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S.M. Gazioglu
J.L. Withiam

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Evaluation of a Differentially Settled Tank

S. M. Gazioolu and J. L. Witham
Senior Project Engineer and Staff Consultant, D'Appolonia Consulting Engineers, Inc.

SYNOPSIS The paper discusses studies undertaken to identify the cause(s) for differential settlements experienced by a large floating roof tank. The studies included an evaluation of existing subsurface and tank performance data, additional subsurface exploration and laboratory test programs, a monitoring program during the restricted use of the tank and recommended remedial measures to allow full use of the tank. It is concluded that the affected portion of the tank was sited over a thicker and more compressible soil layer than the remaining portions and that releveling by mud-jacking would allow unrestricted future use of the tank.

INTRODUCTION

In 1978, the Department of Energy (DOE) began the construction of the St. James Terminal facility which included six tanks having a total capacity of two million barrels. These tanks were to serve as surge tanks for transfer of crude oil to and from the Strategic Petroleum Reserve (SPR) sites at Weeks Island and Bayou Choctaw where crude oil is stored in underground caverns in salt domes.

Prior to tank construction, a site investigation was performed to develop recommendations for design of the tank foundations and foundation preparation. Because of a concern for potentially large tank settlements, these recommendations included either (1) a monitored preload of the site with a soil surcharge of 2,000 pounds per square foot (psf) prior to tank construction, or (2) a controlled and monitored stage loading of the tanks with water prior to their placement into service. Due to the urgency of putting the tanks quickly into service, the second alternative was selected and the tanks were constructed. Tank Nos. 1 through 5 were satisfactorily preloaded with water and placed into service. However, during the preloading of Tank No. 6 from June through October 1979, the northeast quadrant of the tank experienced greater settlements which occurred at a faster rate than the remaining portions of the tank. Tank No. 6 was subsequently emptied and a field investigation program was implemented to determine the cause(s) for the observed differential settlements.

This paper describes the studies that were undertaken to analyze the Tank No. 6 settlements, including a comparison of observed settlements and tank distortion with various tank performance criteria and the efforts that were made to put the tank into service on a restricted basis by monitoring the comprehensive soil and tank instrumentation.

TANK NO. 6 CONSTRUCTION AND LOADING

Tank No. 6 is a floating roof 300 feet in diameter tank with a usable product storage capacity of 33 feet and a storage volume of 400,000 barrels. As shown in Figure 2, Tank No. 6 is located at the easternmost portion of the site within 2,000 feet of the Mississippi River.

Foundation designed for Tank No. 6 consists of an 18-inch wide by 3.5 feet deep concrete ringwall to support the tank superstructure and a
three to six foot sand pad within the ringwall foundation to support the tank floor and product load. The sand pad was crowned approximately three feet at the center of the tank during construction to accommodate future tank settlements. The tank shell consists of four courses of butt welded steel panels varying from 1.041 inches at the bottom to 0.375 inches at the top. The bottom plate of the tank includes 5/16-inch thick lap-welded steel panels. A ring stiffener at the top and a floating roof comprise the remainder of the tank structure.

As shown in Figure 3, Tank No. 6 was filled with water in stages to consolidate the underlying soils prior to placing the tank into service following completion of construction activities in May, 1979. During staged loading, a field program was instituted to monitor the behavior of the tank and foundation soils by means of settlement surveys, pore pressure measurements and slope indicator measurements.

Tank settlements were surveyed using 32 settlement survey points established on the rings of the tank. Ringwall settlements measured four survey points are plotted in Figure 3. As indicated in the figure, Tank No. 6 settlement was approximately 8.5 inches at Settlement Survey Point (SSP) 6 and approximately three inches at SSP 21. Furthermore, settlement surveys indicated that the tank shell was differentially distorted downward for a maximum 5.5 inches over a relatively short perimeter length of 220 feet in the northeast quadrant. The remainder of shell settlements were relatively uniform. Figure 4 shows plots of pressures from four piezometers located at depths below the center of the tank and indicates that the pore pressures induced by water loading of the tank were dissipated relatively quickly. Inclinometer data shown Figure 5 indicates that maximum lateral placements in the foundation soils were on order of one inch in the northeast quadrant.
compared to negligible displacements in the remaining portions of the tank.

Starting in late August 1979, Tank No. 6 was unloaded in stages and the tank was completely empty by the middle of January 1980. An investigation program was then begun to determine the cause(s) for differential settlements experienced in the northeast quadrant of the tank.

