On Short-Term and Long-Term Behavior of Large Diameter above Ground Steel Storage Tanks Founded on Glacial Tills

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ABSTRACT

The paper summarizes settlement records taken over periods of weeks and up to 45 years on above ground steel storage tanks 20 m to 50 m in diameter, 14 m to 20 m high, founded on fine-grained glacial tills. Soil information for each of the tanks is provided from different sources such as conventional boreholes, test pits, and sometimes Dilatometer tests. Three newly constructed tanks have been instrumented with piezometers and a tank base hydraulic profiler for monitoring during hydrotesting. The presented long-term settlements for the older tanks, and the short-term monitoring data collected from the hydrotested tanks are examined and commented on with respect to the face value of the records. The ability to apply practical geotechnical engineering methods to provide reasonable predictions of the behavior of tank foundations is also discussed.

INTRODUCTION

The typical tank sizes under this review range from 20 m to 50 m in diameter and 14 to 20 m in height. The specific gravity for the product stored in the tanks would range from 0.9 to 1.20. The operation regime for all the tanks in discussion involves frequent fill ups to near the capacity followed by emptying to variable levels.

In the process of designing the retrofitting of several old large diameter steel tanks and the construction of three new tanks, some historical records of settlements have been made available. Also, in the pursuit of increased tank capacities a trend to raise the standard tank height to 18 m, or more is manifested. For these later cases a monitoring of the porewater pressures in the foundation soils and of the deflections under the tank base was added to the usual leak proof test, also called “hydrotest”, which all repaired tanks, and new tanks have to be subjected to as a standard procedure in the industry.

All the tanks in discussions are located within a relatively limited geographic region within Lambton County, in Ontario, Canada, which is covered mostly by Pleistocene deposits of fine-grained glaciolacustrine materials.

The older tanks were built in the late 1950s or early 60s with floating roofs and were placed typically on a thin granular pad over pre-existing ground surface. Retrofitting of some of these tanks after decades of operations incorporated different levels of structural upgrading varying from base plate reconstruction, replacement of the floating roofs with fixed roofs, and also some levels of improvement to the bearing surface such as regrading of the granular base, or inserting of a “ring wall” under the shell, or a complete overhauling of the entire tank pad.

Geotechnical investigations of various complexities (from test pits to sampled boreholes, flat-blade dilatometer probes – DMT) and analyses were commissioned for different tank repairs, or new constructions. The investigations of the existing tanks revealed almost invariably that the granular pads under the tanks did not have sufficient thickness and appropriate drainage and at times became impacted by seasonal freezing and thawing, alternating with periods of excessive drying. The new standards for tank pads provide for elevated granular pads above the general grades within the tank lot. Also, the thicknesses of the pads are increased to extend below the depth of frost penetration to protect frost-sensitive native soils from exposure to freezing.

Settlement records extending for long periods of time comparable with the life span of a tank were available only for a reduced number of tanks. The records were taken at the outside perimeter of the tank base (rim) and suggest that the order of magnitude of the rim settlements could be over 200 mm. By extrapolation, the settlements under the tank center should have reached 300 or 400 mm. Yet, there is no evidence that the foundation soils have been completely consolidated even after 45 years in operation. This condition may have to do with the frequent fluctuation of the loads in conjunction with the very low permeability characteristics of the native foundation soils.

The mechanics of settlements under such highly fluctuating loads and fully exposed foundation soils to the elements seem to be a bit more complex than the simple models of fixed loads bearing on inert foundation materials.

Records and monitoring information are provided in this paper along with limited interpretations and highlights. The authors are engaged mainly with the practical aspects of the projects. In essence, the objective of the tank investigations was to provide recommendations for retrofitting, and / or new construction on the basis of conventional geotechnical methods. It is hoped that by publicizing these records and data the engineering community will be assisted when dealing with similar projects.

BRIEF ANECDOTAL HISTORY OF HYDROCARBON STORAGE TANKS IN LAMBTON COUNTY

Since the discovery of crude oil in Lambton County in the mid 1800's ways and means of temporarily storing the oil were developed on an experimental basis. Tank construction evolved from plank-lined excavations in clay, to above-ground wooden and steel-plate vessels. The height of the upright vessels was often restricted by the ability to contain the fluid pressure near the base. As construction materials improved the increased height and diameter imposed greater loads on the soil and settlement became a problem. By the mid 1900's a common design height of about 15 m was established. Recently, attempts to "push the envelope" have resulted in the successful construction and operation of large diameter 19.8 m high tanks.

GEOLOGICAL BACKGROUND

The geographic outline of the region under the scope of the present paper is described in Fig. 1. The area is within Lambton County, Ontario, Canada, and is confined by Lake Huron at the north, the St. Clair River at the west, Hwy 21 at the east, and Hwy 80 at the south. The area lies at the northwest corner of the physiographic formation known as the St. Clair Clay Plains which are essentially till plains smoothed over by shallow deposits of lacustrine clays that settled in depressions, while the knolls were eroded by wave action (Chapman, Putnam, 1984). The ground surface is undulating within the elevation range of 180 m to 205 m above sea level (masl). The underlying bedrock is mostly shale from Upper Devonian Kettle Point formation with patches of shaly limestone from the Hamilton formation (Ontario Geological Survey, 2000a). Deeper sedimentary post Cambrian deposits include several oil and gas producing horizons, as well as a rather thick deposit from the Salina group. The Cambrian base lies at about 1500 m + below the surface (Raven *at al.* 1992)

There is an apparent acceptance that the bulk of the Quaternary deposits in the region, typically of an average thickness between 40 m and 45 m, were formed within a fresh water environment, during the latest glaciation-deglaciation of the Late Wisconsian age between 10,000 and 20,000 years ago. The vast majority of the overburden thickness is comprised of a fine-grained silt and clay matrix with embedded sand and gravel, sometimes stringers / seams / pockets / lenses of silts, sands, fine gravels, occasional clast, occasional cobble, and even boulders. The Ontario Geological Survey (2000b) describes this deposit as a "Fine-Grained Till" deposit with a content of up to 55% clay-size particles, up to 40 to 70% silts, and usually less than 15% sand, or fine gravel size fraction. In the usual practice, the native soils in this region are generically called "silty clay" tills, or glaciolacustrine "silty clays".

Quigley (1980) states that the Port Huron readvancement of about 13,500 BP caused a "major complication" in the geotechnical understanding of the overburden soils in the Sarnia area. He believes that the ice sheet was partially floating when it overrode previously deposited soft clays, and as such, there was little consolidation imparted to the older (and deeper layers). Notwithstanding, except for the upper portion of the overburden which exhibits strong overconsolidation (easily explained by hardening of the crust as a result of desiccation, frost action, and general weathering) the entire deep deposit still presents some slight to moderate levels of overconsolidation. Different authors suggest possible causes such as some minimal loads form the partially buoyant ice sheet, groundwater drawdown (Soderman, Kim, 1969), reworking by glaciers of the previously deposited lacustrine silty clays (Adams, 1970), or some sort of cementation, such as from carbonates, or other agents (Boone & Lutenegger, 1997).

The general low permeability characteristic of the bulk of the clayey till overburden causes this deposit to form a regional aquitard, overlying a thin, and perhaps discontinuous "freshwater" aquifer, known as the Interface Aquifer, which occurs at the interface between the Quaternary deposits and the underlying Devonian shales (Husain *at al.* 2004). The hydrostatic pressure in the aquifer was measured to fluctuate, but in general has a slight artesian character, i.e. the potentiometric surface is above El. 180 masl, and up to 220 masl while the ground surface elevations hover between 180 m and 205 masl within the area of interest. It is interesting to note that the average water level in Lake Huron and the St. Clair River is around El. 176 m which arguably illustrates the hydraulic separation between the buried aquifer and the surface waters.

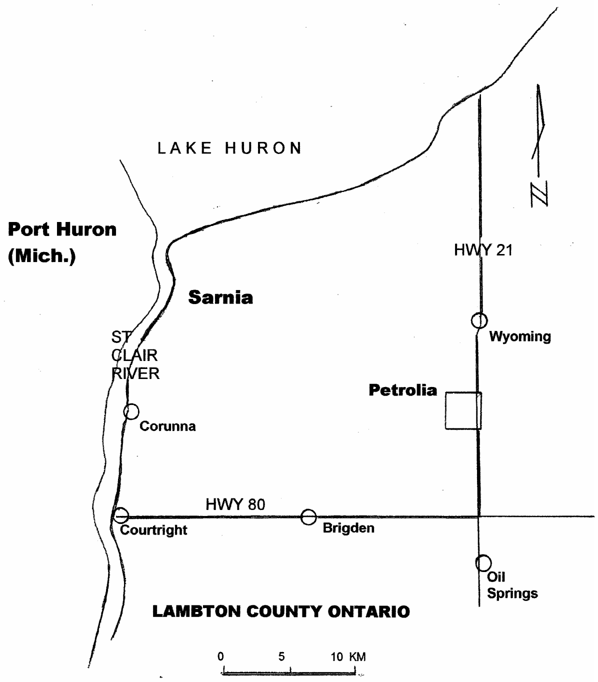

Fig. 1. General tank site area

GEOTECHNICAL PROPERTIES

Disregarding the very shallow fills and natural topsoils, the bulk of the overburden is comprised of fine grained silty clay, sandy clays, clayey silts, and silt beddings with occasional seams, pockets, or lenses of sand and gravel mixtures. The fine grained component material has a low to medium plasticity and low activity. A typical geotechnical profile could be quite well represented by a three layer system.

