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ABOVEGROUND STORAGE TANK FOUNDATION REINFORCEMENT DURING CONSTRUCTION

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Abstract: Aging infrastructure and requirements for additional capacity necessitated the replacement of an existing 30 m diameter tank with a 55 m diameter tank with a floating roof in an area of Regina with significant foundation design challenges. The tank was to be founded on high plastic clay ranging in thickness from 6 m to 12 m underlain by a silt and sand aquifer, with local restrictions limiting the depth of development to 6 m below ground surface. Based on these restrictions, it was proposed that the tank be supported on a gravel ringwall foundation. Foundation plans and records were not available for the existing tank. During demolition, it was discovered that the existing tank was founded on piles. The piles were cut off 3.5 m below surface and backfilled with material from stockpiles within the terminal. This material had questionable properties for support of the tank: it contained construction debris and had variable moisture contents and densities. Due to concerns regarding the ability of these soils to support the original design, a cone penetration testing (CPT) program was completed to gather additional information and compare the properties of undisturbed material and the material within the foundation backfill zone. Proof-rolling was also completed to identify localized soft spots. The design team needed to ensure that the tank foundation performed as originally designed with minimal changes to the overall design. A cellular confinement system and layers of geogrid were added into the design of the ringwall to increase the bearing resistance and reduce the potential for significant differential settlements. The addition of geosynthetic reinforcement within the foundation and underlying soils allowed the overall design to perform as originally proposed – verified with hydrostatic testing settlements correlating with original design predictions – and construction to progress with minimal removal of the pile backfill material.

1 Project Background

Aging infrastructure and requirements for additional storage capacity necessitated the replacement of an existing 30 m diameter aboveground storage tank. With a diameter of 50 m and a height of 19 m, the new tank would provide an increase in storage from approximately 100,000 bbl to 250,000 bbl.

Early proposals for the new tank location included constructing the tank on an unused portion of the facility, but due to additional infrastructure requirements, which included a self-sufficient fire suppression system, as the location would be in a previously undeveloped area of the terminal, the costs associated with these proposals exceeded the benefits. In addition to the fire suppression system, a rail line crossing from the previously developed area of the terminal to the proposed tank location required more intensively designed pipeline infrastructure, as the rail line would have to remain operational during all construction.

A preliminary review of available background information about the site conditions did not yield significant information, as the foundation design for the existing tank was unavailable. Review of previous tank inspection reports and visual inspections indicated that the existing tank was supported on a steel I-beam ringwall foundation, but no other information was available at the time the geotechnical investigation was coordinated for the new tank.

At, the time of the investigation, the tank was still operational, which further limited the gathering of information. Access to the site was limited due to buried infrastructure, and information under the footprint of the existing tank could not be collected.

2 New Tank Foundation Design

2.1 Design Specifications

For aboveground storage tanks, the American Petroleum Institute (API) specifications API 650, *Welded Tanks for Oil Storage*, and API 653, *Tank Inspection, Repair, Alteration, and Reconstruction*, provide industry standards for the design of new tanks and design considerations for long-term performance of existing tanks, respectively. The definitions of settlement criteria for long-term performance are outlined in Annex B of API 653 and are broken down into three main categories: uniform settlement, rigid body tilting of a tank (planar tilt), and tank shell settlement, with additional areas of localized settlement, such as edge settlement and bottom settlement of the tank floor (API 653, 2014).

Uniform settlement is defined as the minimum amount of settlement the tank undergoes and assumes that the tank moves uniformly as a rigid structure.

Rigid body tilting of a tank (planar tilt) is defined as the rotation the tank undergoes from differential settlement, assuming the tank moves uniformly as a rigid structure. This component is more commonly used for tank shell design.

Tank shell settlement is defined by measuring set points at a maximum 32 ft. (10 m) spacing around the circumference of the tank (API 653, 2014). This component is used to determine stresses within the tank shell as well as to determine the performance of the foundation settlement and identify differential settlement around the tank. These measurements are also adopted for hydrostatic testing.

