Performance of large diameter storage tank foundations on clay shale

Exécution des bases de réservoir de stockage de grand diamètre sur le schiste d'argile

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ABSTRACT

Canadian Natural Resources Ltd. (CNRL) has constructed storage tanks up to 20 m high and 60 m in diameter over weak soils at their new Horizon Oil Sands Plant near Fort McMurray, Alberta, Canada. A full-scale load test was performed to obtain physical data on soil behaviour in the area in which the tanks were to be constructed. The results of the load test were used to calibrate a finite difference model and to obtain realistic engineering parameters for the foundation soils. This model was then used during detailed design to predict foundation performance and to optimize the foundation design. The actual foundation performance was measured during hydro-testing of the tanks and compared to predicted behaviour.

RÉSUMÉ

Canadian Natural Resources Ltd. (CNRL) a aménagé de nombreux réservoirs de stockage à leur site d'Horizon Oil Sands près de Fort McMurray en Alberta. Les réservoirs mesurent jusqu'à 20 m de haut et 60 m de diamètre, et reposent sur des sols à faible capacité portante. Un essai de chargement à pleine échelle a été réalisé pour caractériser le comportement du sol sur les sites où les réservoirs seraient construits. Les résultats de l'essai de chargement ont été utilisés pour calibrer un modèle numérique basé sur les différences finies, et pour assigner des valeurs réalistes aux propriétés mécaniques des sols d'appui. Le modèle a ensuite été utilisé pendant la conception détaillée pour prédire la performance et pour optimiser la conception de la fondation. La performance des sols d'appui sous les réservoirs a été évaluée et comparée aux comportements prédits par le modèle numérique pendant l'inspection hydraulique des réservoirs.

Keywords : Storage tanks, weak foundation, FLAC, soil modeling,

1 INTRODUCTION

Canadian Natural Resources Ltd. (CNRL) has constructed a new Oil Sands Mine and Plant near Fort McMurray, Alberta Canada. It includes two large tank farms – the East Tank Farm (ETF) and West Tank Farm (WTF). Storage tanks up to 60 m in diameter and 20 m high are used to manage process fluids during plant operation. The primary geotechnical design issue for the tank farms is overall stability given the presence of weak soils in the foundation. The Cretaceous Clearwater Formation (a very stiff to hard clay, locally referred to as "clay shale") can be pre-sheared, has low residual strength, is known to exhibit strain-weakening behaviour, and experiences large pore pressure increases in response to loading.

Due to uncertainty concerning the behaviour of the Clearwater clay shale and the cost of mitigation, a full-scale field load test was conducted to assess foundation performance. A 40 m square test fill with two vertical faces was constructed using local sand, taken nearly to failure at a fill height of 10 m. Foundation performance was monitored using a variety of instrumentation (Moore et al. 2006). Using a numerical (FLAC) model calibrated to the behaviour of the test fill, the performance of the large storage tanks was then predicted, and a design selected that appropriately balanced risk of failure against cost of construction. The client elected to adopt an observational approach with a moderately conservative design.

All 3 tanks in the WTF and 10 tanks in the ETF were instrumented to monitor performance during hydro-testing. The amount of instrumentation varied, depending on the perceived risk of mal-performance. The most heavily instrumented tank was 72-TK-1A in the WTF. Alarm levels were set for all instruments and procedures developed to modify the progress of the hydro-test, depending on tank performance. Designs were also developed to mitigate poor foundation conditions and ensure acceptable performance during operations, which would be implemented if necessary.

This paper briefly outlines the early work on the test fill, and presents both the predicted and measured foundation behaviour during hydro-testing of Tank 72-TK-1A.

2 SURFICIAL GEOLOGY

The surficial geology in the plant site consists of Holocene and Pleistocene deposits overlying Cretaceous age Clearwater and McMurray Formation "bedrock". The near surface soils are: peat, fluvial and glacial-fluvial sand, glacio-lacustrine clay, and clay till. The "deeper" soils at the site consist of the Clearwater Formation clay shale and the McMurray Formation oil sand. Both of these are Cretaceous deposits and could geologically be defined as bedrock, based on their age. However, they have strength and deformation characteristics similar to a hard to very hard and/or very dense soil. In the area of the plant site, the Clearwater and McMurray Formations total about 100 m in thickness, and are underlain by Devonian Limestone.

More detailed information on these two Cretaceous formations is given in Moore et al. 2006.