- *Actively weathered* layer of up to 1.2 to 1.5 m depth below grade where under the present climatic environment seasonal moisture and temperature changes are intensively felt. The maximum depth of frost penetration in the region is considered currently to fluctuate near the 1.1 m mark. The soils in this layer exhibit a typical brown-grey mottled aspect, very wet texture in springs, and very dry-crumby texture after prolonged dry summers.
- *Desiccated crust* layer, extending usually to 3.5 to 4.5 m below grade. The crust is essentially unsaturated, of a prevalent uniform brown color, currently with little seasonal variations in temperature and especially in moisture content. From a geological perspective the desiccated crust is included in the "weathered" zone of the overburden on the accounts of the major transformation suffered by the material since its underwater deposition, be it in proglaciar lakes, or under the ice sheet itself. The desiccation experienced after the receding of the meltwaters is believed to be responsible for the elevated levels of overconsolidation of the crust, ranging from OCR 20 to over 100.
- *Grey zone*, extending from the underside of the crust to the underlying bedrock. This zone is in a virtually permanently saturated condition, with almost no seasonal changes in temperature or moisture content. The material is nearly normally consolidated to slightly oveconsolidated (OCR typically ranging from 1.3 to 3.5). As mentioned earlier, the explanations for this overconsolidation are still debated. It is interesting that the sheer aging of the deposit of 15 k, and older, seems not to appear in debates as a possible factor of consolidation via slow rate creep.

LONG-TERM TANK SURVEYS

Long term surveys available to us cover periods from several years and up to 4 decades, and consist essentially of several sets of elevation surveys along the top of the tank base rim. The surveys have been completed by different Surveyors. Sometimes there is no information about the date of the survey other than the year. Unfortunately very little effort was made to track the tank loading at the time of the settlement surveys. Perhaps this is because of the cyclical nature of the fluid levels during normal operation. On an anecdotal basis, most of the tanks in the industry are frequently loaded to near the design capacity followed by unloading to variable levels, but seldom to complete emptying. Notwithstanding the lack of more rigorous data, the number of surveys over extended periods of time is believed to provide a reflection of the order of magnitude for the integrated effects (settlements, in particular) of the extensive use of the tanks, as it happens in the real world. The picture provided by these records offers the basis of judging the engineering design and helps with the calibration of our methods of prediction.

The following is a presentation of several tank settlement records along with available soil data and brief commentaries on the survey results. Table 1 summarizes the tank sizes and basic information about the surveys.

Table 1. Long-Term Survey Tank Summary

Tank	Year Built	Diameter/Height (m)	Estimated max. Load (kPa)	Survey Period
E203	1957	41/14.6	143	1958-2003
E204	1957	41/14.6	143	1959-1963
E208	1967	46/14.6	143	1989-2003
E209	1968	46/15.3	150	1989-2002
E214	1974	46/14.6	143	1989-2006

Tank E203

This tank was built in 1957 as an above ground structure with floating roof. In 2003 the tank was taken out of service for maintenance and cleaning. At that time the tank showed visible settlements at the shell and across the tank base.

Select soil information from 4 relatively shallow (6.5 m deep) conventional boreholes in the vicinity of this tank are shown in Fig. 2. More extensive subsurface information will be provided later, under the heading for Tank E209 that is located within the same vicinity along with all the other tanks in the "E200" series.

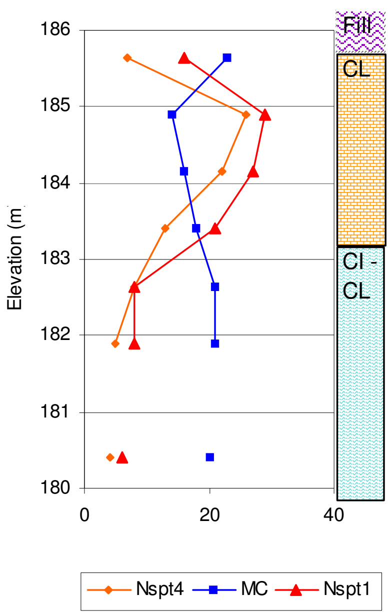

Fig. 2. Tank E203. Borehole summary

From test pits dug directly below the edge (rim) of the tank base, it was determined that the tank sits on a granular bedding 200 to 225 mm thick underlain by weathered native silty clay. Occasional lenses of old topsoil are present below the granular base at some locations.

Elevation records taken along the perimeter of the tank on the rim of the steel base projecting about 20 or 30 millimeters away from the tank shell are available between 1958 to 1963 by one surveyor firm, and 1989 to 2003 by a different firm. The shots have been taken at angular spacing of 36 degrees, which represent about 42.5 feet (12.95 m) length of arc. Charts of the recorded settlements at the survey stations starting from Sta.1 at the tank north, and increasing clockwise to Sta10 are provided in Fig. 3. Obviously, the first set of readings listed 14 September 1958, taken in the following year after the tank construction, would not necessarily represent the initial preloading condition of "zero" settlements.

There is no information about the tank load level at the time of survey other than that at the 2003 survey the tank was empty. Yet, compared to the previous 3 sets of readings spreading over more than one decade, thru which the survey data suggest there were virtually no more changes in the settlements, the 2003 survey is posting a "jump" of about 2 inches (50 mm) in the settlement values.

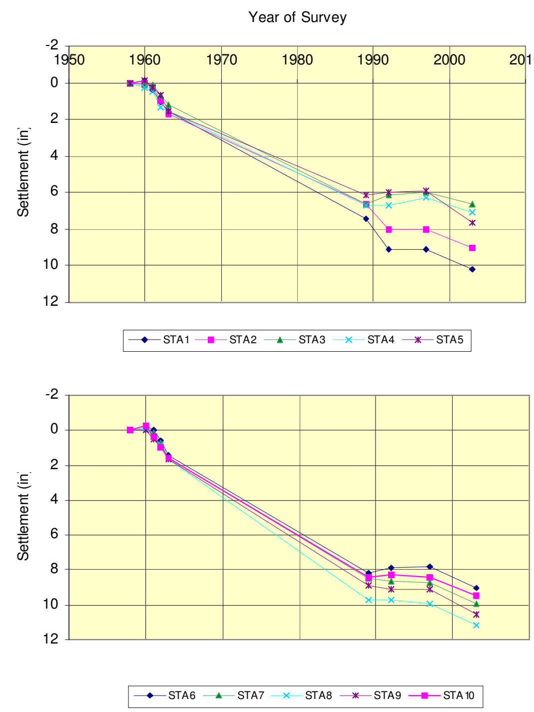

Fig. 3. Tank E203. Long-Term Rim Settlements

Also, there is no information about the day and month for any of the surveys since 1989. The earlier surveys were all conducted within summer months which eliminate the suspicion about the ground frost heave from the long list of uncontrolled factors impacting the survey results. In spite of all the uncertainties discussed above, the trends of the settlement plots seem to indicate that more settlements should have been expected after 2003, i.e. after 45 years in operation, if it weren't for the retrofitting in 2003.

Tank E204

There is no subsurface data in the immediate vicinity, and directly under this tank. However, it is reasonable to assume than no major bearing differences form the conditions exposed at the other tanks in this series E200 should be present at this particular E204 tank.

Rim settlement records (Fig. 4) were taken between 1959 and 1963 by the same surveyor at 10 stations similar to those described for Tank E203. As before, it is not known how much settlement had occurred between construction in 1957 and the date of the first available survey on 14 September 1959. In spite of all uncertainties and some scatter at some of the readings, the charts in Fig.4 seem to indicate a distinct trend of acceleration of the settlements beginning at year 4 after construction and maintaining at year 5. Should the tank load have been relatively constant over this time period, than the noted acceleration would signal the initiation of a massive foundation failure, notwithstanding the very slow rate of settlement increase of about 10 mm to 15 mm between year 4 to year 5, and about 20 mm to 25 mm between years 5 and 6. Obviously there was no major subgrade failure in this case since the tank "survived" essentially unscathed until its overhauling in 2003.