Excessive uniform settlement is a concern: as the tank settles, connected piping will be stressed, and it can also produce gauging inaccuracies. Differential settlement under the tank is an additional concern, as it will stress the tank shell/bottom locally and can force the tank out-of-round, affecting roof seals and other operating functions of the tank. Additionally, as the tank utilizes a floating roof system, excessive differential settlements can affect the performance of the roof.

API 650 provides designers with recommendations and considerations for foundation design as well as design tolerances and the maximum settlements a tank can undergo during hydrostatic testing (short term). The specification dictates that “any differential settlement greater than 13 mm per 10 m (½ in. per 32 ft.) of circumference or a uniform settlement over 50 mm (2 in.) shall be reported to the Purchaser for evaluation. Filling of the tank shall be stopped until cleared by the Purchaser.” (API 650, 2013). This has been adopted as the standard method for determining the performance of a tank foundation during hydrostatic testing.

2.2 Proposed Foundation Design

Based on the results of the geotechnical investigation and the calculated bearing resistance of the site soils, it was determined that the new tank could be founded on a gravel ringwall foundation system. While high settlements were predicted beneath the proposed new tank (shell settlements between 75 mm and 140 mm and tank centre settlements between 175 mm and 250 mm) due to the high plastic clay soils underlying the foundation, as long as the supporting infrastructure could be designed to withstand the settlements predicted, the system would allow for significant cost savings over other foundation systems (e.g. pile supported) or ground improvements. Local development restrictions were an additional consideration that did not support using a pile foundation; these restrictions limited the depth of disturbance to 6 m below ground surface due to a regional aquifer.

The proposed foundation system underlying the tank shell consisted of a 2.5 m top width, 1.5 m high gravel berm with a 1.5 horizontal to 1 vertical slope. Underlying the tank floor would be a 1 m thick gravel layer.

All of the gravel material underlying the tank shell and floor was specified as 75 mm crushed gravel. The gravel ringwall design also incorporated the leak detection and cathodic protection systems.

Based on the layout of the proposed new tank foundation, the footprint of the existing tank would be entirely within the floor and would not impact settlements under the tank shell, only the tank floor, which is a more flexible system.

3 Existing Tank Foundation

During the demolition of the existing tank and foundation, it was discovered that the steel I-beam ringwall was supported on cast-in-place concrete piles that were not visible during previous inspections. Upon further research, the records of the existing tank were found, and it was discovered that the tank had settled between 175 mm and 275 mm early in its life, higher than anticipated during design. Due to the excessive settlement, the tank had to be jacked up, and a pile-supported foundation was installed to control the settlement that was occurring.

Details of the pile foundation indicated that there were 36 cast-in-place concrete piles and pile caps around the circumference of the existing tank. As the existing tank was still intact at the time of the foundation remediation, the piles were constructed outside of the perimeter of the tank, with the pile cap extending inward to support the tank shell. The I-beam ringwall was then placed on the pile caps to support the tank shell.

4 Construction Impacts

4.1 Existing Foundation

Following the discovery of the pile foundation beneath the existing tank, concerns were raised over the impact of the previous foundation system on the proposed gravel ringwall foundation system of the new tank. The original foundation design for the new tank took into consideration the consolidation that had occurred beneath the existing tank, assuming that the soil had consolidated uniformly. However, the piles beneath the footprint of the new tank would introduce hard spots: localized areas beneath the tank where soils would not consolidate the same as the soils surrounding the piles under the new tank loads. The impacts of the “hard spots” (i.e. leaving the pile caps and full pile length in place) were assessed by the design team, it was determined that leaving them in place would prevent the new tank from settling as originally predicted which would affect the performance of floating roof and other appurtenances.

The depths of the piles were unknown, as this information was not included in the design documents. It was assumed, based off local development restrictions, that the piles would have likely extended to approximately 6 m below existing ground.

4.2 Foundation Removal

In order to ensure the new tank performed as originally designed, a plan was formulated to remove the piles to remove the hard spots beneath the tank footprint. With the pile depth assumed to extend approximately 6 m below existing ground surface, the proposed plan was to remove the pile caps at each location, excavate to 3 m below grade, and, using an excavator, attempt to break the piles free. A trial attempt was carried out to remove a pile at one location to calibrate this plan. The results of the trial indicated that the piles could not be removed to their full depth, leaving the full depth of the piles still undetermined.

