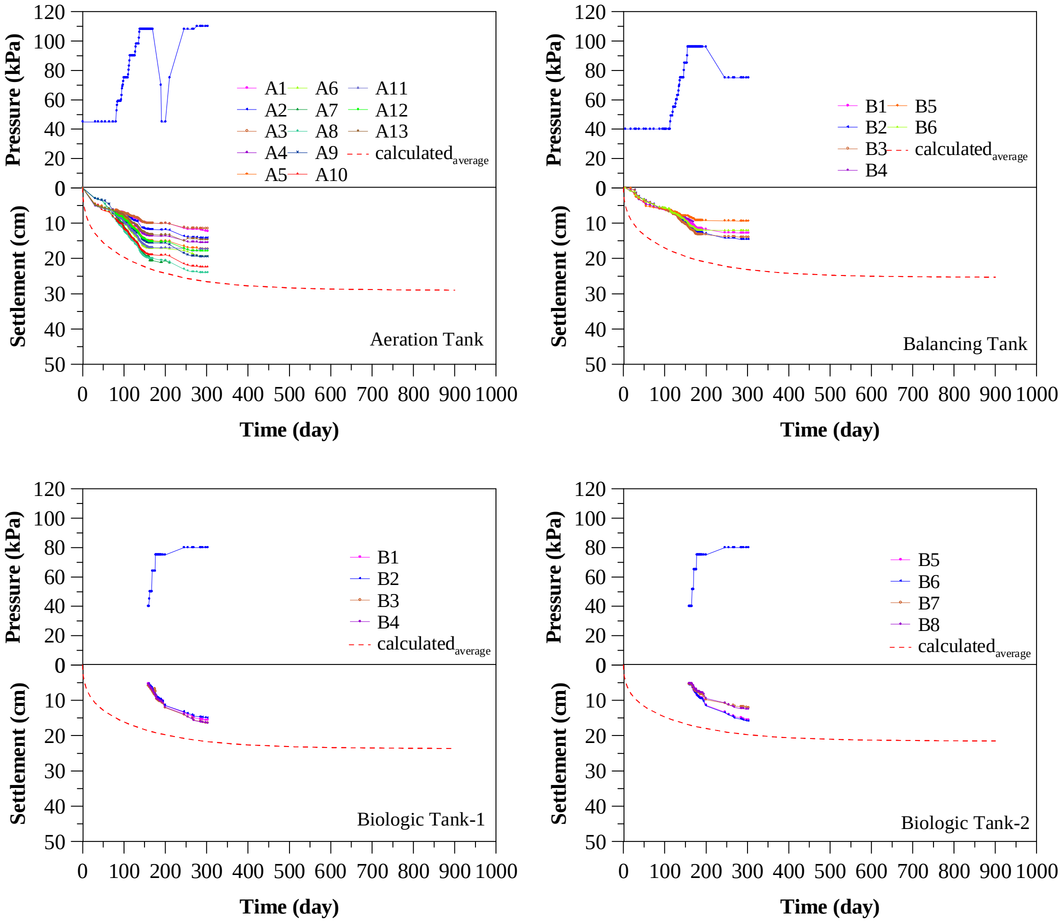


Figure 19. 3D model and field data responses: settlement vs time curves for aeration, balancing and biologic tanks.