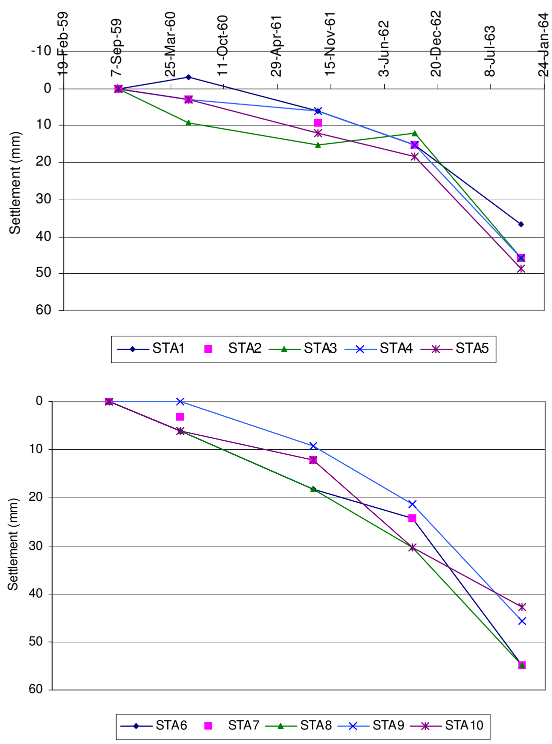

Fig. 4. Tank 204. Long-Term Rim Settlements

There could be several other explanations for the apparent acceleration of the settlements, such as a series of special circumstances, if not coincidences, about the tank load levels at the particular dates of surveying (one in the month of June, and all the others were taken in the months of September and October), coupled with some special weather conditions causing shrinkage or swelling under the rim, and not the least, survey errors. It is obvious that all of the above are everything but attractive explanations.

In March 2003 the tank was structurally overhauled, including, without being limited to the shell lifting, replacement of the outer 1.2 m annular floor ring with a new steel plate, and regrading of the granular fill under the replaced floor ring. Following the overhauling, tank base elevation profiles were completed before, and after the tank hydrotest.

<u>Tank E208</u>

This tank was slated for maintenance in 2007. Built in 1967 as an above ground tank with floating roof, the tank was allegedly in undisrupted service since construction, except for 1987 when the tank was emptied, cleaned and the floor painted. From a few test pits completed in 2007 it is known that the tank base bears on about 225 mm to 275 mm thick layer of sand and gravel fill placed over native mottled brown-grey-green silty clay subgrade.

The general condition of the foundation soils was examined in 4 conventional 6 m deep boreholes. The summary of the SPT and moisture content data shown below (Fig. 5) confirm that under this tank the soils are similar with the soils under all tanks in the class E200 discussed in this submission.

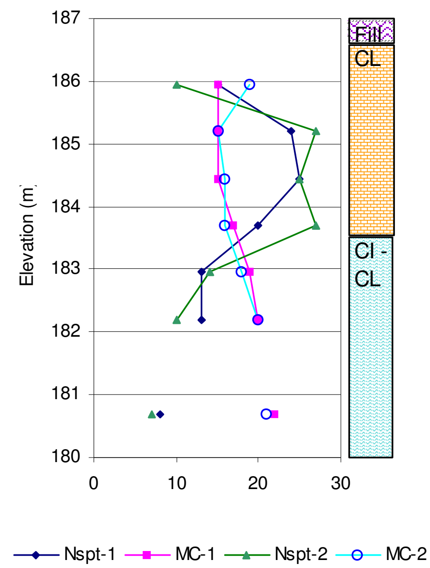

Fig. 5. Tank E208. Summary of borehole data

Elevation records taken along the perimeter of the tank base rim were available between 1989 and 2003. The shots have been taken by the same surveyor at an angular spacing of 11.25 degrees, which represent about 15 feet (4.57 m) length of arc. Charts of the recorded settlements at the survey stations starting from 0+00 at the tank north, and increasing clockwise by 15 feet are provided in Fig. 6.

Obviously, the plotted charts describe the settlement changes with respect to the elevation survey of 1989. There is no information regarding the settlements that occurred over the previous 22 years in operation, nor are the dates available of the surveys and data regarding the tank loads at the time of survey. The unusual shape of the time-settlement variation curves may be genuinely related to a lower level of tank load in 1997 than in 1989 followed by a larger load in 2003. If so, these records would reflect more of the cyclical-elastic response of the foundation soils and less of the long-term consolidation settlement.

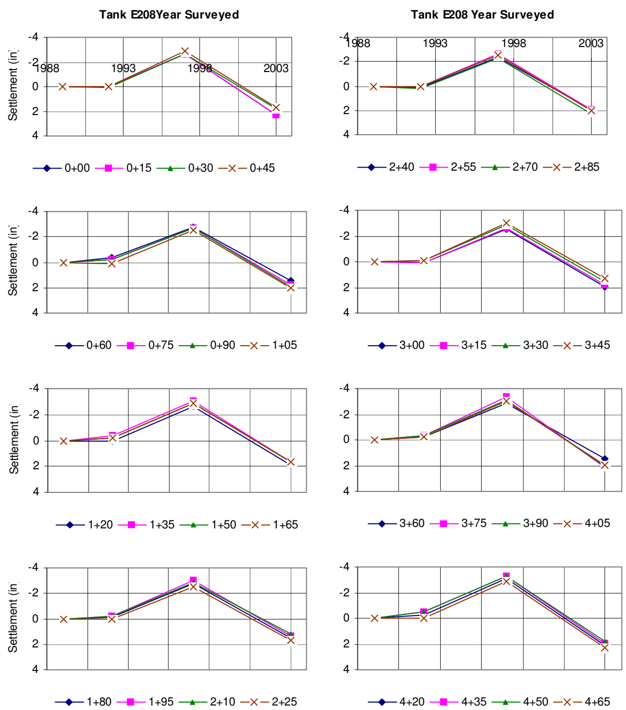

Fig. 6. Tank E208. Rim survey between 1989 and 2003

Tank E209

This tank, constructed in 1968 as an above ground structure with floating roof, was in operations until 2002 when it was emptied and submitted to a structural inspection and repairs.

Also in 2002 one conventional sampled borehole using the Standard Penetration Test (SPT) method and one flat-blade Dilatometer Test (DMT) were advanced in the close vicinity of the tank. From test pits excavated right below the tank rim it was found that the tank base was essentially flush with the surrounding grades (near El. 187.1 m asl) and was supported by an average of 300 mm of and gravel fill over about 150 to 200 mm of fine sand fill over old clayey topsoil followed by native inorganic weathered silty clay soils. The borehole and the DMT were advanced to a maximum depth of about 28 m and encountered silty clays with lenses of silt and silty sand that are typical for the region. A summary of the determined soils properties are provided in Fig. 7 and Fig. 8 of below.

Elevation records were taken along the perimeter of the tank base rim since 1989 by one surveyor at an angular spacing of 11.25 degrees, similar as described at Tank E208. The measured settlements since the initial survey in 1989 are provided in Fig. 9. The shapes of the charts are suggesting a trend of consolidation, however, this cannot be proven beyond doubt because it is highly unlikely that the tank load was constant over the entire period. Notwithstanding, the amplitude of the settlements in this case approaches 100 mm

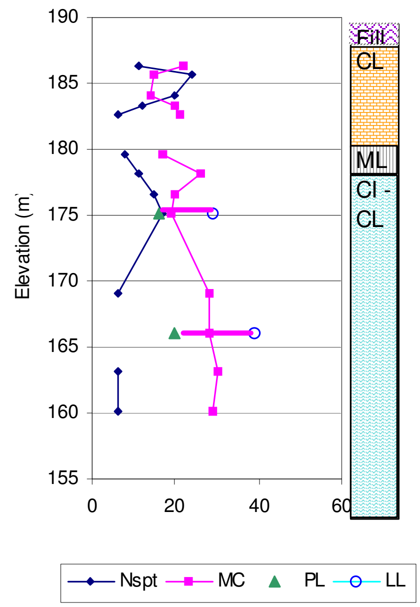

Fig. 7. Tank E209. Borehole summary

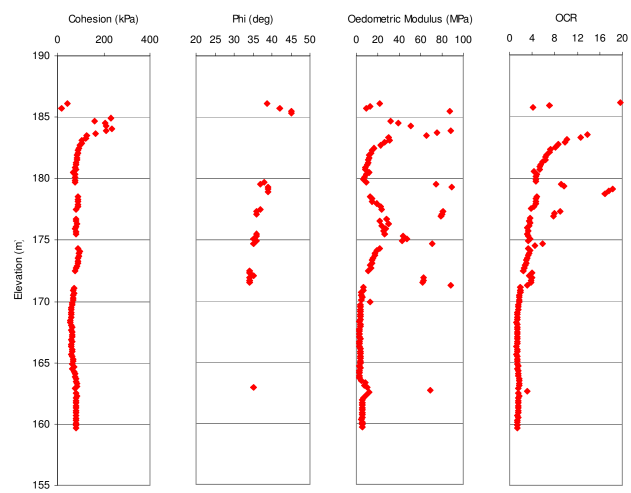

Fig. 8. Tank E209. DMT data

(4 inches), or even 125 mm, which is significantly greater than the range of 15 mm to 30 mm that will be shown later to represent the elastic component of the soil deformation. Hence, some distinct levels of consolidation should have occurred over the survey period.