In order to move forward without creating an excessive excavation on the site, a review was completed on the impact of leaving a portion of the piles in place. It was determined that the piles could be excavated to a depth of 3.5 m and cut off, with the remaining portion left in the ground.

4.3 Foundation Backfill

The excavation to remove the piles was seen as an opportunity to re-establish a condition similar to a previously undisturbed state beneath the footprint of the new tank; this could potentially reduce the differential settlements between the undisturbed areas and the previously consolidated areas. To attempt to return the soil to a condition similar to that outside of the previously loaded area, density testing was carried out using a small test-pitting program and a nuclear densometer to determine in situ dry density and moisture conditions of the clay material at various depths and locations around the previous tank foundation. The results of the testing indicated that an average maximum dry density of 1,530 kg/m^3 with an average moisture of 24% would represent the native conditions surrounding the existing tank.

Upon completion of the removal of the piles, the excavation was 1.5 m wide at the base with side slopes of 1H:1V and extended to a maximum depth of 3.5 m extended around the perimeter of the previously existing tank. This was to be backfilled using native material excavated from the pile removal process and additional stockpiled material (also native to the site) as needed.

4.4 Backfill and Compaction Issues

During the backfill process, a third-party testing firm was retained to complete compaction quality control. As backfill progressed, the testing firm noted a large variation in the density and moisture content of the backfill material, and through proctor testing demonstrated that the dry densities and moisture contents of the various site materials varied greatly. Grain-size analysis and Atterberg limits testing were also performed on representative samples of the backfill materials. Concerns were raised over the quality control of the backfill process as well as the quality of the backfill material itself. It was noted in the compaction records and laboratory testing completed on this material that the sand, silt, and clay contents varied significantly in the stockpiled material and that it occasionally included gravel and deleterious materials.

As work continued on the backfill, it was determined that there was a broad range of moisture content and dry density values in the backfill material, which was not consistent with the proposed backfill plan and design assumptions. Moisture contents ranged from 15% up to 40%, and dry density values ranged from 1,430 kg/m^3 to 1,800 kg/m^3. A review of the compaction results called into question the ability of the soils to support the foundation as it was originally designed and the impacts it would have on the predicted settlements of the tank.

4.5 Tank Footprint Shifted

As the design of the tank progressed and work was carried out on the demolition of the existing tank, space constraints within the tank lot resulted in the footprint of the new proposed tank being from its original location. As a result, a portion of the new tank shell overlapped a portion of the old tank foundation. This change resulted in a more significant portion of the new tank ringwall foundation overlapping the excavation.

4.6 Backfill Material Assessment

As questions arose over the ability of the backfill material to support the tank foundation as it was designed, methods to assess the condition of the backfill material were proposed. Some of the proposed methods included completing further test pitting to assess the material at different locations and depths, completing a supplementary geotechnical investigation with a drill rig, completing a supplementary geotechnical investigation using a cone penetration testing (CPT) rig, or removing and replacing all of the questionable materials and replacing them with controlled imported fill. It was determined that a CPT program would provide the most cost-effective method for determining the in situ conditions with minimal disturbance within the tank lot.

4.6.1 Cone Penetration Testing (CPT) Program

Cone Penetration Testing (CPT) consists of advancing an instrumented cone through soil. The use of CPT would allow on-site geotechnical engineers to review live results of the conditions, enabling them to alter the program as needed to target potential high-risk or questionable areas. The CPT cone used for this investigation recorded data every 2 cm as the cone was advanced and provided live results of tip resistance, sleeve friction, and pore pressure to the operator. This data is recorded, and an interpretation of the site condition is provided by the contractor, typically within one week of the field program completion.

In total, 19 locations were identified for CPT around the tank lot in order to assess the condition of the backfill materials in relation to the undisturbed materials outside of the backfill footprint. Locations were completed in both undisturbed and disturbed locations in order to complete a direct comparison of the CPT results.