3 TEST FILL

Based on early analyses, the stability of many of the tanks in the WTF and ETF was marginal and the cost to mitigate poor performance was high. Therefore, CNRL decided to perform a full-scale load test to properly understand both the Clearwater

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strength and pore pressure response parameters, which were identified as the critical factors that would affect foundation performance. A 40 m (square) by 10 m high test fill, with vertical faces, was constructed in the WTF to simulate actual foundation loading conditions. A detailed description of the test, with observed behaviour, is given in Moore et al. (2006).

The WTF test fill was loaded at a rate of 10 kPa per day (over a 40 m square area). The test was stopped when the fill height reached a vertical load of 150 kPa and there was 40 mm of movement in a concentrated shear zone in the clay shale, (equivalent to a local shear strain of 2.5%). The finite difference program FLAC was used to model the foundation performance during the test. It predicted failure of the test fill at a load of 180 kPa, with an associated maximum horizontal displacement of 200 mm and 13% (localized) shear strain.

Back analysis of the test suggested significant weakening of the Clearwater clay shale with increasing strain. This behaviour was not incorporated directly in the FLAC analysis, but handled indirectly with a strain-softening model. Although a good correlation between the FLAC predicted behaviour and observed field performance was achieved, it was felt that extrapolating predicted behaviour beyond the limits of the test fill experience should be done with caution.

4 WTF TANK ANALYSIS

4.1 *Performance criteria*

To meet the requirement described in the previous paragraph, for the WTF tank design, acceptable foundation performance was defined as a maximum predicted horizontal displacement of 20 mm and an associated foundation shear strain of no more than 1%. The failure criterion was set at 10% to 12% shear strain and/or a maximum horizontal displacement of 200 mm.

4.2 *Geometry, geology and boundary conditions*

The most critical foundation condition for the WTF existed beneath tank 72-TK-1A, where the Clearwater formation was thick and shallow. The geometry of the foundation in the east-west direction was the least favorable to stability. The tank performance was analyzed using a 2D, axisymmetric model, adopting typical boundary conditions for a problem of this type.

4.3 *Model parameters*

The FLAC model was run in a small-strain, un-drained mode, using a Mohr-Coulomb constitutive model. A strain-softening function was used to update the elastic parameters of the soil with changes in stress and strain, to simulate weakening of the foundation, as described earlier.

Although an un-drained analysis was performed, the method required that drained soil properties be input into the FLAC model, along with porosity and the bulk modulus of water. Un-drained parameters were then calculated by the FLAC program based on the saturation level of the soil.

The soil and water properties used in the FLAC analyses for the WTF and ETF were obtained from back analysis of the WTF test fill data and are provided in Table 1.

The back analysis of the WTF Test Fill gave a value of 100 MPa for the bulk modulus of water, compared to a bulk modulus for "pure" water of 1 GPa. Small amounts of free gas in the pore space of the soil reduce the combined pore fluid modulus significantly. The modeled modulus of 100 MPa equates to a soil saturation of about 99%.

Table 1 – Soil and Water Properties for FLAC Analysis.

Material	K (MPa)	G (MPa)	γ (kg/m^3)	η	ϕ (°)
Water	100	N/A	N/A	N/A	N/A
Sand Fill	40	18.5	1456	0.25	40
Sand 860	40	18.5	1700	0.25	40
Pl Clay 880	30	13.8	1580	0.42	30
Clay Till 830	30	13.8	1915	0.30	30
Kcc 710/720	50	23.1	1650	0.41	16
Kcb 700	50	37.5	1425	0.48	30
Kcb 650	65	48.8	1820	0.31	30
Kca 625 (Strong)	65	48.8	1540	0.45	30
Kca 625 (Weak)	25	5.4	1540	0.45	20
Kcw 600	70	32.3	1790	0.35	32

Legend: K – bulk modulus; G – shear modulus; γ - dry density; η - porosity; ϕ - angle of internal friction

4.4 *Initial stress state, stress path and strain history*

The behaviour of geologic materials is governed by effective stress laws and is strain / stress path dependant. In the FLAC model, the initial stress state and stress path was established as described in the following paragraphs.

Total and effective vertical stresses were determined based upon the unit weight of the soils, the initial distribution of pore pressures, and the model geometry.

Total and effective horizontal stresses were established using a horizontal to vertical effective stress ratio (K_o). No measurements of horizontal stress had been made at this site. Empirical correlations to lab testing results indicated that K_o lay between 0.7 and 1.0. A K_o value of 0.66, which gave the best fit to the measured data in the WTF test fill, was used in the analyses of the tank foundations.

