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GROUND IMPROVEMENT BY STONE COLUMNS AND SURCHARGE AT A TANK SITE

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ABSTRACT

The ground improvement performed at the site of two 190-ft (57.9 m) diameter, 40-ft (12.2 m) high, 8 million-gallon (30,300 m^3), circular steel water storage tanks consisted of installation of stone columns to mitigate liquefaction and lateral spreading potential and a surcharge program to reduce post-construction settlements. Settlement during the surcharge program ranged between 9 and 15 in. (225 and 375 mm) and post-construction settlement during the hydrotest was about 1.2 inches (305 mm).

INTRODUCTION

Construction of two 190-ft (57.9-m) diameter, 40-ft (12.2-m) high, 8 million-gallon (30,300 m^3), circular steel water storage tanks side-by-side on a 4-acre (1.62-hectare) site was proposed. A geotechnical investigation by another firm originally recommended that tank foundations could be supported on 5 ft (1.52 m) of recompacted onsite soils. This recommendation was apparently made assuming settlement of the tank was controlled by the load on the ring wall footing and 5 ft (1.52 m) of removal and recompaction would be adequate. The fact that the primary loading is the weight of the water was not considered. A review of the soil conditions by the authors disclosed significant geotechnical problems with the site including potential for large settlements, liquefaction, and lateral spreading.

A subsequent investigation, which included drilling soil borings, Cone Penetration Tests (CPTs), and laboratory tests, confirmed that the site was underlain by highly compressible and potentially liquefiable soils. Also present were a 10-ft (3-m) deep channel and a detention pond in close proximity to the tank pad. The authors estimated that static settlements of more than 12 in. (305 mm) could occur under the tank loading. Also the potential for liquefaction and lateral spreading and resulting tank failure was high due to the presence of the adjacent channel and detention pond.

In lieu of costly pile foundations, the authors proposed an economical site improvement plan which included installation of stone columns to mitigate liquefaction and lateral spreading potential and a soil surcharge (preloading) to decrease post-construction settlement of the tanks. The measured settlements during the surcharge ranged from 9 to 15 inches (228 to 375 mm). The stone columns densified loose granular soils, thereby mitigating liquefaction potential, and increased the average strength of the soft clays to improve the bearing capacity and mitigate lateral spreading potential. The tanks were successfully constructed and hydrotested. Measurements during hydrotest showed total settlements of about 1.2 in. (305 mm) and differential settlements of 0.5 inch (12.6 mm).

SITE & SUBSURFACE CONDITIONS

Surface Conditions

Surface site conditions relevant to the tank design are summarized below:

- The lot is roughly rectangular in shape, measuring about 560 ft (171 m) in the east-west direction and 330 ft (101 m) in the north-south direction. Site layout is shown in Fig. 1. The tank site has grades ranging between El. 33 to El. 35 feet (10.1 to 10.7 m).
- A new detention pond with a depth ranging from 6 to 10 ft (1.83 to 3 m) and 3:1 (horizontal to vertical) side slopes was planned to be constructed about 25 ft (7.6 m) south of the reservoirs.
- The site is bounded on the north, and along the northwest corner, by a 10-ft (3-m) deep concrete-lined drainage channel with vertical side retaining walls.