In April 2002 the tank was completely emptied and subjected to a condition survey which included the tank base elevation. Elevation shots were taken along 16 radial profiles, S1 thru S16, at distances measured from the tank shell of 0.5 feet

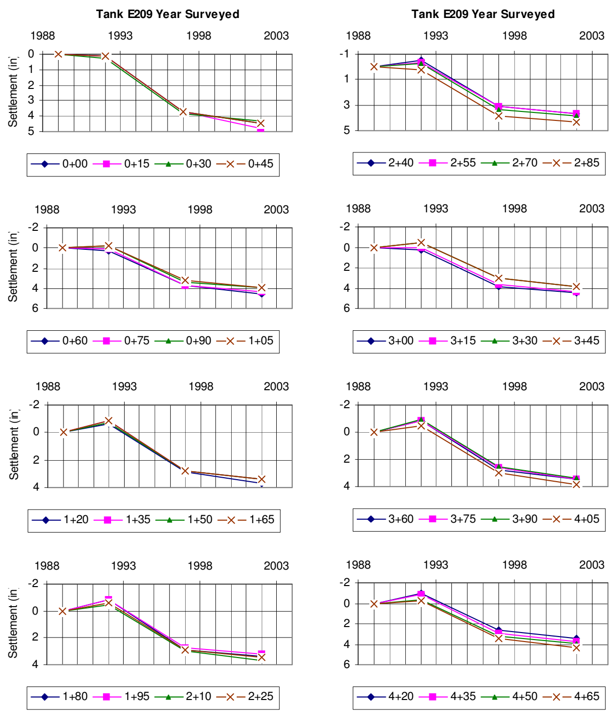

Fig. 9. Tank E209. Rim Settlement between 1989 and 2002

(0.152 m), 1 foot (0.304 m), 2 feet (0.608 m), 5 feet (1.52 m), 10 feet (3.05 m), 15 feet (4.57 m) and 20 feet (6.1 m) within each profile. The profile S16 was oriented along the site North and the remaining profiles S2 thru S15 were counted in the clockwise direction at equal intervals of 22.5 degrees. In Fig. 10 are plotted the tank bottom deflections relative to the edge of the tank base. It should be noted that the edge of the tank itself was undulating up-and-down along the tank perimeter, so that the shape of the tank base was even more complex than suggested by the charts.

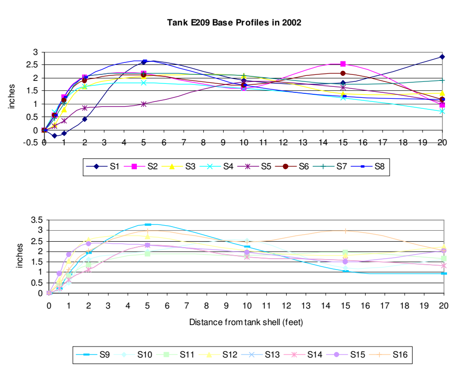

Fig. 10. Tank 209. Base deflection profiles

Recognizing that the thin steel tank base plate does not follow strictly the ground surface beneath the tank because of random flexural distortions caused by welds, plastic yielding, especially under the floating roof supporting posts, etc., yet there appears to be an almost consistent pattern of a 3 to 4 inches (75 mm to 100 mm) elevation differential (crossfall) within about 5 feet (1.5 m) from the edge (rim) of the base plate. The construction drawing of 1968 consulted by us does not provide for such local crossfall; instead, the drawing calls for a general slope of one inch (25 mm) over 15 feet (4.57 m), which represents a gradient of 0.55% between the tank center and its edge. The recorded local crossfall is about 10 times steeper than the presumed "as-build" condition, which could reflect a "sinking" of the tank rim, possibly due to local subgrade failures along the tank perimeter.

Tank E214

This tank, constructed in 1974 as an above ground structure with floating roof, was in operations until 2006 when it was emptied and subjected to a structural inspection that determined the replacement of the tank base in 2007.

Elevation records were taken by the same surveyor at 32 stations along the perimeter of the tank base rim between 1989 and 2006. Charts of the calculated settlement variations at the survey stations starting from 0+00 at the tank north and increasing clockwise by 15 feet are provided in Fig. 11.

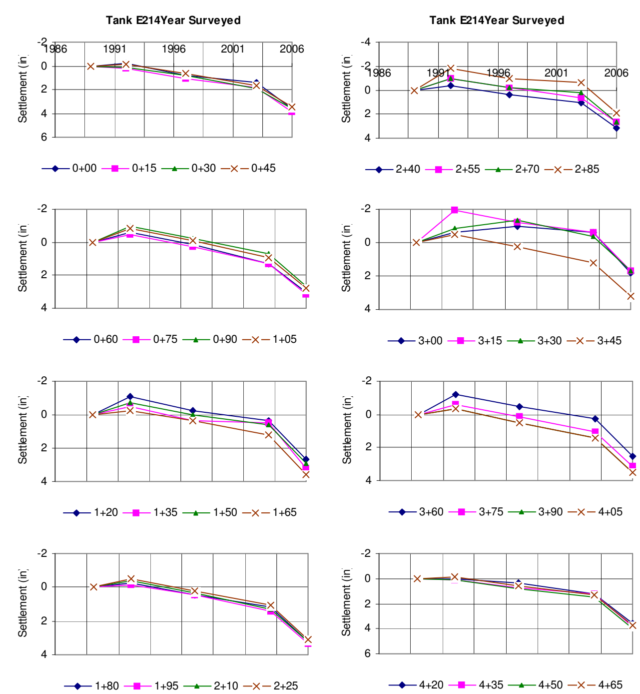

Fig. 11. Tank E214. *Rim Settlement between 1989 and 2006*

The records cover only the settlement changes which occurred after 1989. Yet, the trend is indicative of acceleration of the deformation, especially during the last 3 years interval of the survey, i.e. between 2003 and 2006, i.e. after more than 29 years in operation. The total settlement increase over the 7 year period of surveying approached 4 inches (100 mm). The second set of readings, in 1992, showing a relative heave compared to the 1989 readings, or "flat movement", points to the assumption that the tank load in 1992 should have been distinctly lower than that at the survey in 1989. Then, since 1992 till the last survey in 2006 the settlements gained a full 4 inches (100 mm) plunge. Again, probably there is no better explanation for this apparent acceleration than a coincidental situation that the tank had more load at the survey in 2006 than it had at the survey in 2003, and much more than in 1992.

SHORT-TERM TANK SURVEYS

These surveys were conducted during limited periods of time when the tanks were tested for leaks under the so-called 'hydrotests'. A typical hydrotest is conducted over a period of one or two weeks, and comprises the filling of the tank with water at some prescribed heights (usually 25%, 50%, 75% and 100% of the tank capacity) and holding for one day, or so under each load step. Among other checkups, a rim elevation survey is implemented by some companies. There are little variations of the test protocols in term of rate of loadings, durations of holding at different load levels, and rates of unloading. Many companies would not complete any rim settlement survey during the hydrotest.

Recently, the need to increase the height of three tanks arose due to real estate restrictions. The hydrotests for these tanks (ES600, ES800A and ES800B) were augmented with tank instrumentation including a tank base profiler and pneumatic piezometers at different depth levels under the tanks. Also, the tank filling programs were modified to accommodate more refined loading-unloading schemes and to achieve some pre-established response targets in terms of settlement and porewater pressure rates. While these special hydrotests took somewhat longer than the conventional hydrotests, something in the range of one to a few months, yet the levels of primary consolidation achieved were negligible, and as such all these hydrotests are considered to reflect essentially the short-term, or "immediate", or "instantaneous" response of the foundation soils.

Table 2 summarizes the sizes and load information for the tanks subjected to hydrotesting and presented in this paper.

Tank E204

Detailed soil and foundation conditions for Tank E204 were not available, but, as addressed before, at the Long-Term Tank Survey section, there are no reasons to believe significant differences from other tanks the E200 series. As mentioned, this tank was overhauled in 2003 which included also a hydrotest and a tank base surface survey before, and after the hydrotest. Below (Fig. 12) are provided the settlement survey results for the 50%, 100% load steps, and immediately after unloading.