For the WTF analysis, before placement of any grading fill, pore pressures were taken as hydrostatic to a water table located 1 m below ground surface (~ Elev. 295 m).

To model the construction sequence, the general loading path followed in the analyses was as follows:

- Apply grading fill to design elevation.
- Apply tank load in increments of 10 kPa or less.

During simulated application of the grading fill, pore pressures were allowed to increase in the lacustrine clay, clay till and Clearwater clay shale. The native sand and the sand fill were considered free draining (i.e. no pore pressure generation).

Little pore pressure dissipation was expected between completion of the grading fill, tank hydro-testing and tank operation. As a conservative approach, it was assumed that no pore pressure dissipation would occur over this time frame.

4.5 *Tank 72-TK-1A analysis - results*

Approximately 5.5 m of fill would be required in the vicinity of tank 72-TK-1A to reach the surrounding nominal design grade elevation of 300.0 m. The maximum predicted horizontal displacement in this load phase was approximately 20 mm and occurred beneath the fill slope, approximately 45 m from the edge of the tank. The maximum predicted shear strain was 0.4%, localized within the Kcc-710 unit approximately 45 m from the tank edge. Some shear strain was also predicted in the lower Kca-625 unit. The maximum post-fill, immediate settlement was estimated to be 20 mm.

The maximum applied load for this tank was understood to occur during hydro-testing and to be 200 kPa. For this load phase, the horizontal displacements were predicted to reach 50 mm within the clay till and the upper portion of the Clearwater clay shale, beneath the tank edge. A maximum shear strain of 1% was predicted within the Kcc-710 unit. A shear strain of 0.5% was predicted in the (deeper) Kca-625 unit.

Although the horizontal displacement criterion was exceeded for this case, the analysis was considered to be acceptable given the limitations of the model, some conservative assumptions concerning pore pressure dissipation, and the fact that the predicted shear strains did not exceed 1%.

To determine the global Factor of Safety, the simulated load was increased in steps until the ultimate capacity was reached at 400 kPa. The maximum predicted shear strain was 9%, localized within the Kcc-710 unit, with an associated horizontal displacement of 225 mm. The Kca-625 unit continued to show significant shear strain and displacement. The calculated Factor of Safety against foundation failure under the hydro-test load with a 0.5 m tank pad was thus 2.

5 HYDRO-TEST PERFORMANCE

This section evaluates the geotechnical performance of the tank foundation during the hydro-test, and compares the observed performance to predicted values.

5.1 *Settlement*

Settlement of the tank foundations was determined from high accuracy survey data. Sixty equally spaced monitoring points were located around the perimeter of each tank. During filling, settlement of the tank edge was measured at 30 of those points on a rotating basis when the tank was 25%, 50% and 75% full. A full survey of all 60 points was completed before the test and once the tank reached full load. Settlements at the centre of the tanks were determined by observing the displacement of the centre posts forming part of the roof support system. The settlement of the tank foundations were also monitored using 4 settlement plates installed around the inside edge of the tank.

The measured centre post settlement versus the applied tank fluid pressure is shown on Figure 1 (for tank 72-TK-1A). The measured centre post displacement exceeded the predicted settlement at a load of 25 kPa. However, the displacement was below the predicted value for the remainder of the test. At full tank load, the maximum settlement was 45 mm, 60% of the predicted settlement at that load.

The average edge settlement is shown on Figure 2. The edge settlement remained below predicted values throughout the hydro-test. At full tank load, it was about 17 mm, and after the 32 hour hold period it reached a maximum of 20 mm.

Out of plane settlements are particularly important in evaluating the structural integrity of the tank. A maximum out of plane settlement of 6 mm was recorded along the tank perimeter when the tank was at 50% load. At 100% load, all out of plane settlements were less than 4 mm.

All four settlement plates showed a similar response to the filling of the tank (Figure 3). The settlement plates exhibited displacements between 21 mm and 37 mm when the tank was fully loaded. Settlement continued, at a decreasing rate, over the hold period. As the tank was emptied, the plates showed 5 mm to 12 mm of rebound.

5.2 *Pore-water pressure response*

The foundation pore-water pressure response was monitored using vibrating wire piezometers installed at 2 depths beneath the tank centre. The variations in the piezometric elevations versus time and tank fluid level are shown on Figures 4 to 6.