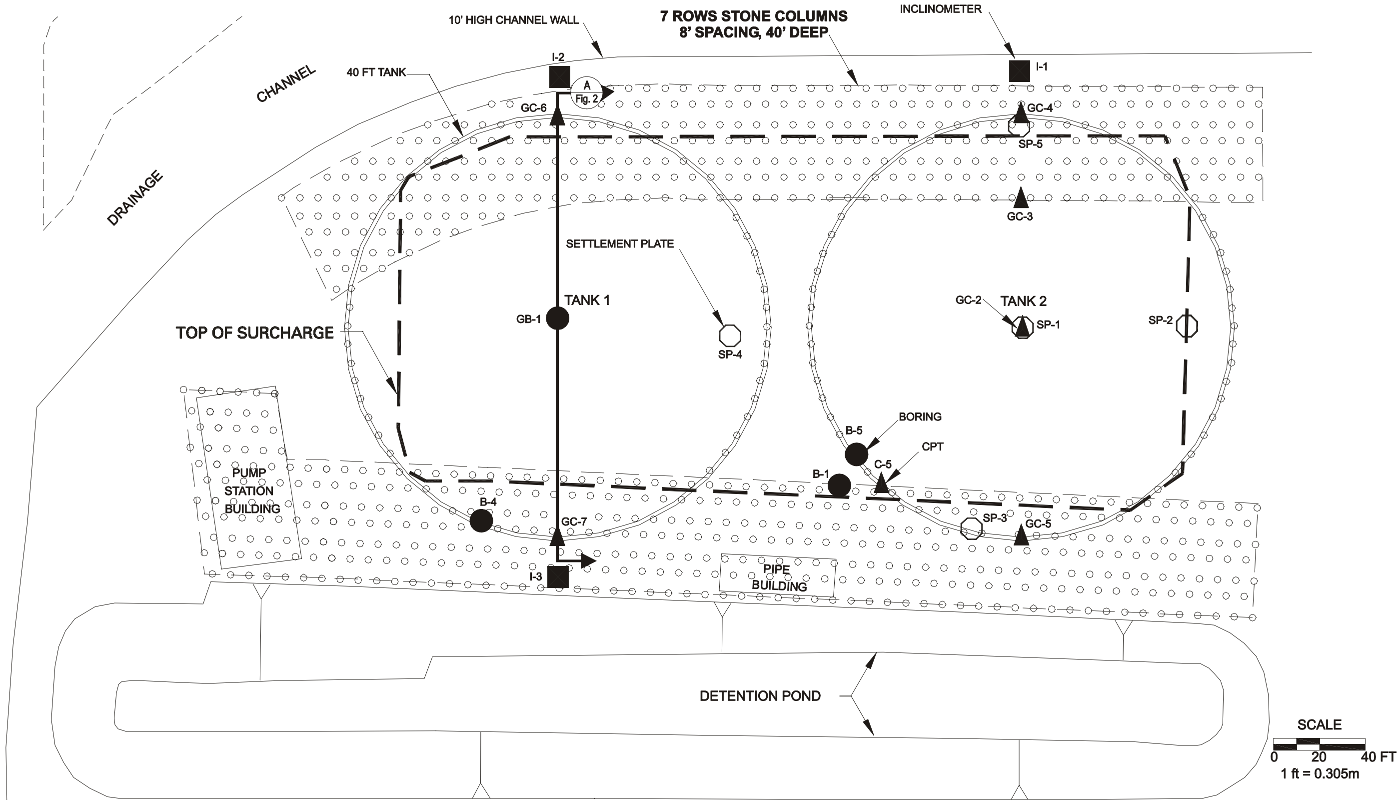

Fig. 1. Site Layout

Table 1. Generalized Soil Profile

Layer Number	Depth (ft)	Elevation (ft)	Soil Type	Undrained Strength[1] (ksf)	CPT Tip Resistance (tsf)	Compressibility[2,3,4]
1	0-9	34 to 25	Stiff Clay / Silt (CL/ML)	2.0	N/A	E=600 ksf
2	9-20	25 to 14	Soft Highly Plastic Clay (CL/CH)	0.6 to 1.2	N/A	$C_r/(1+e_0)$=.02 $C_c/(1+e_0)$=.17
3	20-25	14 to 9	Loose Sand (SM)	N/A	60	E=360 ksf
4	25-40	9- to -6	Firm to Stiff Clay / Silt (CL-ML)	>1.25 (avg. 2.25)	N/A	$C_r/(1+e_0)$=.007 $C_c/(1+e_0)$=.07
5	40-52	-6 to -18	Medium Dense Sand / Silt (SM/ML)	N/A	75	E=450 ksf
6	52-60	-18 to -26	Dense Sand (SP-SM)	N/A	225	E=1350 ksf
7	60-63	-26 to -29	Stiff Clay / Silt (CL-ML)	3.5	N/A	E=700 ksf
8	63-76	-29 to -42	Dense Sand (SP-SM)	N/A	200	E=1200 ksf
9	76-90	-42 to -56	Very Stiff Clay / Silt (CL-ML)	5.0	N/A	E=1000 ksf
10	90-96	-56 to -62	Very Dense Sand (SP-SM)	N/A	325	E=2000 ksf

NOTES:
1. Undrained strength estimated from CPT (N_k=15).
2. Young's Modulus (E) for stiff clay estimated from correlations with undrained shear strength.
3. Young's Modulus (E) for sands estimated from correlations with CPT tip resistance.
4. Compressibility for soft to firm clay/silt based on interpretation of consolidation test results.
5. Groundwater at a depth of 10 to 16 ft (3 to 4.9 m).
6. 1 ft = 0.308 m, 1 ksf = 47.8 kN/m^2, 1 tsf = 95.6 kN/m^2.