Table 2 Summary of Short-term Tank Monitoring

Tank	Diameter (m/ft)	Height (m/ft)	Estimated maximum test Load (kPa/psf)
E204	41/135	14.6/48	145/3000
ES600	27.45/90	18.3/60	180 /3750
ES800A	21.35/70	18.3/60	180/ 3750
ES800B	21.35/70	18.3/60	180/3750

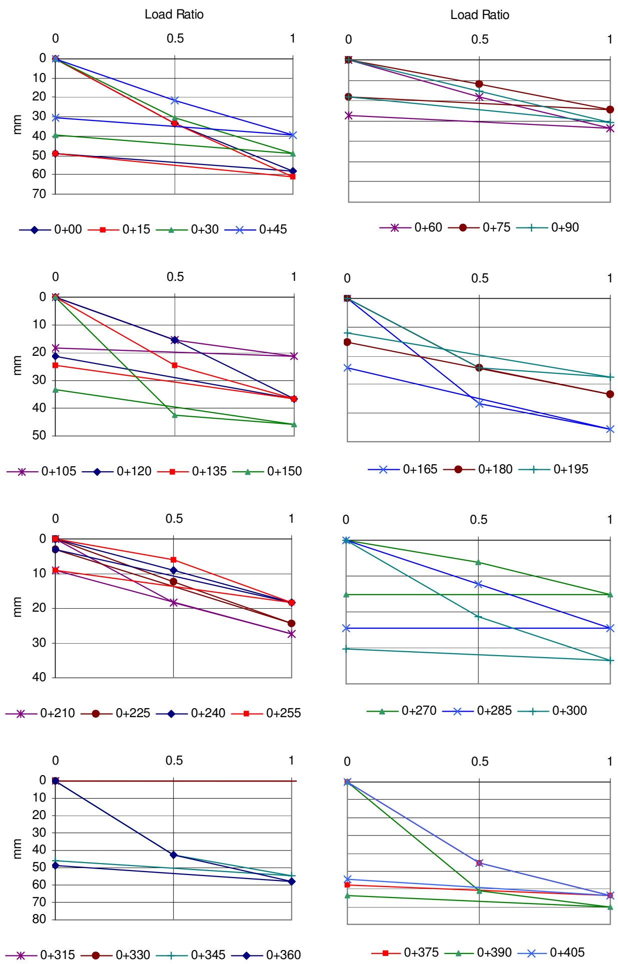

Fig. 12. Tank E204. Rim survey during Hydrotest March 28-May 2, 2003

To help visualize the actual distortion of the tank rim, the maximum settlements reached during the hydrotest along the tank rim, and of the immediate residual settlements one day after the tank unloading are plotted in Fig. 13 below.

Maximum settlements of 60 to 70 mm were reached around the North stations. At Stations 225 and 240 (close to the West rim). the maximum recorded settlements were only 16 mm.

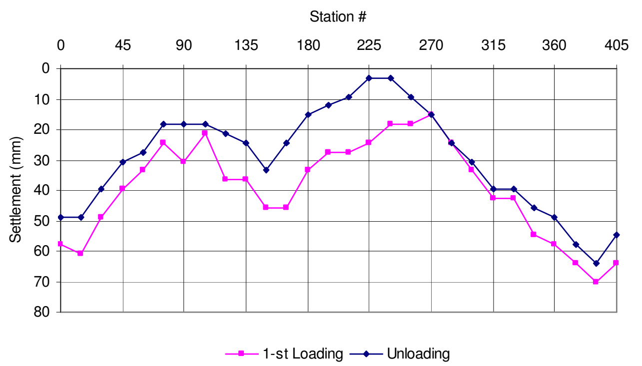

Fig. 13. Tank E204. Rim profile of maximum and residual settlements during 2003 hydrotest

At the majority of the stations, the settlements grew almost linearly with the load increase. However, at about 40% of the stations, mostly within the NW quarter of the rim, the settlement evolution showed a strain-hardening pattern which likely suggests that the granular pad had areas of poorer compaction where higher settlements and tank deflections were necessary before engaging the more consistent subgrade reaction, once the load was stepped up.

After unloading, the immediate residual settlements are quite close to the maximum recorded settlements. This suggests that most of the reached settlements could have been due to the denting (punching) of the granular pad by the tank rim. Obviously, the balance of the settlements should have been on accounts of the elastic deformation of the foundation soil and on some levels of irreversible distortions and compression. Probably the closest representation of the elastic component of the total settlement is given by the recoverable settlements. Using this assumption, the back calculated elastic moduli, E, assuming an ideal elastic half-space foundation soil would vary from about 130 MPa to over 1000 MPa, with the majority of the data between 300 and 400 MPa. At two stations the recovered settlements were zero, which obviously had to be eliminated from the discussion. Such elastic moduli seem to be 2 times to almost 10 times larger than the general recommendations E = 500 to 2000 times the undrained cohesion (USACE 1990).

<u>Tank ES600</u>

Tank ES600 was erected in 2005 and was subjected to an extended hydrotest program intended to demonstrate that the foundation soil can safely accommodate rapid loading to stress levels that would cause a reduction of the immediate factor of safety against the general subgrade failure to about 2. Traditionally, the previous tanks have been designed for a factor of safety closer to 3.

The design operation load for this tank is about 205 kPa while the maximum hydrotest load was only up to 180 kPa. Notwithstanding, it was considered that the hydrotest load would be close enough to the design load, and if the foundation response is proven acceptable under the short-term hydrotest load, than the tank will be safe also under the intended long-term operation loads not exceeding by more than 15% the maximum hydrotest load.

The subsurface conditions under the tank were explored in a conventional borehole (Fig. 14) augmented with laboratory routine testing and a couple of consolidation tests, plus a DMT profiling (Fig. 15).

The tank was built on a gravel pad raised by about 1 m above the adjacent ground surface. The pad was placed over native undisturbed stiff subgrade silty clay, after the removal of any pre-existing fills, or softer soils. The total thickness of the granular pad under the tank was not less than 2.1 m.

Three pairs of twin-tube pneumatic piezometers were installed in individual holes drilled at three depth levels of 4.6 m (15 ft), 10.65 m (35 ft) and 19.8 m (65 ft) below the tank base using solid stem augers. The location of the piezometer was selected within a 1.5 m (5 feet) radius from the tank center. Twining of the piezometers was decided as a mitigation of potential malfunctioning of some of the gauges.

Prior to placement of the steel floor plates, a 100 mm diameter PVC pipe was installed in a shallow trench cut within the granular pad along a tank diameter to accommodate the sensor of the “Consoil” hydrostatic profiler (Fig. 16) used to measure the deflection of the tank base.

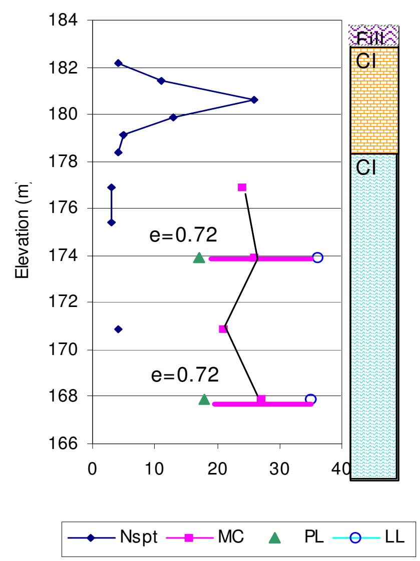

Fig. 14. Tank ES600 Borehole Information

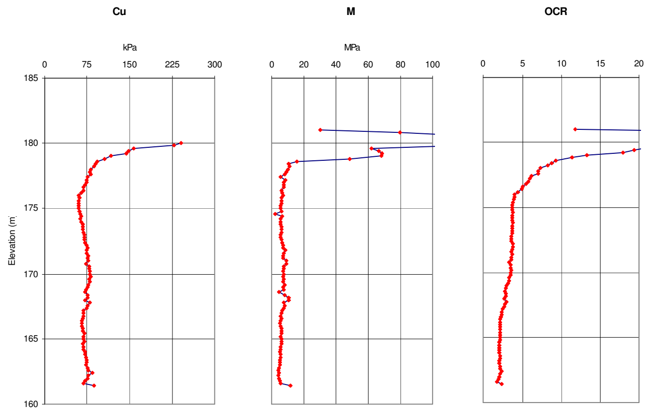

Fig. 15. Tank ES600. Summary of DMT Data

A controlled loading-unloading program was implemented. In essence, a variable waiting time under a given load was implemented based on the observed response of the porewater pressures. Moving up to a higher load was allowed when the porewater pressure rise generated by the current load step ceased to increase, or slowed down, or tended to decrease.

The readings at all 6 piezometers expressed in terms of total head in meters above the sea level are summarized in Fig. 17.

Fig. 16. PVC pipe, sensor and Consoil profiler setup

Almost consistently the pore pressures changed rapidly with the load changes, and kept creeping up for days, and up to two weeks after every load step increase was applied. This is readily visible at the deeper piezometers at 35 feet (10.6 m) and 60 feet (18.3 m) below grade, after the 14th of November 2005 when the maximum test load was attained; the pore pressures kept increasing for two weeks under constant total stress. Given the very low permeability of the soils (typically k = 10^{-8} cm/s), and the spacing of about 9 m between the piezometer horizons, it seems highly unlikely that the noted creeping up of the porewater pressures under constant total load is entirely caused by the mechanics of the water movement thru the soil pores under the imposed hydraulic gradients generated by the soil stressing. Rather, we tend to believe that the soil mass has crept under imposed stress increases, both compressive and more so in shearing.