FIELD INVESTIGATION PROGRAM

A survey of Tank No. 6 ringwall and floor elevations was made in late January 1980 to define the distortions of the tank shell and floor. Survey indicated a localized depression near SSP's 6 and 7 as shown by the floor settlement contours in Figure 6.

The original ground surface near Tank No. 6 was approximately at Elevation +13 feet mean sea level (El +13 MSL) and was raised to El +15 MSL during construction of the perimeter road. Three borings (Boring Nos. JD-1, JD-2 and JD-3) were drilled and four test pits (Test Pit Nos. 1 through 4) were excavated at the locations shown in Figure 2 to supplement pre-construction data obtained by others to more clearly define the subsurface soil conditions around the perimeter of the tank. The subsurface conditions encountered from the borings and test pits are shown in Figure 7. The upper ten feet of soil underlying the tank consists of brown to gray silty clays which are overconsolidated probably due to desiccation. Below this layer, alternating sequences of soft to medium stiff clays, loose to medium dense silts and fine silty sands are encountered to El -50 MSL underneath the northeast quadrant and to El -30 MSL underneath the remaining portions of the tank. These alternating layers of clays, silts and fine sands consist of recent alluvial deposits which were underlain by stiff silt clay and dense fine sand deposits of Pleistocene Age.

Due to the proximity of the Mississippi River to the tank, the ground water table is strongly influenced by the river stage. Based on piezometer readings, the ground water table at Tank No. 6 was conservatively established near El +12 MSL for the analyses.

 Upon completion of the field investigation program, laboratory testing was performed to characterize the strength and compressibility of various soil layers. Laboratory testing included water content determinations, Atterberg Limits, grain size analyses, unit weight determinations, consolidation tests, consolidated undrained triaxial tests with pore pressure measurements and torvane tests.

EVALUATION OF TANK NO. 6

The integrity of Tank No. 6 was analyzed using...
Figure 6. Tank No. 6 Floor Settlement Contours

Figure 7. Subsurface Profile Beneath Tank No. 6
differential ringwall and floor settlements. The cause(s) for the observed differential settlements were evaluated with respect to bearing capacity failure and consolidation settlements.

Differential Settlements

Survey data obtained during January 1980 and on September 20, 1979, when the tank experienced the maximum differential settlements, were analyzed to assess the existing and most distressed condition of the tank. The results of these analyses were then compared to various tank performance criteria that were available in the literature at that time.

Tilting: The best-fit rigid-settlement tilt plane for Tank No. 6, as shown in Figure 8, was determined using procedures suggested by Bell and Iwakiri (1980) and Greenwood (1974). This evaluation was made to determine the significance of differential settlements with respect to bending or distortion of the tank shell or tank floor. The angle of the best-fit rigid-settlement tilt plane was determined to be 0.07 percent, which is well below with the limiting criterion of 0.5 percent. Greenwood (1974) suggested that an average tilt of possibly more than 0.5 percent of the best-fit rigid-settlement tilt plane could be experienced before the distortion (i.e., out-of-roundness) at the top ring girder of a floating roof tank would cause binding between the roof and the shell.

Differential Tank Shell Settlement: Figure 8 also presents the results of an analysis of distortional shell settlements following the method described by Belloni et al. (1974). They define the maximum out-of-plane perimeter settlements as the maximum change in slope between three adjacent settlement points on the shell, computed in relation to the best-fit rigid-settlement tilt plane through all of the settlement survey points. The results of this analysis indicated that the maximum differential settlement (referenced to the best-fit rigid-settlement tilt plane) was 0.13 percent in January 1980 and 0.16 percent in September 1979. These maximum out-of-plane settlements were within the limiting criterion of 0.22 percent as suggested by Belloni et al. (1974) as a "working hypothesis" for safe operation of large tanks with floating roofs. Sullivan and Nowicki (1974) found that large tanks with diameters up to 360 feet could experience as much as 1.2 inches of differential settlements (with respect to best-fit rigid-settlement plane) without problems, but that out-of-roundness problems occurred for differential settlements greater than 1.8 inches regardless of the settlement distribution or tank diameter.

The angular distortion, which is defined as differential settlement between two points divided by the distance between those points along the tank perimeter, was computed to be 0.39 percent between SSP's 3-4 and 8-9. This differential settlement is in excess of the allowable differential shell settlement criterion of 0.35 percent for floating roof tanks (Belloni et al. 1974). DeBeer (1969) suggested a similar criterion.