Subsurface Conditions

Based on data from the borings and CPTs, the soil profile is relatively uniform. Beneath a cap of stiff clay/silt man-made fill soils, deep alluvial sediments underlie the site. Above El. –18 ft (5.5 m), the alluvial sediments consist primarily of compressible soft to stiff clay/ silt layers, with isolated zones of loose to medium dense sands. Below El. –18 ft (5.5 m), the sediments consist of dense to very dense sands and stiff clays. We developed a generalized soil profile including strength and compressibility parameters shown in Table 1. A generalized cross-section illustrating the soil profile is shown in Fig. 2. The groundwater was present at depths ranging from 10 to 16 ft (3 to 4.9 m) below existing grades.

Seismic Conditions

The site is located in a seismically active area of Southern California. Ground shaking due to nearby and distant earthquakes is anticipated during the life of the reservoirs. The closest active major fault to the site is the Newport-Inglewood Fault located about 4 miles (6.5 km) from the project site. This fault is a strike-slip fault with a maximum credible magnitude of 6.9.

The largest maximum credible ground acceleration computed using deterministic methods and mean value of three attenuation relationships for the site was 0.43g. Probabilistic analyses indicated the following maximum ground accelerations:

	Acceleration, g	
Probability of Exceedance	10%	50%
50-yr design life	0.30	0.15
100-yr design life	0.36	0.20

Ground acceleration associated with 10% probability of exceedance in 50 years was selected for design. A peak ground acceleration of 0.36 g, including 20% increase for near-field effects, was used for liquefaction analyses and tank design.