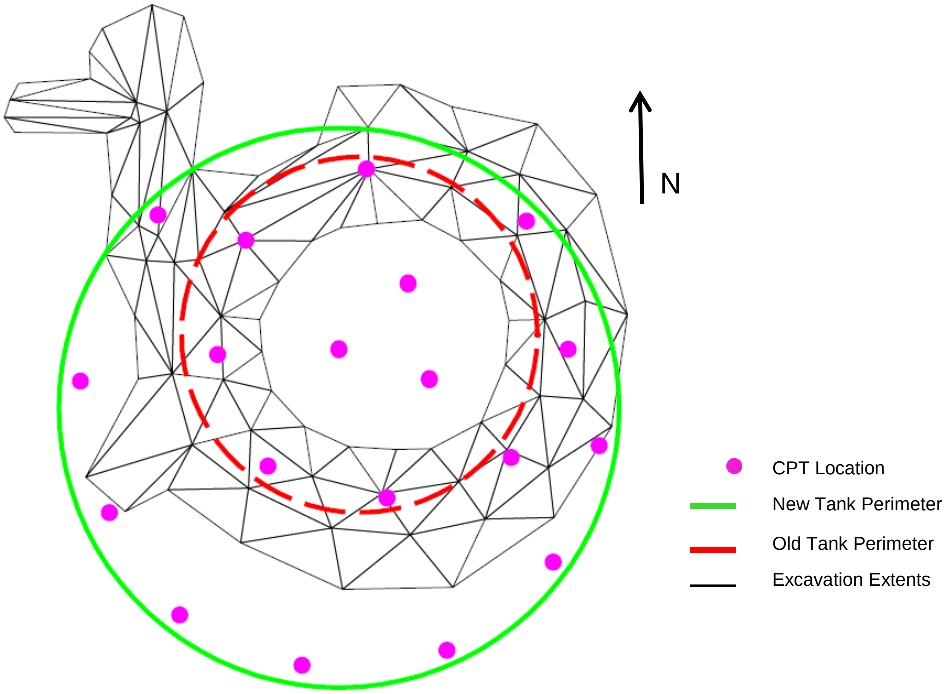


Figure 1 – Cone Penetration Testing Locations

In Figure 1, above, the pink dots indicate the locations of the CPT soundings, the red dashed line indicates the location of the demolished tank ring, the green line indicates the location of the new tank ring, and the polygon lines indicate the extents of the excavation to remove the previous tank foundation. The polygon extending north in the upper left corner is a pipeline that tied into the decommissioned tank. Another pipeline on the east side of the tank was also removed, but it is not shown in the attached drawing as it was mostly within the existing excavation and extended outside the proposed new foundation area.

CPT soundings were advanced to depths between 10 m and 15 m below existing ground surface in order to capture the upper 3.5 m of backfill material where present as well as the underlying native materials. Additionally, select CPT soundings were advanced to practical refusal (exceeding the capacity of the CPT rig) in order to collect data from beneath the existing tank, which was not obtainable at the time of the original investigation.

The CPT program was completed using a 25-ton truck unit, which due to the weight of the drill rig was capable of providing sufficient thrust to reach beyond 15 m without having to anchor the unit. The use of this rig allowed the investigation to be completed in two days, with preliminary results provided within two days of completion and final results within five days of completion. This allowed data analysis to commence in a relatively short timeline, limiting the impact on the overall construction schedule.

4.6.2 Results of CPT Program

The results of the CPT soundings provided information such as the soil behaviour type (SBT), which provides a correlation to the soil type based on the soil's reaction to the advancement of the cone, as well as additional information that can be correlated to determine in situ soil consistency and strength.

In order to determine whether the backfill materials were suitable to stay in place or if they required removal and replacement, an overlay of the CPT interpretations was plotted to compare all results in one location. Upon reviewing the results, it was determined that, in general, the upper 3.5 m material within the excavated area did not vary greatly from the surrounding materials. The CPT soundings did indicate that there were localized soft areas within the foundation zone and that some reinforcement would be required to ensure the soils and ringwall would perform as originally designed.