The settlements shown in Fig. 17 would be the maximum settlements under the tank center, and were inferred from the

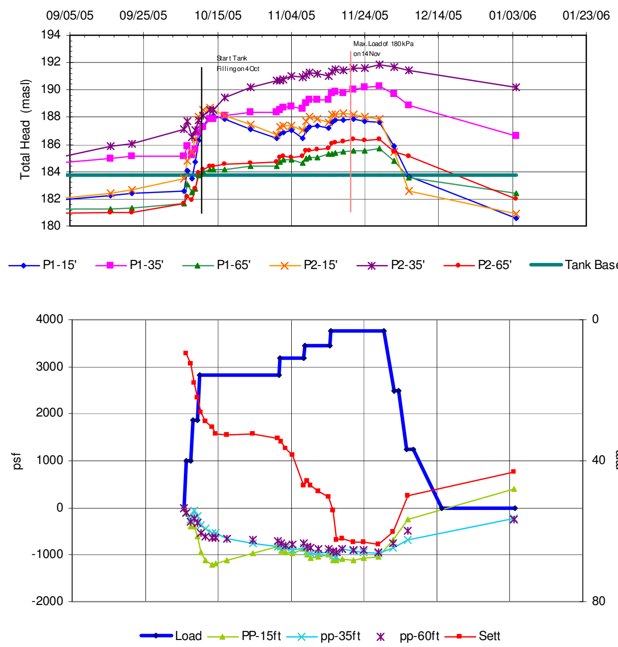

Fig. 17. Tank ES600 Piezometer readings, loading history and inferred maximum settlements during hydrotest

hydrostatic profiler readings. For convenience in the presentation, the porewater pressures were also expressed as piezometric pressure increases to help relate more directly to the applied total tank load. The pressures were all positive but have been plotted as negative values to reduce the congestion of the chart. The maximum recorded pore pressures reached about 1200 psf (about 60 kPa) which represent approximately 32% of the maximum applied tank test load.

As mentioned earlier, the tank base deflections were monitored using a hydraulic profiler installed within a shallow trench immediately below the steel base plate. A selection of the profiler readings is provided below (Fig. 18a). Based on these, the load-unload vs. settlement path under the tank center was inferred and plotted in Fig. 18b. . There seems to be quite a scatter of the measured deflections under the tank base, which should be explained mostly by the intrinsically coarse accuracy of this type of instrumentation. Nonetheless, the trends, and the orders of magnitude, seem to be quite consistent with the expectations.

A conventional rim survey was also completed during the hydrotest. The measured settlements at 16 stations along the perimeter are plotted in Fig 19. Station "0" was set at the north side of the rim, and like always, the numbering of the other station was in the clockwise direction.

Back calculation of the 'elastic' moduli on the basis of the recoverable rim settlements from the recorded maximum settlements yielded a much narrower range of values from 130 MPa to 185 MPa than was recorded at Tank E204. This may be explained by the longer waiting periods under each load step of days, and up to two weeks as opposed to only one day waiting at Tank E204. It can be argued though that the current

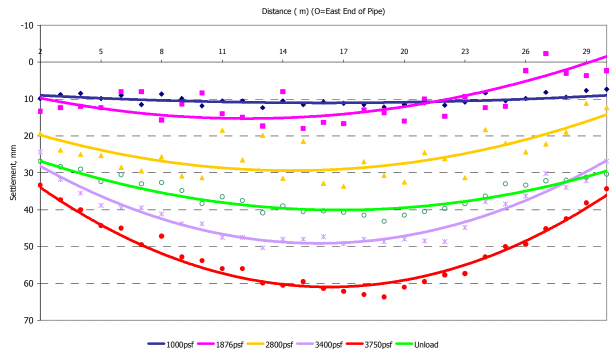

Fig. 18a. Tank ES600 Select tank base deflection readings at the hydrostatic profiler during hydrotest

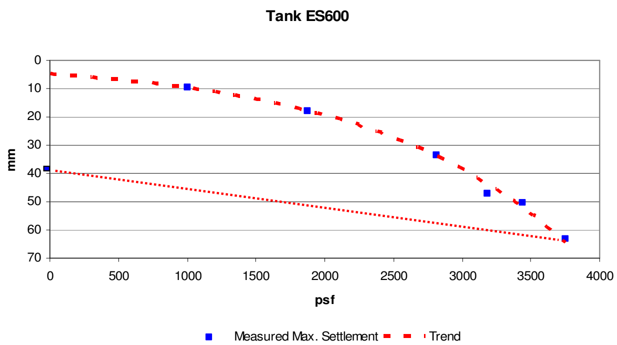

Fig. 18b. Inferred maximum and residual settlements below the tank center during hydrotest

“elastic” moduli are not so close to the true elastic response as in the case of Tank E204 exactly because of the noted creep and some consolidation components allowed at the present tank, given the increased waiting times at the different loading stages.

<u>Tanks ES800</u>

The tanks ES800A and ES800B were erected in 2006 as twin tanks within the same tank lot and placed on raised gravel pads. Similar to ES600, these tanks were subjected to an extended hydrotest program conducted along an almost identical pattern as for ES600. The instrumentation at both Tanks ES800s included the hydraulic profiling pipe under the tank base and 3 pairs of piezometers at three depth levels that in this case were established at 6.1 m (20 ft), 9.9 m (32.5 ft) and13.7 m (45 ft) below the tank base within a 1.5 m (5 feet) radius from the tank center.

The subsurface conditions under the tank were explored in a conventional borehole (Fig. 20) augmented with routine laboratory testing and a couple of consolidation tests, plus a DMT profiling (Fig. 21).

The load test history and the associated averaged porewater pressures along with maximum deflections inferred from the profiler readings are provided in Fig. 22. As before, the porewater pressures were plotted also as piezometric pressure increases to help compare directly to the applied total tank load. The porewater pressures were all positive but have been plotted as negative values to reduce the congestion of the chart. The porewater pressure increases for both tanks approached 1500 psf (72 kPa) which represents 40 % of the nominal maximum test load.

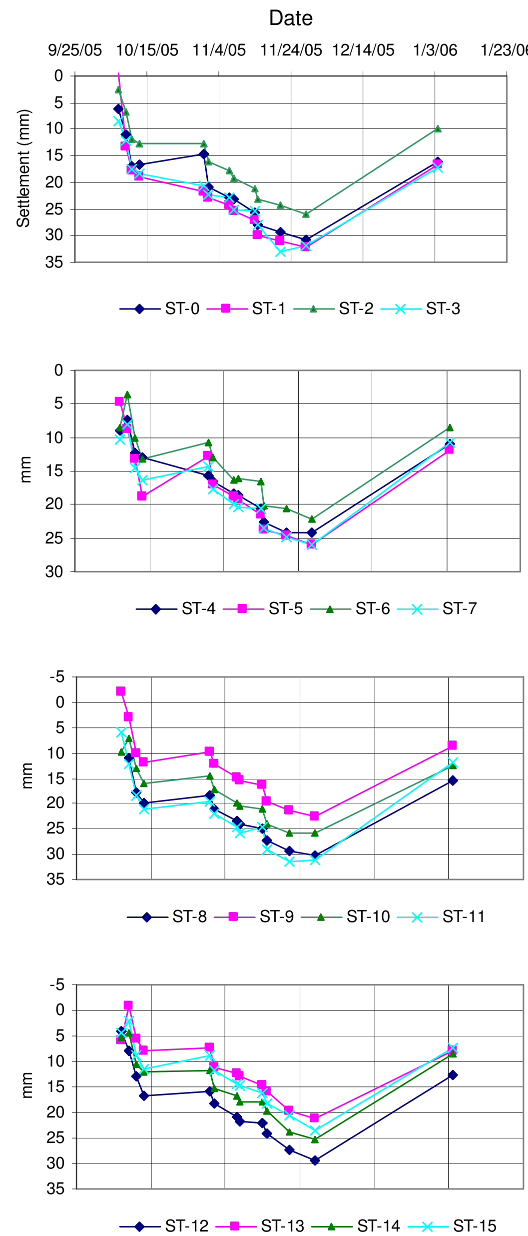

Fig. 19. Tank ES600. Rim survey during hydrotest

The settlements plotted in Fig. 22 were inferred from the maximum deflection readings at the hydraulic profiler. There were some issues with the profiler baseline reading at ES800B so that the settlements plotted for this tank are only the incremental values for the last 2/3 of the total hydrotest load.