As expected, the Pleistocene clay showed only a small response to the loading of the tank (Figure 5), with the piezometric level rising 1.21 m, corresponding to a B-bar value (ratio of piezometric response to applied load) of 0.06. During the 32-hour hold at full load, the piezometric level dissipated 0.29 m, or 24% of the total increase.

The Clearwater formation also responded as expected with a large pore pressure (Figure 6). The piezometric level increased 22.72 m at full hydro-test load, corresponding to a B-bar of over 1.1. During the hold at full load, the piezometric level dissipated by 0.71 m, only about 3% of the prior increase.

5.3 *Lateral ground displacement*

Measurements from 5 of the 6 slope inclinometers (WT1A-SI-1 to 5) installed around tank 72-TK-1A were used to assess lateral ground displacements during the hydro-test. Three of the SI's were located 8 m from the tank edge, one at 16 m from tank edge and one 32 m from tank edge.

The cumulative horizontal displacements observed in the SI's are within the predicted ranges obtained from the FLAC modeling. Figure 7 compares the measured deflections in inclinometers WT1A-SI-2, WT1A-SI-4 and WT1A-SI-5 (the three with an 8 m offset) with predicted values.

Based on the inclinometer data and the FLAC modeling, confined, low strain shear zones had started to develop during the hydro-test in the Clearwater formation – between elevations 275 m and 277 m in the Kca 625 facies, and between elevations 286 m and 288 m in the Kcc 710 facies. The shape of the inclinometer trace was very similar to that predicted by the FLAC model.

6 CONCLUSIONS

The deformation and strength properties of the WTF foundation was determined by FLAC modeling of a full scale load test. The resulting model was used to predict tank performance during hydro-test loading. The observed behaviour of tank 72-TK-1A under hydro-test loading was similar to or less severe than predicted values.

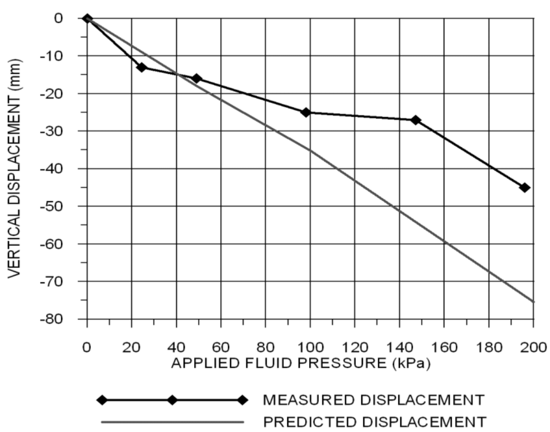

Figure 1. Settlement at tank centre post.

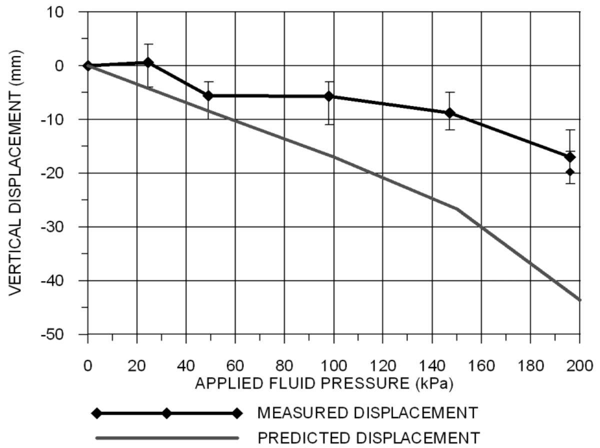

Figure 2. Average settlement at tank edge.

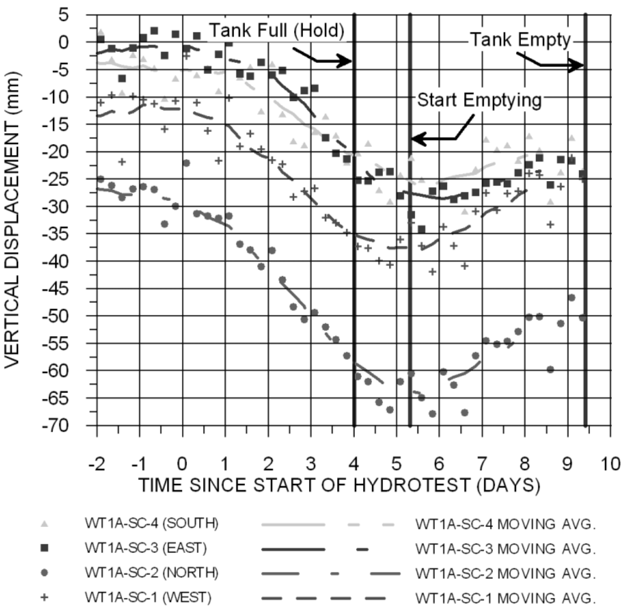

Figure 3. Settlement plate behaviour during hydro-test.