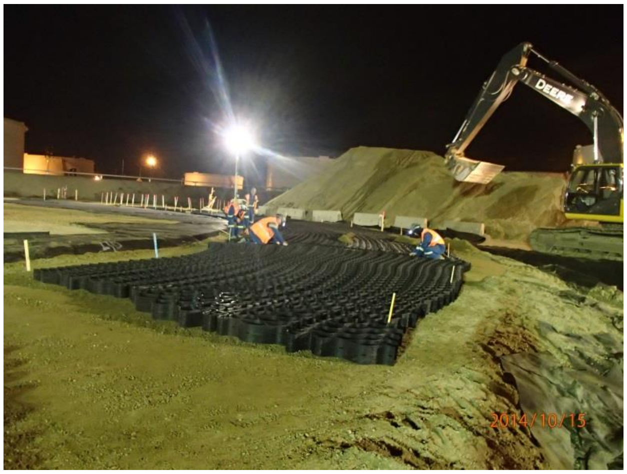
In addition to facilitating the assessment of subsurface conditions, the investigation using CPT also provided additional data that could not be obtained during the original investigation and allowed for an updated prediction of settlement around the tank ringwall and within the centre of the new tank.

5 Revised Foundation Design

Upon review of the CPT program and the compaction records for the site, it was determined that the gravel ringwall foundation could still be used but would require additional reinforcement to ensure it would perform as originally designed. Due to the condition of the backfill, it was determined that localized areas around the tank would likely result in small areas of settlements due to pockets of soft material. In order to ensure that the soils underlying the foundation had sufficient bearing resistance to support the tank, it was determined that a combination of geosynthetic materials installed within the gravel ringwall and foundation soils could be used to reinforce the soils and help distribute the loads more uniformly.

Upon completion of the CPT program, and once construction on the clay liner recommenced, proof-rolling of the tank ring was carried out to identify any soft areas. A fully loaded rock truck was used for the proof-rolling. Upon identifying soft areas, the extents were marked out and the soft soils were excavated to more competent soils below. If the soft soils extended too deep, the excavation was built back up using layers of geogrid to add additional strength to the backfill soils. The backfill materials used for the soft-area repair consisted of imported clay till material that was more consistent than the stockpiled materials on-site.

In order to add strength to the gravel ringwall foundation and to help it bridge soft areas that may have remained within the backfill areas, a geosynthetic cellular confining system (GCCS) was added to the design. GCCS is a system typically associated with construction of roadways over soft, compressible materials. The use of GCCS allows the load to be spread out more evenly, bridging soft areas and distributing the loads to stronger materials below. The new ringwall design incorporated a 200 mm thick layer of gravel at the base and a 150 mm thick GCCS system filled with gravel followed by additional gravel with a layer of biaxial geogrid placed 200 mm above the GCCS. The critical addition to the design of the tank foundation was the GCCS, allowing the foundation to transfer loads more uniformly. The design of the GCCS allow the tank foundation to bridge any soft areas that may have remained upon completion of the proof-rolling. The overall new design allowed the dimensions of the ringwall, including the leak detection and cathodic protection systems, to remain unchanged, resulting in significant cost savings when compared to a redesign of the tank foundation.. Construction of the ringwall foundation was carried out in fairly rapid succession upon completion of the CPT program and resulted in minimal impacts to the construction schedule in comparison to other remediation measures proposed. Construction of the ringwall is shown below in Figure 2, Figure 3, and Figure 4.



2014/10/15

Figure 2 – Crews Installing Geosynthetic Cellular Confining System (GCCS)

2014/10/16

Figure 3 – Placing Gravel within Geosynthetic Cellular Confining System

2014/10/18

Figure 4 – Biaxial Geogrid Placed above Installed Geosynthetic Cellular Confining System (GCCS)

6 Performance of the Revised Ringwall Design

As stated previously, the additional CPT program allowed the design team to provide updated predictions of the differential and total settlements of the tank foundation for both short-term and long-term scenarios. Settlement of the foundation system during hydrostatic testing and long-term operation gives the best indication of tank performance relative to design. The revised predicted settlements were provided to the design team to ensure that they were still within the tolerances of the overall design and that they met API 650 and 653 specifications prior to moving forward with the construction of the tank ring.