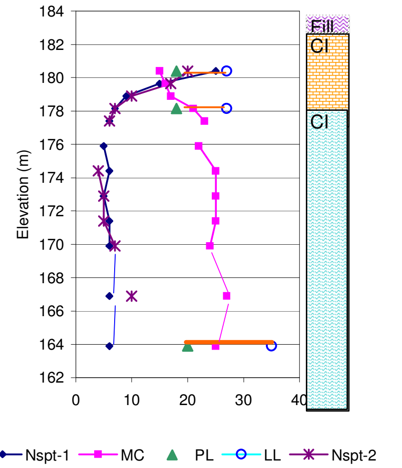

Fig. 20. Tanks ES800. Summary of Borehole Information

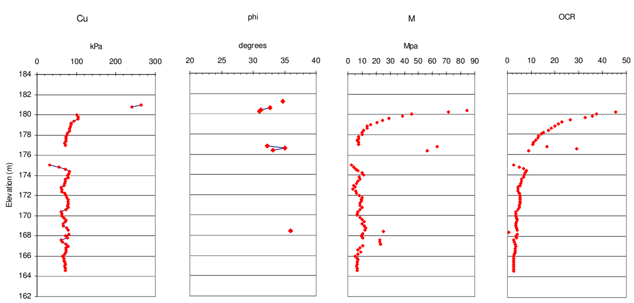

Fig. 21 Tanks ES800s. Summary of DMT data

The rim elevation survey at the two tanks is provided in Fig.23. Back calculation of the 'elastic' moduli on the basis of the recoverable rim settlements from the recorded maximum settlements yielded quite narrow ranges: of values from 87 MPa to 113 MPa for Tank ES800A and 107 MPa to 129 MPa for Tank ES800B. The similarity of these values with the results at Tank ES600 is notable. The same discussion raised for Tank ES600 about the unknown consolidation and creep component that is incorporated in these estimates for the elastic moduli applies to the current tanks.

The profiler readings are shown in Fig. 24. As mentioned earlier, there were problems to set a reliable base line for the profiler at Tank ES800B before the tank loading started. The first stable baseline was only when the tank load reached 1100 psf (53 kPa) and the subsequent profiler information was related to this origin. Obviously, the deflections which occurred before reaching this load step were lost.

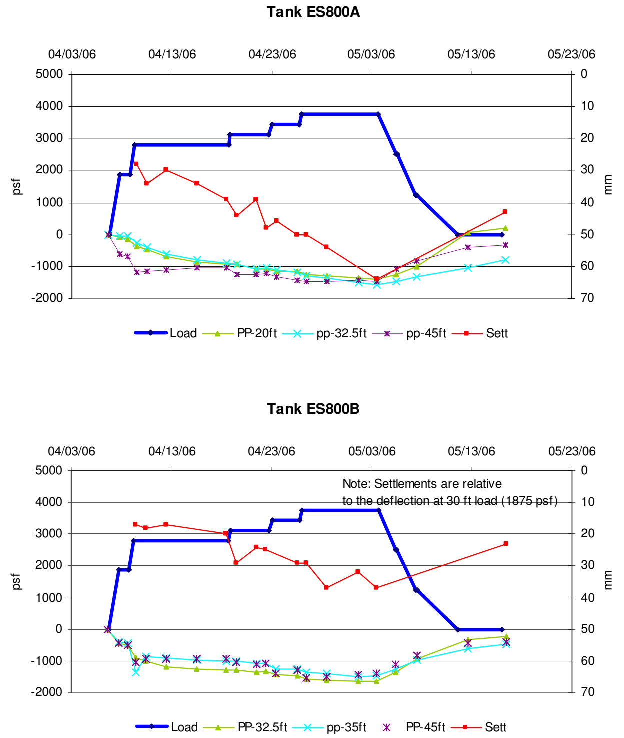

Fig. 22. Tank ES800s Piezometer readings, loading history and inferred maximum settlements during hydrotest

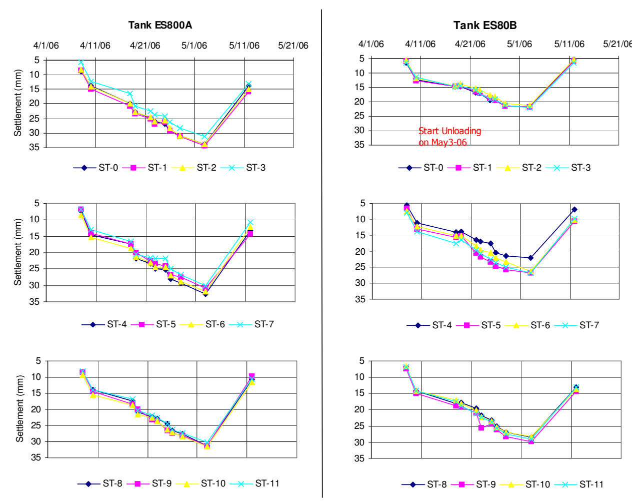

Fig. 23. Tanks ES800. Rim Settlement during hydrotest

At these ES800s series tanks an opportunity was available to read part of the instrumentation several months after the tanks had been in operation. Detailed records of the porewater pressures under the two tanks are provided in Fig. 25.

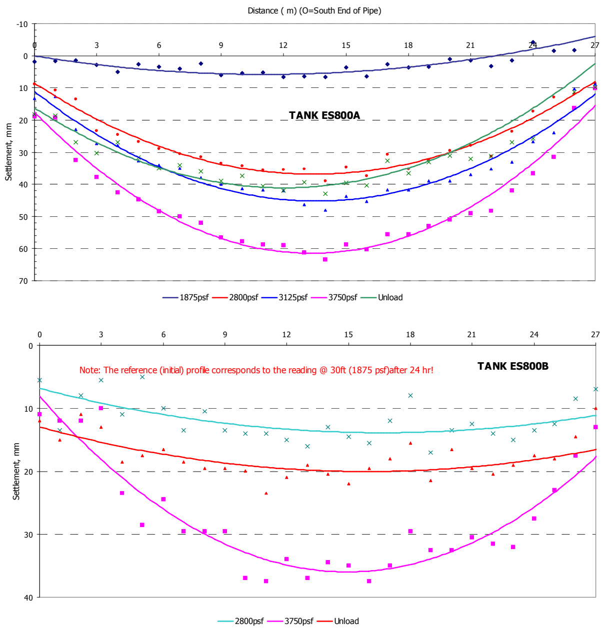

Fig. 24. *Tanks ES800 Select tank base deflection readings at the hydrostatic profiler during hydrotest*

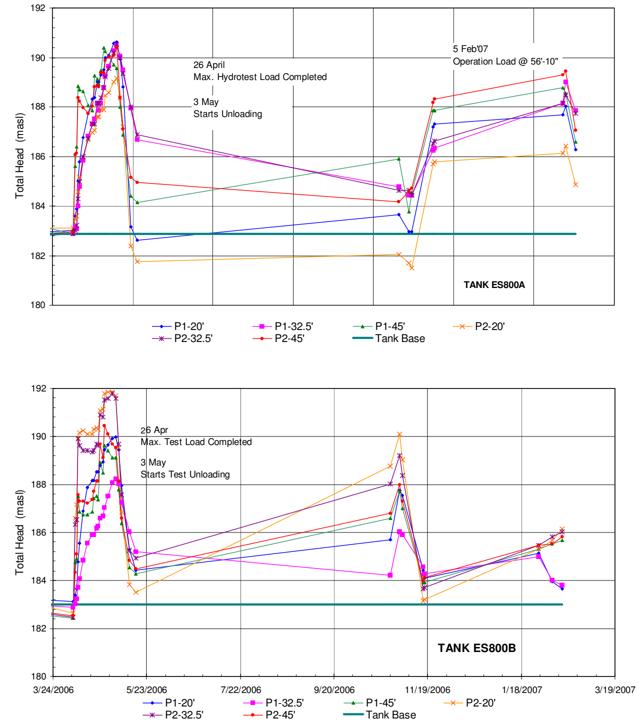

Fig. 25. Tanks E800s. Detailed porewater pressure records during hydrotest and operations

However, no accurate detailed information was available regarding the exact short history of operation of the tanks following the completion of the hydrotest in May 2006. Reportedly, at the beginning of November 2006 the product load in Tank ES800B was estimated to have reached the level of the previous peak load (3750 psf) of the hydrotest. Similarly, Tank ES800A reached this same load around 15 of November, 2006. Then, Tank ES800B started to be unloaded while more product was pumped in Tank ES800A with an unknown loading rate. The fact is that on February 5, 2007 the height of product in Tank ES800A reached 56 feet and 10 inches which should be equivalent to about 4100 psf (based on a specific gravity of the product that reportedly could be 1.1 to 1.15). This load level is believed by the Owners to be close to the maximum design load. In Tank ES800B the uncertainty about the actual operation loads at the time of readings in late October early November 2006 was even greater. Reportedly, this tank had not yet been filled to capacity.

It is notable that the recurrence of the porewater pressures at Tank ES800A was close to, but less than the peaks attained during the hydrotest. Since the operation load was larger than the hydrotest load, the fact that the peaks pore pressures were lower seems to confirm that a certain level of ground improvement has been achieved by the preloading during the hydrotest.

DISCUSSION

Table 3 summarizes the estimated settlements and the ranges of the measured settlements. The results of the calculations were all rounded to the closest 5 mm value. All the settlement calculations assumed the tank would be loaded permanently to the shown loads. The long term consolidation component of the settlements was estimated using the DMT based compression moduli, M. In all the cases, the total thickness of the compressive layer was assumed to be 40 m, where bedrock and / or dense gravelly till is usually present.