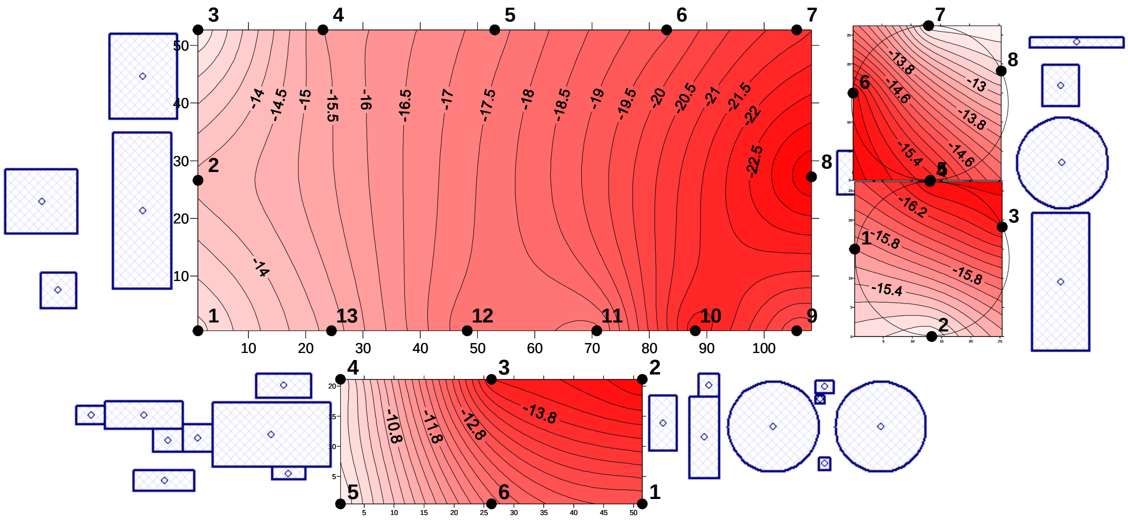


Figure 20. The distribution of measured final settlements on the plan.

CONCLUSIONS

This paper presents the results of settlement monitoring data at a waste water treatment plant constructed on a soft clay site reinforced with Rammed Aggregate Pier® elements (RAPs). Design goals in the implemented soil improvement scheme were to reduce total and differential settlements, eliminate liquefaction induced strength and rigidity losses of bearing layers under structures and the detrimental effects of the differential settlements reflected on the ground surface during an earthquake by forming a thick strong crust on top of the soil profile. After the careful review of soil improvement methods available and which can be implemented at the site to achieve the goals set, the use of Rammed Aggregate Pier® (RAPs) is preferred.

In order to verify design assumptions, 7 modulus load tests were performed on 14 - 16 m long RAPs installed at the project site, and 35 - 70 MN/m^3 column stiffness values measured showed that the stiffness value used in the preliminary design (25 MN/m^3) was on the safe side.

A full scale area load test was performed by a test embankment of 6.1 m height and 36.8 m x 36.8 m base dimensions, at an area where soil conditions were relatively more unfavorable and foundation layers were improved with RAP elements of 15 m length, installed in a square grid with 1.4 m spacing. Settlements are measured at 9 surface points, also a vibrating wire piezometer (transducers at 22.5 m and 27.5 m depths) and four inclinometers installed to the depth of 40 m, enabled monitoring of pore pressures and lateral soil movements for a period of 52 days. The immediate settlements are observed to be take place during the construction of the embankment, whereas the lower zone consolidation settlements observed to continue at a decreasing rate. Settlements expected to occur under all structures when service loads are imposed are calculated utilizing the estimated soil parameters from back analysis of observed behavior at the field load test.

The construction of the waste water treatment plant structures on soil layers improved with RAP elements was completed in about 6 months. During the water loading tests are carried out to check the construction performance, settlement measurements were taken for 302 days. It is observed that under service loads of 96 - 108 kPa, settlements reached values varying between 14.5 cm and 24 cm, in 5.0 - 5.5 months after the maximum of water level is attained at the aeration tank and the balancing tank, and rate of settlements indicated almost final primary consolidation stage is reached. At the biologic tanks measured settlements under 75 kPa loading reached to 16.5 cm in 4 months after the maximum of water level is attained, and it is estimated that 70 % of the final settlements were reached.

The total settlements recorded in the monitoring period were about 50 % smaller than those predicted by 3D settlement calculations. It is believed that the main reasons for calculated settlements being higher than the field results are the difference in assumed thickness of the compressible layer and the flexiable foundation assumption adopted in the calculations. The distribution of measured settlements on the plan has shown the differential settlements are considerably reduced by the implemented soil improvement and controlled to remain between 0.015 % and 0.25 % in accordance with the limiting values adopted as among the project targets.

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