Following completion of the tank system and construction of the tank shell, hydrostatic testing was carried out in May and June of 2015. Elevations were collected at 16 points around the tank perimeter to gather baseline values for use in comparison to the various stages of hydrostatic testing: 25% full, 50% full, 75%

full, 100% full, a 24-hour hold at 100% full, and again once the tank was emptied (API 650, 2013). The estimated settlements at these locations as well as the measured settlements collected during the hydrostatic testing are provided below in Table 1.

Table 1 – Predicted and Measured Settlement Comparison (Hydrostatic Test)

CPT Program Settlement Estimation			Surveyed Settlement Data		
CPT Location	Approximate Location on Tank Perimeter (North is 0°)	Predicted Settlement (mm)	Station	Location on Tank Perimeter (°)	Measured Settlement at 24-Hour Hold (MacWhirter 2015) (mm)
40016 (Centre)	N/A	90 ± 5	N/A	N/A	
40018	0	30 ± 5	1	0	41
N/A	22.5	30 ± 5(1)	2	22.5	37
40006	45	30 ± 5	3	45	35
40006	75	30 ± 5	4	67.5	38
40014	90	35 ± 5	5	90	36
40013	115	45 ± 5	6	112.5	44
N/A	135	45 ± 5(1)	7	135	46
40012	170(2)	50 ± 5	8	157.5	48
40011	185	50 ± 5	9	180	48
40010	210	50 ± 5	10	202.5	44
N/A	225	45 ± 5(1)	11	225	41
40009	235(2)	40 ± 5	12	247.5	38
40008	270	35 ± 5	13	270	41
N/A	292.5	35 ± 5(1)	14	292.5	46
40007	315	30 ± 5	15	315	49
N/A	337.5	30 ± 5(1)	16	337.5	45

(1) Values were interpolated between stations if testing had not been completed at within 10° of the location

A comparison of the settlement profile around the tank is provided below in **Error! Reference source not found.**. This provides a side-by-side comparison of the settlement profile at various stages of the hydrostatic testing to review uniform, planar tilt, and differential settlements around the tank to compare against the tolerances specified earlier.