Bearing Capacity

The measured settlement configuration of Tank No. 6, as shown in Figure 3, suggests no indication of foundation soil failure since the underlying soils were still consolidating and gaining strength at the time of maximum loading. An analysis was performed to assess the factor of
safety against a soil bearing failure below the tank.

**Local Bearing Capacity:** The factor of safety against a local bearing capacity failure at the northeast quadrant of Tank No. 6 was computed to be on the order of 1.6 with a full water preload of 2,000 psf. The computed factor of safety is somewhat greater than the conventional minimum factor of safety against a local bearing failure of 1.5. A two-dimensional, limit equilibrium stability program (Siegel, 1978) was used to compute the factors of safety against localized bearing failure. This computer program is based on a modified Bishop procedure (Bishop, 1955) which utilizes circular failure surfaces. Various searching techniques were employed to define the most critical failure surfaces. The undrained shear strength used in the analysis and the most critical bearing failure surfaces are summarized in Figure 9.

**Overall Bearing Capacity:** Because of the flexibility of large tank structures, the possibility of a general bearing capacity failure is remote and is usually controlled by localized subsurface soil conditions. Nonetheless, an overall factor of safety against a general bearing failure should be examined. A simplified procedure for evaluating this aspect of tank behavior consists of averaging the shear strength of the soil to a depth of approximately two thirds of the foundation's width (Skempton, 1951). Using this approach, a factor of safety against an overall undrained bearing failure of at least 4.5 was obtained for the design load of 2,000 psf. Standard design practice for overall bearing capacity requires a safety factor of at least 3.0 for rigid foundations. Because the soils beneath the tank were only partially consolidated under water test loads, the factor of safety for both localized and overall bearing capacity failure would improve with time.

**Plastic Flow:** When the induced shear stresses reach the bearing strength of the soil, a condition known as "plastic flow" develops in soils. Soils in the plastic state deform under a constant shear stress (Jorgensen, 1934). The distortions produced by plastic flow alter the structure of the soil skeleton causing a loss of shear strength, a further increase in strain and subsequent redistribution of stresses. Assuming homogeneous, isotropic and elastic soil conditions, the maximum shear stress induced in the soil under a circular footing applying uniform bearing pressure, \( P_{u} \), is approximately \( P/\pi \), or about one third of the foundation bearing pressure. The maximum shear stress develops in the soil along a bowl-shaped surface with a maximum depth of approximately 0.4 times the diameter of the loaded area as shown in Figure 8. The shear stresses are lower above and below this surface.

The average undrained shear strength of the upper 60 feet of soils underlying the northeaster quadrant of Tank No. 6 is approximately 675 psf based on laboratory test data. Using the approach outlined above, bearing pressure of 2,000 psf resulted in a maximum shear stress of 6 psf in the soils below the tank. Since the average undrained shear strength of 675 psf greater than the maximum induced shear stress 635 psf, plastic flow probably did not significantly contribute to the overall settlements of the tank although localized overstressing may have occurred. Figure 5, which shows a plot of the inclinometer data near SSP's 6 and 7, indicates that plastic flow may have resulted on the order of one inch settlement below the northeast quadrant of the tank.

**Settlement Analyses**

Using one-dimensional consolidation theory analyses were performed to determine the total primary and secondary settlements below the northeast quadrant and other locations of the tank. Localized soil profiles (See Figure 4) were developed at each location considered using data acquired during the subsurface exploratory programs. The soil properties used in these analyses are provided in Table I.
The soil profile and soil parameters used for settlement analysis at the center of the tank were compiled by averaging the data obtained from investigations conducted before and after tank construction.

**Primary Settlements:** Using the laboratory consolidation test results, the maximum total primary settlements with a design load of 2,000 psf were estimated to be approximately 20 inches at the northeast quadrant and at the center and four inches for the remaining portions of the tank. Based on the settlement survey data, the northeast quadrant of the tank would have experienced 12 inches of additional settlements as compared to only one inch for the remaining portions of the tank. These additional settlements would have resulted in unacceptable tank distortions and could potentially have resulted in structural failure of the tank.

Settlement data were also analyzed to determine the expected maximum settlements underneath the northeast quadrant and other locations below Tank No. 6 using a procedure suggested by Su (Transportation Research Board, 1976). Using the settlement survey data and this procedure, the total primary settlements at SSP's 6 and 21 were estimated to be 12.8 and 4.4 inches, respectively as shown in Figure 10.