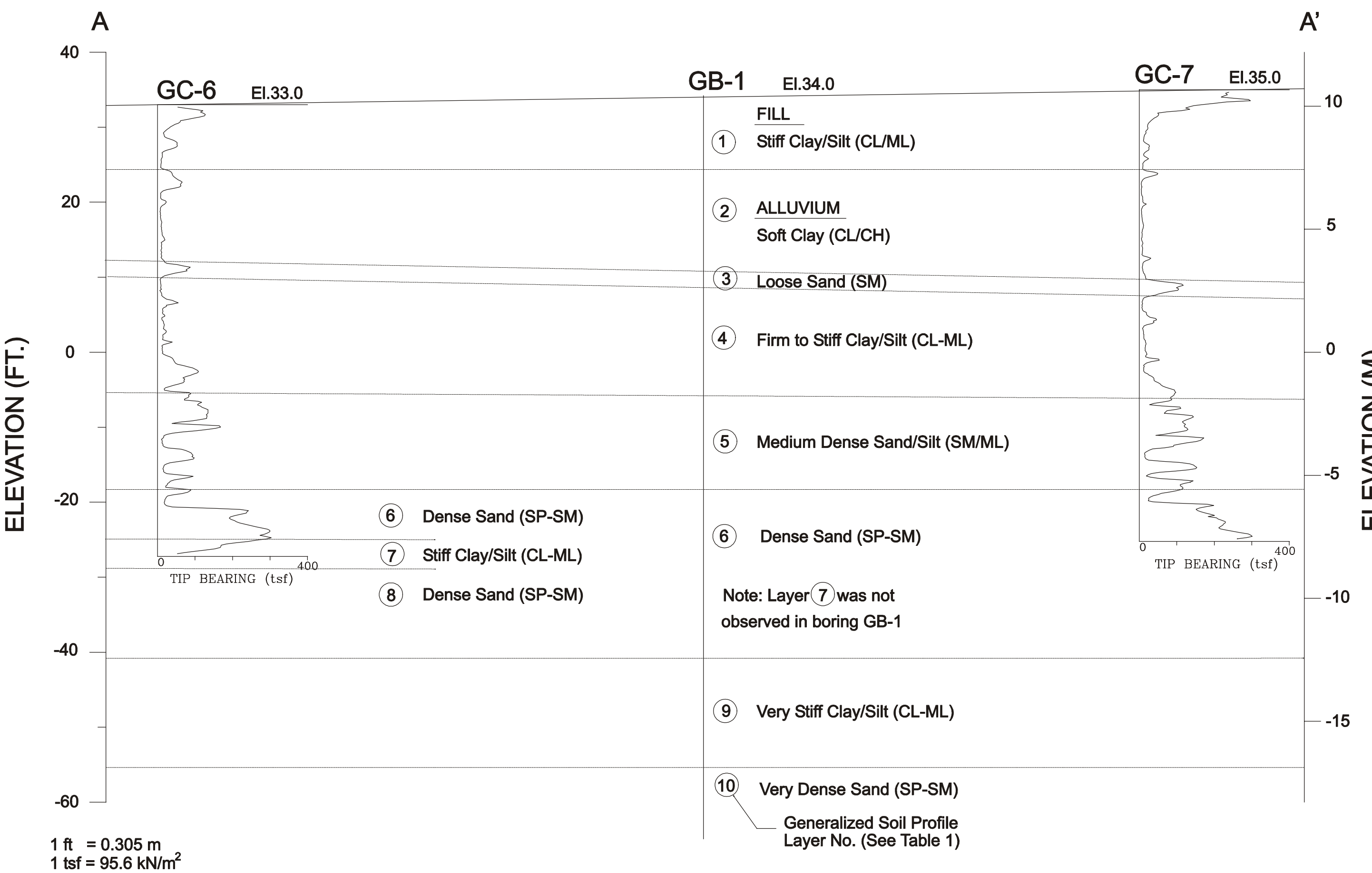

Fig. 2. Generalized Soil Profile

Liquefaction and Lateral Spreading

Liquefaction refers to loss of strength in a saturated granular soil due to buildup of pore water pressure during cyclic loading. When the pore water pressure becomes equal to the weight of the overlying soil, the soil is temporarily transformed into viscous fluid that is weaker than the non-liquefied material. For liquefaction to occur, three ingredients are required:

1. Liquefaction susceptible soils (loose to medium dense sand/silt)
2. Groundwater
3. Strong shaking, such as an earthquake

Isolated zones of loose to medium dense sands below the groundwater table are present at the site, and could liquefy during the design earthquake. We used equivalent SPT blow counts from CPT GC-2 to determine liquefaction potential (Youd, T. L., et al., 2001) and to estimate the magnitude of associated ground settlements (Tokimatsu and Seed, 1987) that could occur. We estimated that liquefaction-induced settlement would be on the order of 1 in. (25 mm) when the site is subjected to an earthquake producing 0.36 g acceleration at the site.

Besides settlement, liquefaction could result in reduced lateral stability due to the potential for lateral spreading, or sliding along the liquefied layer or weak clay layers.

NEED FOR GROUND IMPROVEMENT

Bearing Capacity

The reservoirs are underlain by about 5 ft (1.5 m) of compacted fill, which in turn is underlain by about 15 ft (4.6 m) of soft to medium stiff clay. Due to the relatively thin fill as compared to the large size of tank, the stronger fill soils have little effect on the overall bearing capacity of the tank. Based on CPT correlation, the minimum undrained shear strength in this layer is 600 psf (28.7 kPa), and the average is about 1,000 psf (48 kPa). Based on these shear strengths, the factor of safety against a bearing failure is 1.38 and 2.31 for minimum and average shear strength, respectively, which is inadequate.

Settlement

Using the compressibility parameters in Table 1, we calculated settlements in the range of 8 to 10 in. (200 to 250 mm) under a full tank. These settlements were not acceptable and ground improvement in the form of preloading was recommended to limit post-construction differential settlements to less than 1 in. (25 mm).

Lateral Stability

The vertical 10-ft (3-m) high wall of the drainage channel is located about 25 ft (7.6 m) north of the two reservoirs (Fig. 1). Tank foundations were proposed at roughly 10 ft (3 m) above channel invert elevation. In addition, a detention pond was proposed about 25 ft (7.6 m) south of the two reservoirs. The detention pond was proposed with 3:1 (horizontal to vertical) side slopes and the bottom of the pond was up to 10-ft (3m) below the tank foundations. The presence of these low-lying areas results in reduced lateral stability compared to tanks on level ground. Results of stability analyses performed by PCSTABL/5M are shown in Fig. 3. For seismic stability, we used a pseudo-static coefficient of 0.15, and we reduced the shear strength of the soft to firm clay to 80% of the static shear strength. The calculated factors of safety for static and seismic conditions are 1.3 and 0.9, respectively, and are lower than the normally used values of 1.5 and 1.1, respectively. Since the calculated factor of safety for pseudo-static analysis was less than 1.1, we estimated that potential for large lateral movements using simplified Newmark-type analysis (Blake et. al., 2002) was present during the design earthquake.

Foundation Options

Based on the preceding analyses, it was concluded that from bearing capacity, settlement, and lateral stability considerations, the tanks could not be constructed with adequate factors of safety without ground improvement or use of deep foundations.

Foundation options included supporting the tanks on pile foundations or performing ground improvements which included reducing post construction settlement by preload and mitigating liquefaction and lateral spreading by use of stone columns.