Since the DMT probes were shorter than the overburden thickness, a projection with the depth of the relevant soil characteristics (compression moduli and the undrained cohesion) was assumed considering a slight rate of increase of 0.125 MPa/m for the compression modulus, and 2.5 kPa/m for the cohesion. Such increases should be reasonably conservative, and seem to be consistent with the prevalent near normally consolidated condition of the deeper portion of the silty clay overburden.

The use of the DMT in the geographical region proved to be useful and reliable in many projects involving large industrial type of loads and earthworks. Notwithstanding, conventional soil testing was also conducted to crosscheck the DMT results.

The elastic settlements were estimated using the assumption that the elastic (Young) modulus is 1000 times the undrained cohesions. Where non-cohesive seams were encountered, the elastic modulus was taken equal to the compression modulus.

The settlements were calculated using the Boussinesq stress distribution under the center, C, of the tank. The settlements

Table 3 Summary of Calculated and measured Settlements

Tank#	Diameter (m)	Load (kPa)	Depth DMT (m)	SIBTDMT (mm)	Calculated Settlements (mm)				Measured Settlements (mm)			Remarks
					C	R	C	R	R	C	R	
203	41	143		95	350	225	40	25	175-280			Survey 58 to 03
204	41	143		95	350	225	40	25	35-55		0-21	Survey 59 to 63 & HT-03
208	46	143	27.4	106	375	240	40	25	50			Survey 89 to 03
209	46	150		110	395	255	45	30	75-125			Survey 89 to 02
214	46	143		105	375	240	40	25	50-100			Survey 89 to 06
ES600	27.5	180	21.5	135	390	250	40	25		26	13-19	Hydrotest 2005
ES800A	21.3	180	17.2	100	270	170	40	25		20	17-22	Hydrotest 2006
ES800B	21.3	180								17	14-17	Hydrotest 2006

SIBTDMT =Settlement Inferred Below the Tip of DMT

Table 4 Basic compressibility characteristics from routine and consolidation laboratory tests for different projects

Ground Elevation	Sample Depth (m)	Sample Elevation ((masl)	Overburder Pressure (kPa)	OCR	Specific Gravity	Insitu Void Ratio	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Unit Weight kN/m3	Compression Index	Recompression Index	Cv @ Po m2/day	Site Location
182	3.35	178.65	66	2.6	n/a	0.52	20	37	17	21.8	0.2	0.03	0.022	Sarnia-Vidal St
187.5	3.5	184	60	6.7	2.72	0.45	16	25	14	21.4	0.12	0.028	0.086	Sarnia-St.Clair Pkw
181.6	4.6	177	67	3	2.72	0.63	23	28	15	20.4	0.21	0.07	0.013	Sarnia-Vidal St
182	4.6	177.4	67	3.7	2.75	0.63	23	28	14	20.5	0.21	0.03	0.028	Sarnia-Vidal St
181.6	9.1	172.5	120	2.9	2.72	0.7	26	29	17	19.8	0.21	0.035	0.042	Sarnia-Vidal St
182	9.1	172.9	120	1.75	2.75	0.71	26	33	16	20	0.21	0.05	0.03	Sarnia-Vidal St
182.9	9.1	173.8	115	5.3	2.77	0.72	26	36	17	19.6	0.29	0.07	0.03	Sarnia-Vidal St
194.7	9.5	185.2	125	2.25	2.72	0.51	16	41	21	20.5	0.16	0.06	0.015	Corunna
182	9.5	172.5	125	1.35	n/a	0.75	28	43	18	20.5	0.23	0.04	0.04	Sarnia-Vidal St
186.5	9.5	177	125	3.2	2.74	0.82	30	38	18	19	0.3	0.07	0.086	Sarnia-St.Clair Pkw
182	10	172	120	2.9			23	32	15	20	0.21	0.02		Sarnia-Downtown
182	12	170	140	2.1		0.85	30	41	11	19	0.34	0.06		Sarnia-Downtown
194.5	12.2	182.3	155	26			27	39	19		0.25	0.08	0.015	Corunna
182.9	15.2	167.7	175	2.6	2.67	0.72	27	35	18	19.6	0.41	0.03	0.06	Sarnia-Vidal St
195.1	16	179.1	180	2.75	2.7	0.75	28	43	21	19	0.33	0.08	0.018	Corunna
187	21.5	165.5	230	1	2.5	0.8	28	39	20	19.4	0.23	0.06	0.012	Sarnia-Indian Rd. S.
194.9	24.4	170.5	270	1			35	35	19	18.3	0.28	0.08		Corunna
195	30.5	164.5	300	1			27	44	19	18.4	0.25	0.06		Corunna

Cv = coefficient of consolidation at the overburden pressure, Po

at the rim, R, were estimated simply as about 64 % of the center settlements.

It is worthwhile to note that all the long-term measured settlements most certainly reflect an incomplete process of consolidation and hence, the fact that they are consistently lower than the calculated settlements seems to be a natural outcome. The fact that the calculated elastic settlements under short-term conditions seem to be consistently larger than the measured values most likely originates mostly from the arbitrary choice for the elastic modulus used for the calculations. The back-calculations from the hydrotests suggested that the elastic moduli are likely larger than 1000 times the cohesion (as inferred from the DMTs).

The conventional compressibility characteristics of soil samples collected from several projects in the vicinity of the subject tanks are presented in Table 4 . The sorting of the data in Table 4 was made on the basis of the sample depth which is one of the determining factors of the soil behavior under foundation loads. However, since the samples were retrieved from locations as much as 20 kilometers apart, some apparent discrepancies should be expected between some of the characteristics in comparison with the assumed typical trends associated with the level of the current overburden pressures.

In principle, the tank loads could be evaluated just as accurately as for other type of structures, if not better. But unfortunately, the actual loads seem to fluctuate frequently within large limits and with virtually untraceable time histories. Giving due considerations to such unpredictable factors, to the relative scarcity of the quantitative records and surveys, and to the relatively low accuracy of the measurements, it can be stated that the order of magnitude of the recorded settlements, both for the long-term and short term conditions seems to match quite satisfactorily the estimates based on conventional soil testing and practical engineering methods.

With regard to the instrumentation, from other projects, it was often proven that nesting piezometers in the same boreholes carries distinct risks of direct hydraulic communication. Considering the incremental costs, once the drilling equipment is on site, it is worthwhile to install the piezometers in separated holes. Also, if possible, backup piezometer should be considered. At these particular tank projects the piezometers at different elevations showed distinct hydraulic separation while the piezometers the same depths responded almost identically to the applied load, which increases the reliability of the collected readings. The piezometer installation in such types of low permeability glaciolacustrine silty clays should be made well in advance of the planned loading.

The profiler sometimes gave inconsistent results, but this could be “blamed” as well on the accuracy (or lack thereof) of the benchmark survey, on weather or temperatures, etc. It is highly crucial to establish a reliable baseline, and for this purpose it is highly advisable to take several sets of readings at various times in the day, in different days, and if possible, by different operators.

CLOSING REMARKS

The Geotechnical Engineer is continually challenged to apply theoretical principles to achieve practical construction solutions. The observations described in this paper point to the fact that the actual long-term settlement of storage tanks during their operation life may be less than a theoretical analysis would indicate. One principal reason for this fact is believed to be the nature of the usual cyclical loading of a tank.

Nevertheless, it was shown that differential settlements well in excess of 100 mm, if not over 200 mm under the thank shells are the norm. When this prediction is reported to designers and owners there are frequent responses that this would create problems with pipe connections while records of actual problems are few. It can be surmised that such structures and their connection lines have a quite large tolerances and can accommodate unusual levels of settlements. Also, for other maintenance reasons, pipes and valves are sometimes replaced before settlement problems have registered.

It is apparent that the tanking industry becomes more and more regulated with respect to the maintenance and overall performance. The mandatory hydrotest protocol for new and retrofitted tanks is a prime example. However, in practice it is not apparent that the completion of periodic surveys, beyond those taken during hydrotests, is mandatory. There would be great economic benefits for the industry if this type of settlement tracking would be done. It would assist in scheduling appropriate and more strategic retrofitting.

Part of the experience reported in this paper seems to suggest that, the benefits of the hydrotest, which till now is essentially a leak/distortion test of the steel shell and bottom, can at the same time become a valuable geotechnical tool. With less than modest instrumentation costs and almost negligible interference with the site program, the hydro test can be turned into a full scale load test confirming the geotechnical performance predictions. Moreover, the geotechnical monitoring of the hydrotest presented herein, suggests the potential for extending the hydrotest as a means to inducing a certain level of rapid provable ground improvement, at least thru a form of strain-hardening mechanism in instances when improvement thru consolidation is not an option because of the necessary length of time. Thus, the permissible height of a tank can be safely maximized with little additional engineering expenses.

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