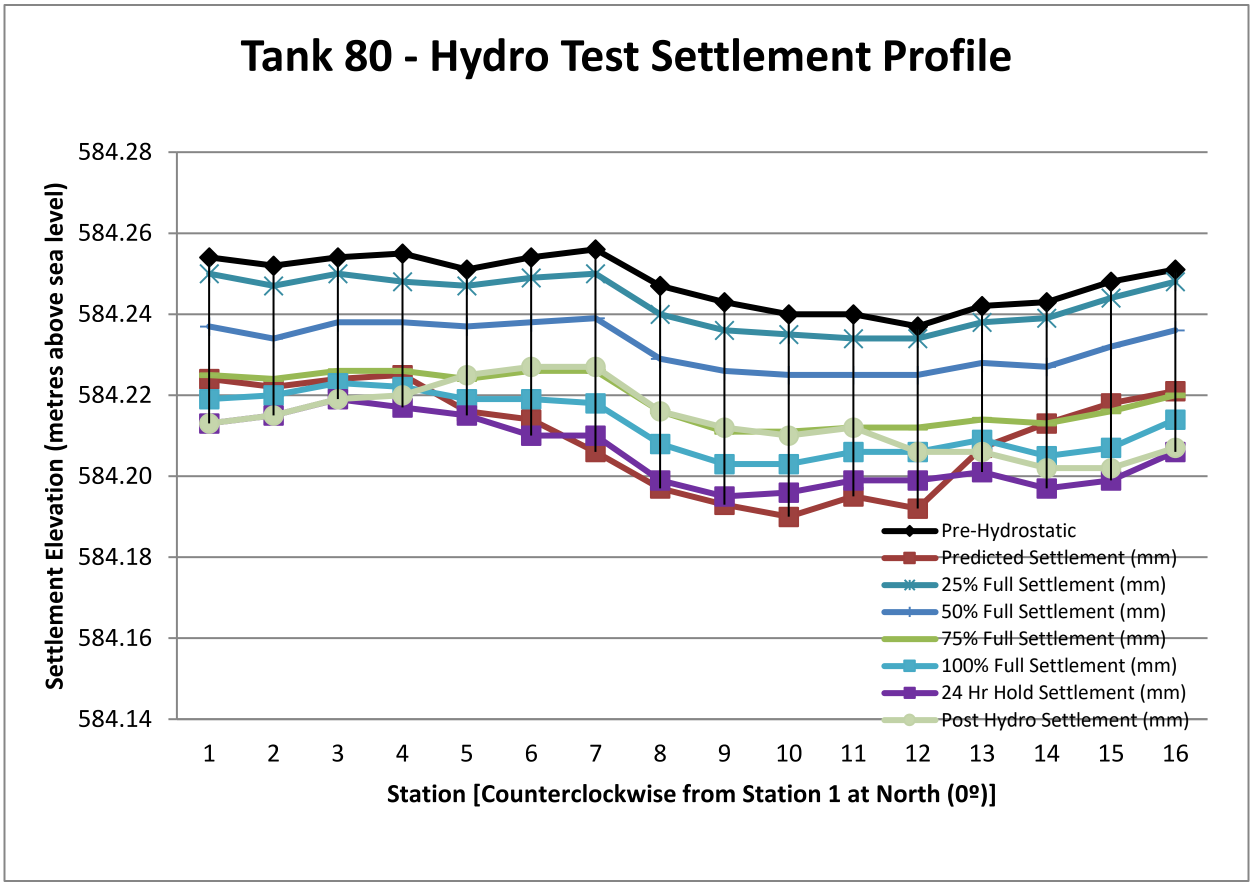


Figure 5 – Hydrostatic Test Settlement Profile: - Estimated and Measured Values (MacWhirter, 2015)