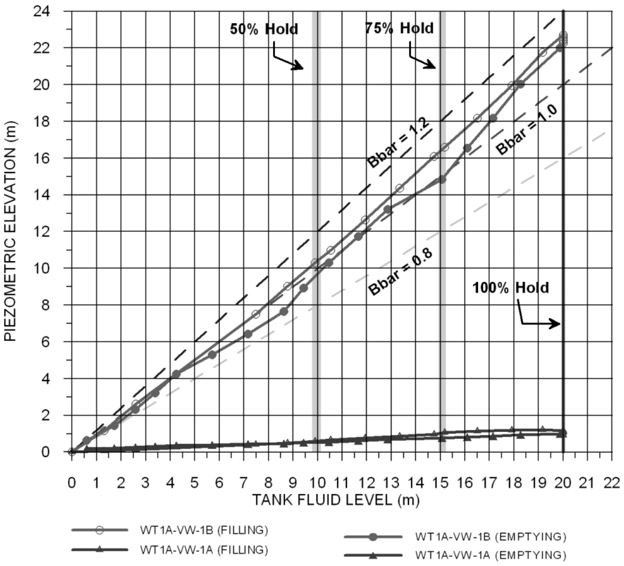

Figure 4. Pore pressure response vs. tank loading.

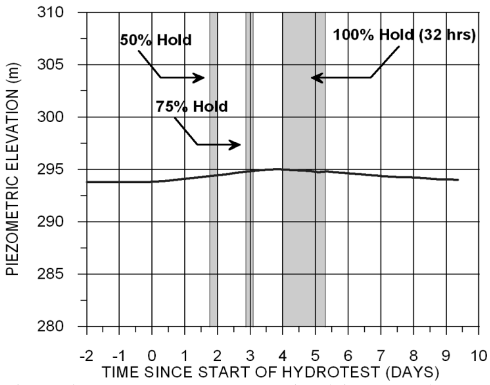

Figure 5. Pore pressure response in Pleistocene clay.

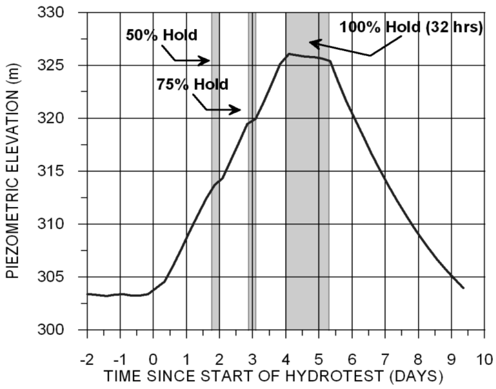

Figure 6. Pore pressure response in Clearwater clay shale.

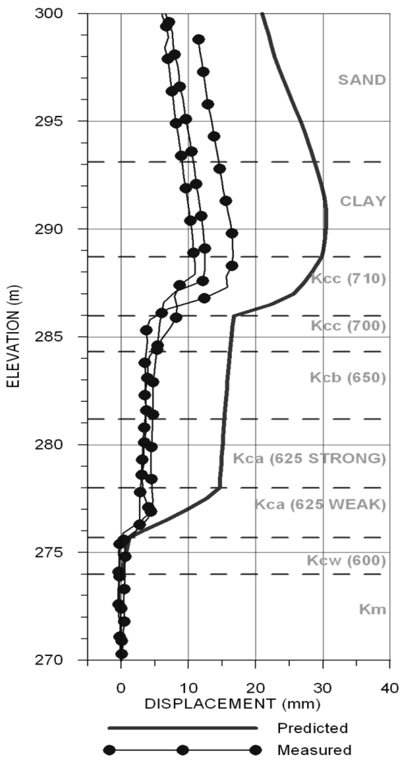

Figure 7. Predicted vs. measured displacements in slope inclinometers.

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