The pile foundations could provide adequate factors of safety for settlement and bearing capacity. However, they would need to be designed for downdrag loads due to liquefaction settlement and could be damaged by soil movements due to lateral spreading. The cost of driven pile foundations and a pile cap was significantly higher than the cost of preloading and surcharge. The stone columns had an added advantage of producing less noise than driven piles in a developed urban area.

Preload was used to decrease post-construction settlements of the tank to tolerable values, and to improve shear strength of soft clays for improved bearing capacity. Stone columns were used to densify loose granular soils and mitigate liquefaction potential, improve lateral stability by reducing liquefaction potential, and improve bearing capacity by reinforcing the weak upper clay soils with stronger gravel columns.

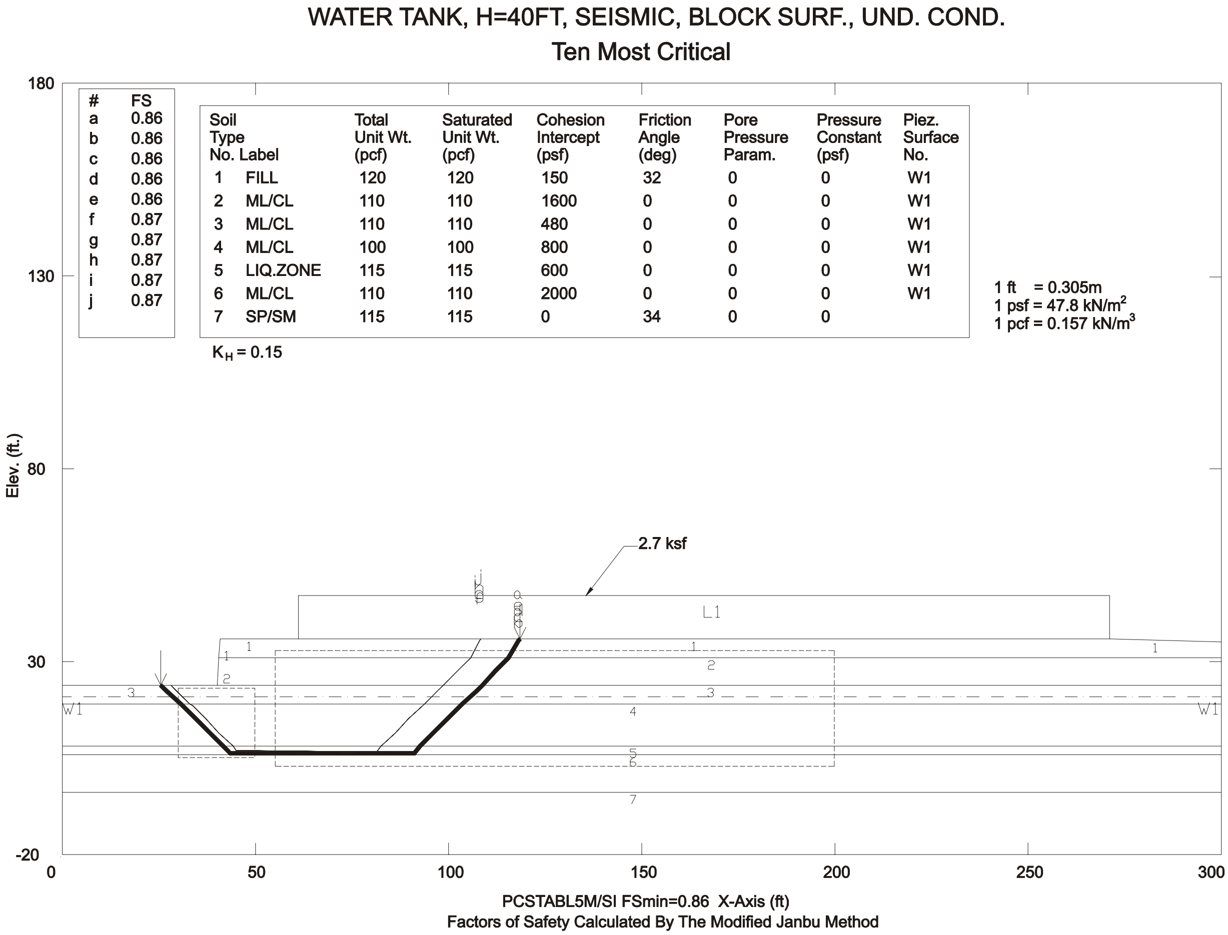

Fig. 3. Stability Analyses

Stone columns alone were not adequate to reduce post-construction settlements to the required differential settlement criteria. Preloading would not improve the shear strength of clays or reduce liquefaction potential sufficiently to mitigate the lateral spreading potential.