**Settlement Rate:** Rates of primary settlement were computed by utilizing pore pressure measurements obtained during the June-October 1979 preloading shown in Figure 4, the ringwall settlement data shown in Figure 2 and coefficients of consolidation from laboratory tests. These analyses indicated that approximately 140 to 280 days would be required to achieve 90 to 100 percent of anticipated primary settlement under a design loading of 2,000 psf. Since the tank had been preloaded approximately 150 days since June 1979, the estimated degree of consolidation was approximately 50 to 60 percent below the northeast quadrant of the tank and 90 to 100 percent elsewhere.

**Secondary Settlements:** Secondary settlements were computed to be approximately four inches and two inches, respectively at the northeast quadrant and remaining portions of the tank over the anticipated 20 year life. Therefore, if the effect of localized differential settlement associated with the consolidation of underlying soils could be corrected, it was concluded that secondary settlements would not adversely affect the structural integrity of the tank due to their relative uniformity.

**REMEDIAL MEASURES**

The analyses performed to assess the integrity of the tank indicated that Tank No. 6 was struc-
turally sound in its present unloaded condition. However, because the soils below the northeast quadrant were only 50 to 60 percent consolidated under the design load of 2,000 psf, recommendations were made to relevel the tank prior to its further preloading with water to minimize the possibility of structural distress. Remedial measures included mudjacking below the northeast quadrant to lift this area to an equal or higher level with respect to the remaining portions of the tank and preload and monitor the behavior of the tank prior to placing it into service.

PERFORMANCE MONITORING

Tank No. 6 remained empty from January 1980 until March 1981 without any attempt to relevel the tank by mudjacking. In March 1981, an elevation survey of the ringwall foundation was performed which indicated two inches of uniform rebound had occurred since emptying the tank in January 1980. At this time, the DOE indicated that it would be desirable to utilize Tank No. 6 to accommodate upcoming oil transfer operations to the Weeks Island and Bayou Choctaw sites. The analyses of March 1981 survey data showed that Tank No. 6 could be loaded intermittently provided that its behavior was carefully monitored. Consequently, strain gages mounted in a rosette pattern were installed at several locations on the tank shell to measure changes in strain level as product was loaded into the tank. In addition, piezometers installed during the post-construction investigation program were prepared for monitoring and arrangements were made to survey the ringwall elevations on a daily basis.

Initially, Tank No. 6 was filled to height of 20

Figure 11. Tank No. 6 Oil Loading and Settlement Data

Figure 12. Tank No. 6 Oil Loading and Pore Pressure Data
As shown in Figure 13, maximum and minimum principal stresses in the tank shell ranged up to approximately 14,000 pounds per square inch (psi) and 9,000 psi, respectively. These shell stresses were well below the allowable stress level of 21,000 psi for the steel tank although the effect of residual stresses remaining in the tank shell before installation of the strain gages was unknown.

CONCLUSIONS

The behavior of Tank No. 6 and its foundation soils were analyzed to determine the cause(s) for differential settlements observed in the northeast quadrant of the tank. The probable cause for the differential settlements was the presence of thicker, normally consolidated compressible clay and silt layer below the north-
east quadrant of the tank as compared to the soil below remaining portions of the tank. This difference in soil layer thickness and soil behavior may have been caused by siting a portion of the tank over a filled river meander. The studies confirmed that the settlement patterns and rate of settlements were generally consistent with consolidation theory and that the tank foundation soils did not experience a bearing failure.

Tank No. 6 was allowed to be used to store product for short time periods with its behavior monitored carefully. Previously experienced levels of differential settlements were used as upper limit guideline to allow restricted use of the tank. The unrestricted use of Tank No. 6 as a surge tank will require its leveling by mudjacking and preloading until the primary settlements in the northeast quadrant are completed.

During preloading and performance monitoring of Tank No. 6, differential ringwall settlements were evaluated with respect to the best-fit rigid-settlement tilt plane and compared with published guidelines for allowable differential movements. The following conclusions are drawn from these observations.

(1) The suggested allowable out-of-plane shell settlement of 0.22 percent appears to be a reasonable guideline for maximum allowable out-of-plane movements for floating roof tank shells. The maximum out-of-plane shell settlement for Tank No. 6 was 0.16 percent.

(2) The suggested allowable angular distortions of 0.35 percent appears to be too conservative. Tank No. 6 has experienced maximum angular distortion of 0.42 percent and has not shown any signs of structural distress.

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REFERENCES