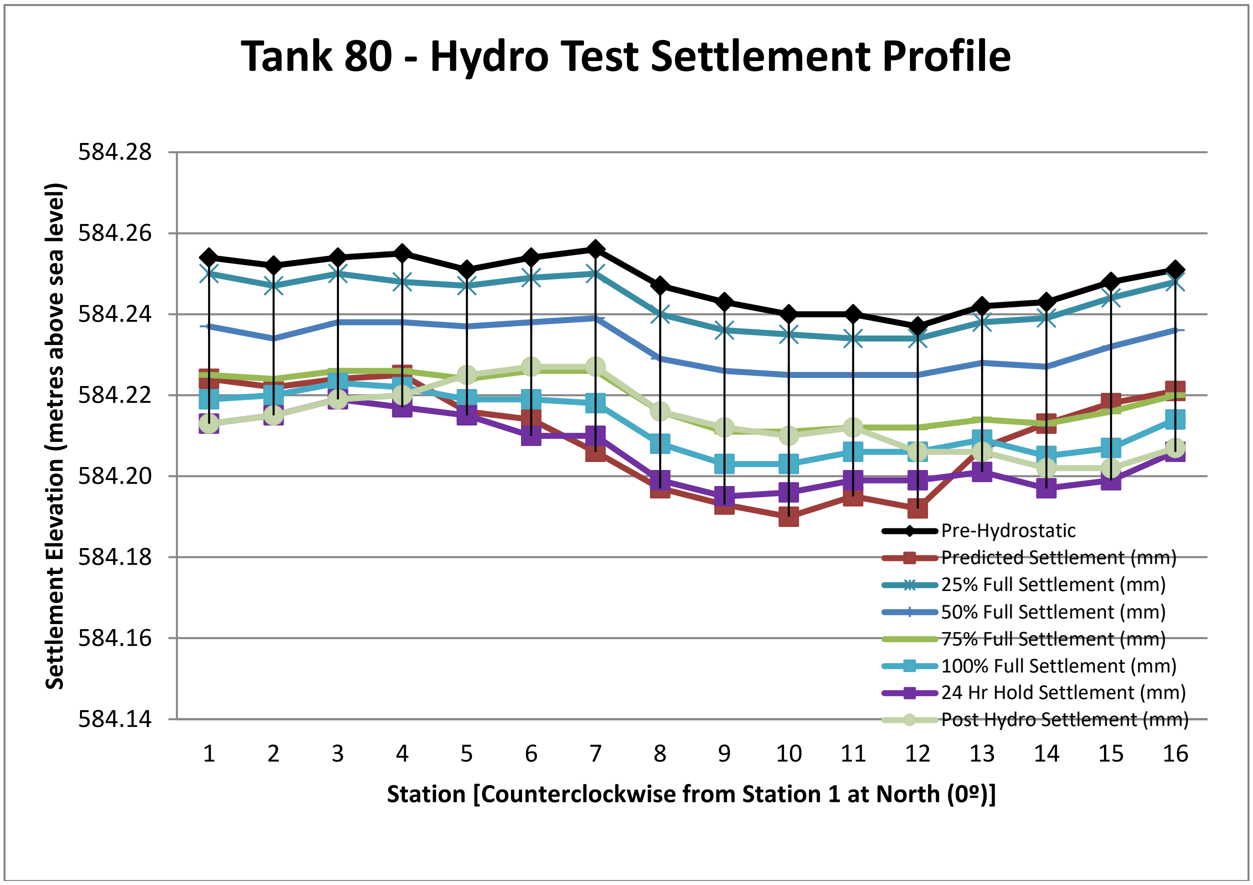


Figure 5 – Hydrostatic Test Settlement Profile: - Estimated and Measured Values (MacWhirter, 2015)

As seen above in, there is good correlation between the predicted settlement and the values measured during the 24-hour hold period of the hydrostatic testing. Along the northwest side of the tank, there is variance between predicted and measured, with the measured values following a more uniform profile. In general, the measured settlement followed a more uniform settlement profile than was predicted, which was the main goal during the backfill of the previous tank foundation excavation and during revisions to the foundation design.

7 Summary and Applications

Several challenges were encountered during the construction of the tank foundation system. The discovery of a pile supported foundation under the demolished tank lead to compounding issues. To reduce the risks associated with hard-spots at the pile locations an excavation was undertaken to cut the piles off. The backfill of this excavation was completed using material of variable quality, moisture condition, and density which lead to localized soft areas within the footprint of the new tank foundation.

In order to assess the state of the backfill material used, a CPT investigation was carried out. CPTs were completed both within and outside of the disturbed areas to compare the strength of the materials. CPT was chosen due to the quick rate at which the rig can be deployed to site, the live feed of conditions to help adjust the program as needed, and the rapid production of results for completing analysis.

Review of the conditions and the results of the CPT program indicated that localized soft areas were still present below the foundation footprint. A GCCS was used to help distribute the stresses within the ringwall across a larger area, to help the foundation bridge the soft areas which were still in-place. The use of the GCCS within the ringwall allowed the design to remain relatively unchanged, saving significant time and costs that would have been required if an alternative foundation system was needed to address the soft subgrade conditions. The incorporation of the GCCS within the ringwall allowed the foundation to perform as originally designed. Bridging soft areas ensured that the floating roof tank would perform as designed and would not be restricted by unexpected settlements along the tank walls which could have occurred due to the localized soft areas.

Once the tank foundation and tank shell construction had been completed, a hydrostatic test was undertaken following API 650 specifications. Using the updated settlement predictions completed with the updated soil information from the CPT program, the results of the hydrostatic test indicated that the tank foundation had performed as intended. The settlement profile of the tank was more uniform than predicted, but this was seen as an achievement, as this was one of the main goals during the backfill of the old tank foundation.

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API (American Petroleum Institute). 2013. *Welded Tanks for Oil Storage*. Section 7.3 (p. 7-7 – 7-9) and Annex B (B-1 – B-7). API 650. Washington, DC: API. [*Note: API650 has undergone changes since this project was completed, the reference included adopts the specifications that were in place during the design and construction of the tank*]

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