Based on this comparison, we selected the ground improvement option consisting of a combination of preload and stone columns over pile foundations.

DESIGN AND CONSTRUCTION OF GROUND IMPROVEMENTS

The height of the preload was selected to produce a total load greater than the loading from the reservoirs, or about 23-ft high (7-m) preload for the 40-ft (12.2-m) high reservoir. We estimated that a settlement of about 12 to 15 in. (300 to 375 mm) could occur under a 23-ft high (7-m) surcharge in about 3 to 6 months. With this preload, the post-construction total settlements could be limited to 1.5 inches (375 mm). The calculated bearing capacity using improved shear strength was greater than 3 for average conditions and greater than 2 for minimum shear strength.

We considered a strip of stone columns 50-ft (15.2-m) wide installed along the channel wall and along the top of detention pond slope. Stability calculations indicated that the stone columns constructed at 8-ft (2.4-m) triangular spacing would improve the static and seismic factors of safety for lateral stability to 1.5 and 1.2, respectively. The stone columns were not necessary under the entire tank area. The stone columns were constructed under the pump station building and under the ring walls of the two tanks.

Instrumentation

Three inclinometers were established along the north and south sides of the tanks near the channel and detention pond to measure lateral movements due to stone column installation and surcharge. Four survey points were set on the top of the channel wall to monitor both settlement and lateral movement of the channel wall. Five settlement plates, SP-1 through SP-5, were installed before placement of the surcharge to monitor settlement. The locations of these instruments are shown in Fig. 1.

Construction Sequence

The following construction sequence was used to perform the ground improvements and construct the tanks:

- The site was cleared, grubbed, and excavated to 5 ft (1.52 m) below the finished grade of El. 35 ft (10.7 m). Compacted fill was placed to 1 ft (0.3 m) above the finished grade.
- Inclinometers were installed at the locations shown in Fig. 1.
- Stone columns were installed along the northern and southern tank boundaries, adjacent to the drainage channel and detention pond. To minimize lateral movement and damage to the channel wall, the closest stone columns were kept at a minimum distance of 12 ft (4 m) from the wall. A total of 908 stone columns were installed in 54 days.
- Installed settlement plates SP-1 through SP-5.
- After completion of the stone columns, import fill was placed to preload the site to the boundaries shown in Fig. 1. The placement of about 80,000 cubic yards (61,210 cubic meters) of fill was completed from April 16, 2001 through May 30, 2001 a period of about 45 days.
- Settlement plates, survey points on the channel wall and inclinometers were read regularly as the fill was placed to verify that no unexpected lateral movement of the channel wall was occurring.
- The preload was completed to a top elevation of El. +58 feet. Monitoring of settlement plates and inclinometers was performed on a weekly basis.
- The preload was left in place for a period of about 100 days from the completion of surcharge or about 5 months including the time required to place the fill.
- The preload was removed and the surface was scarified and the upper 6 in. (150 m) of soils were recompacted to 95% relative compaction.
- The reservoirs were hydrotested by filling them with water and surveying eight points on the ringwall.
- The tanks were put into operation.

The two tanks, pump station, and other facilities are shown in Photo 1.

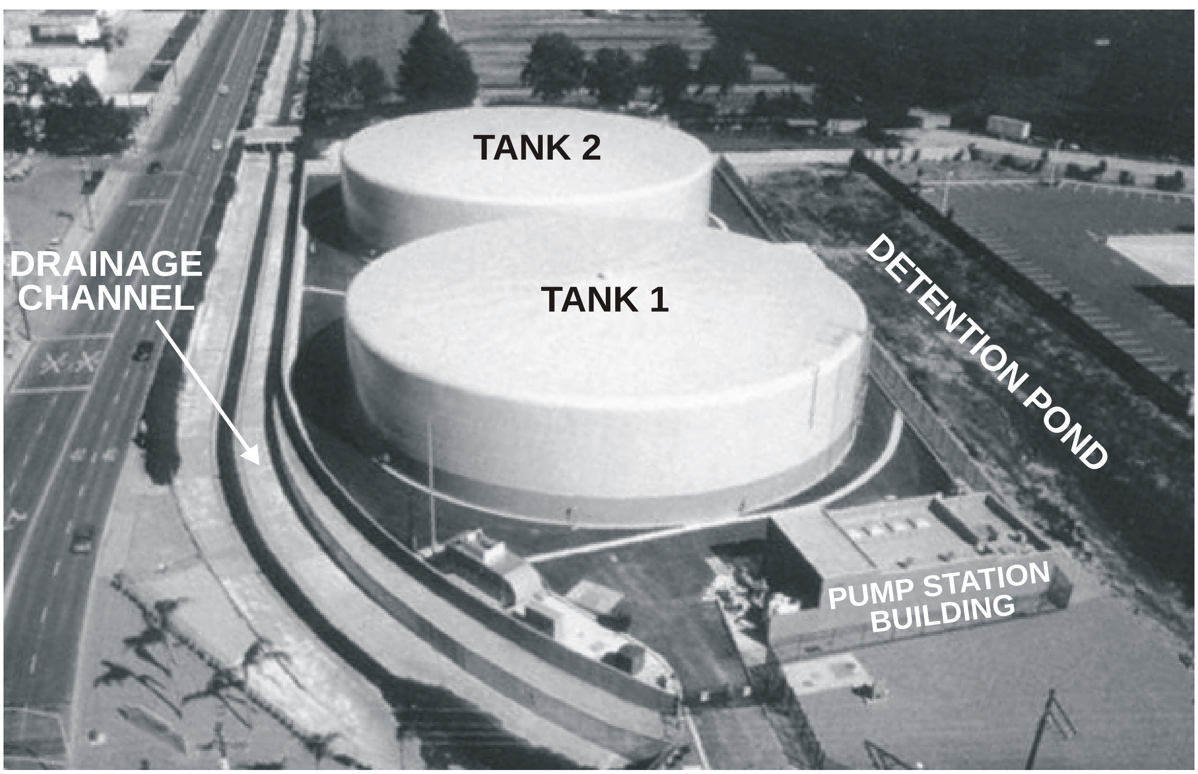

Photo 1. Aerial View of Tanks

MONITORING RESULTS

The monitoring consisted of three items:

- Settlement at five locations in the surcharge area plates SP-1 through SP-5,
- Inclinometer readings, and
- Vertical and horizontal movements of selected points on the channel wall closest to the property line.

Settlement

The settlement data from plates SP-1 through SP-5 is presented in Fig. 4. The following observations can be made from a review of Fig. 4.

- The measured settlement ranged between 9.1 and 14.6 inches (231 and 371 mm). The smallest settlement at 9.1 in. (231 mm) for plate SP-5 is for a plate on the slope. Plate SP-4 was located in an area where previous fill had been stockpiled and thus, has less settlement. The settlement at the other three plates ranges between 12.2 and 14.6 inches (310 and 371 mm). These values compare favorably with the predicted settlements under the surcharge.
- The data in Fig. 4 indicates that the settlement had leveled off in about 100 days after the completion of the surcharge or about 5 months after the start of the surcharge. Depending on the rate at which the surcharge was placed, 90% of the settlement was completed in the first 30 to 45 days after the completion of the surcharge.

Lateral Channel Wall Movement

A total of six test stone columns were installed adjacent to the Westminster Channel retaining wall near inclinometers I-1 and I-2. The centers of these columns ranged between about 12 ft to 32 ft (3.66 to 9.76 m) from the channel wall. The columns were installed, and observations of movement at the channel wall were made by surveying points on the wall before and after the stone column installation. An inclinometer reading was taken before and after installation of the columns.

Three stone columns installed at a distance of 12 to 13 ft (3.66 to 3.96 m) showed an estimated lateral wall movement of up to ¼ inch (6 mm). The inclinometer casing was too close to the stone columns to provide any reliable reading.

Three stone columns were installed at distances of 16, 24, and 32 ft (4.88, 7.32, and 9.76 m) from the wall. The two columns at distances of 24 and 32 ft (7.32 and 9.76 m) from the wall were installed with the normal procedure and were vibro-compacted up to El. 32 feet (9.76 m). The crack monitors, survey points, and inclinometer readings showed no measurable movements in the channel wall. For the stone column at a distance of 16 ft (4.88 m), we recommended installation without compaction in the upper 10 feet (3 m). No movement was observed visually, and survey points and crack monitors indicated no significant movement of the wall. The inclinometer located at a distance of about 6 ft from the stone column showed more than 1 in. (25 mm) of movement in the compaction zone below El. 22 ft (6.71 m), but no significant movement in the zone above El. 22 feet (6.71 m).

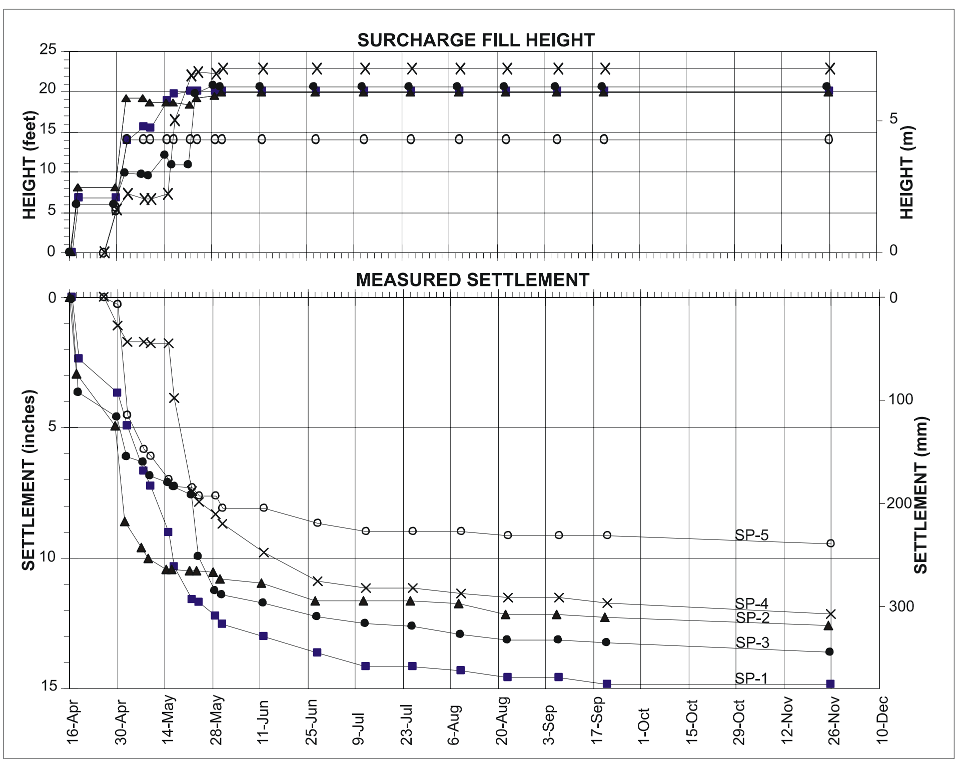

Fig. 4. Surcharge Settlement

Based on these observations, stone columns located within 16 ft (4.88 m) or less have the potential to generate undesirable movement and pressures on the wall. In order to reduce the movement of the wall, we recommended that stone columns be located at a minimum distance of 16 ft (4.88 m) from the wall and the stone columns at the minimum distance be installed without vibro-compaction in the uppermost 10 ft (3 m) or above El. 22 feet (6.7 m).

HYDROTEST RESULTS

Settlement for each tank was measured on 8 points set on the ring wall. The measured settlement on Tank 1 ranged between 0.72 in. (18 mm) and 1.2 in. (304 mm) under a water height of 40 feet (12.2 m). The settlement stabilized quickly and no additional settlement was measured after one day. The settlement for Tank 2 was measured with water height of 29 ft (8.84 m) and ranged between 0.12 and 0.96 inches (3 and 25 mm). Settlement under full water height was not measured but the maximum settlement was extrapolated to about 1.3 inches (33 mm). The maximum differential settlement for Tank 1 was about 0.5 in. (12.7 mm) between Points 5 and 6 located about 74 ft (22.6 m) apart along the circumference of the tank.

CONCLUSIONS

The following conclusions can be derived from this case history.

1. Inexperienced geotechnical engineers can grossly misinterpret subsurface conditions and potential for settlement under large loaded areas such as tanks.
2. Surcharging the site with loading equal to or more than the structure loads can effectively reduce the post-surcharge settlements to acceptable limits.
3. The measured settlements under the surcharge agreed well with the predicted settlements. The actual time required to obtain greater than 90% consolidation was near the lower range of the estimated time.
4. Stone columns can reduce liquefaction and lateral spreading potential and can improve the soil bearing capacity.
5. Stone columns installed at a distances of less than 12 ft (3.66 m) can cause high lateral pressures and displacement of adjacent structures.
6. Stone columns can be installed at distances of 16 ft 4.87 m) or more without damaging existing structures or utilities. The damage to structures can be reduced by elimination of vibro-compaction in the depth range of the adjacent structures.
7. Marginal sites can be used to support large tanks by ground improvement.

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