Investigation and commissioning of Large Tank, Shell Compound, Lyttleton

Ian McPherson
Connell Wagner.

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ABSTRACT

Shell (NZ) Limited is constructing three large fuel tanks at their Naval Point compound, Lyttleton. The area is underlain by a very deep deposit of soft soil and the area has a history of problems with tank foundations. Stone columns were installed beneath each tank to stiffen the subsoils and to provide more rapid dissipation of piezometric pressures during tank commissioning. Extensive monitoring was carried out during commissioning including piezometric pressures and settlements. Settlements in the order of 600mm to 700mm were predicted during hydrotesting compared with measured settlement in the order of 550mm.

1 INTRODUCTION

Shell (NZ) Limited is redeveloping their tank farm at Naval Point, Lyttleton. Three 30m diameter tanks are proposed with a shell height of 16m and a fill height of 15.5m.The site was initially developed in the early 1900's by constructing a perimeter rock revetment and reclaiming land by placing material dredged from the harbour floor. During the reclamation process, substantial failures of the revetment and reclamation occurred that took many years to correct. The Shell site was developed for bulk storage of petroleum in the 1920's with a number of tanks built, and demolished over the last 80 years. In the mid 1950's a tank failed on first filling when the tank had about 12m of fluid inside. Subsequent tanks have had some form of ground improvement with staged filling during the hydro testing and tank commissioning.

Shell (NZ) commissioned Connell Wagner to provide geotechnical engineering services for the redevelopment of the site for three new tanks. This included investigation of the site, design of ground improvement and monitoring during hydro testing.

2 SITE INVESTIGATIONS AND GROUND CONDITIONS

A site investigation was carried out including three Cone Penetration Tests (CPT) and one borehole under each tank footprint, or as close as possible to the tank site within the constraints of site access. The boreholes extended to 30m depth and the CPTs to 30m depth, or when refusal was reached. Push tube samples were taken from the boreholes at 1m intervals to 6m in depth and 1.5m below. The soil undrained shear strength was measured in the ends of the each push tube sample.

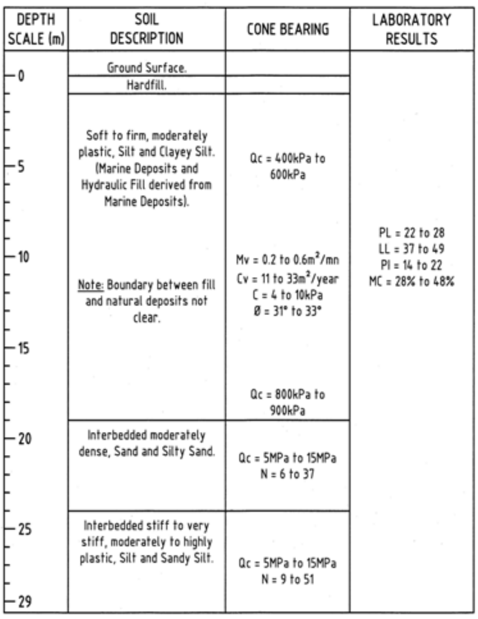

Figure 1: Site Stratigraphy

Lyttleton Harbour is situated in the flooded crater of an ancient volcano. The sea floor typically comprises thick deposits of soft silt and clay derived from loess washed off the flanks of the crater and into the sea. Site stratigraphy therefore typically comprises

20m of soft silt/clay (USC classification CL) overlying stronger silts and sand. The upper 4m to 5m of soil is inferred to be reclamation fill dredged from the harbour but the exact depth of fill is unknown as there was no appreciable difference between the CPT profiles in the reclamation fill and the underlying natural soils. A 400mm to 600mm crust of drier stiffer silt overlies the soft material. An indicative soil profile is presented in Figure 1 together with a summary of laboratory and field test results.

Shear vane testing in push tube samples indicated relatively uniform undrained shear strength between 20kPa and 30kPa down to 8m depth increasing to between 30kPa and 50kPa at 18m depth. This was different from the CPT logs where the gradually increasing cone bearing with depth indicates increasing undrained shear strength with depth. The difference is attributed to disturbance of material in the push tubes.

The interbedded sands and silts underlying the surficial silt/clay were logged as very stiff, sandy clayey silt and moderately dense sandy silt and sand. CPT cone bearing values ranged between 2MPa and 4MPa in the cohesive soils and 5MPa to 15MPa (i.e. cone refusal) in the sandy layers. SPT “N” values ranged between 9 and 37.

3 FOUNDATION DESIGN

Most tanks constructed at Naval Point over the last 40 years have had ground improvement by sand or stone columns. The proposed tank walls are 16m high with a 15.5m fill height. The foundation loads will therefore be up to 160kPa when the tank is hydrotested during commissioning, and 130 kPa in the long term when filled. Initial calculations showed that bearing failure of the surficial soils was likely during hydrotesting and settlements of 1m could occur under long term loads. These calculations confirmed the need for ground improvement.

Two forms of improvement were considered, soil cement mixing and stone columns. Soil cement mixing had the advantage of shortening the tank commissioning time as the treated ground would be relatively strong and incompressible. Stone columns had the advantage of cost and were a proven method of ground improvement in the Naval Point area, and were selected. The settlement of the tanks was assessed assuming that 17% of the ground under the tank was replaced by stone columns extending to 12m depth. The depth was based on economics as columns deeper than this became increasingly expensive to install. Also, initial calculations indicated that treatment to this depth would keep settlements within acceptable limits.

The settlements were assessed using the Sigma/W finite element package and an elastic-plastic soil model under axisymetric loading conditions. The modulus of Elasticity (E) for the various soils underlying the site was based on the triaxial test results, empirical correlations between undrained shear strength and E, and empirical correlations between cone bearing and E.

An E value of 50MPa was assumed for the 1m thick gravel layer to be placed across the site. The stiffness for the stone column reinforced block was based on the method of Almedia and Perry (1985) which uses the proportion of stone columns in the reinforced block, and the relative stiffnesses of the gravel columns and natural soils, to derive an equivalent E value. In this case, an equivalent E value of 15MPa was derived for the reinforced block. The analytical model developed for the site is shown in Figure 2.

The settlement analysis indicated that up to 700mm of settlement could occur under the hydro test load of 160kPa depending on test duration. The long term settlement under the tank full load of 130kPa would be in the order of 600mm at the centre of the tank and 500mm at edges. The 100mm predicted differential settlements complied with the centre to edge industry standard limits of 150mm.

Two of the CPT logs indicated slightly higher cone bearing values than the other CPT logs from the site. It was not known whether this was due to natural variation in the soil properties or due to consolidation by previous developments. Regardless of the cause, a difference in subsoil stiffness as indicated by the CPT testing could lead to differential settlements. Some differential settlements are allowable but are limited to a maximum of 260mm across the tank diameter and a slope of 1% around the tank circumference.

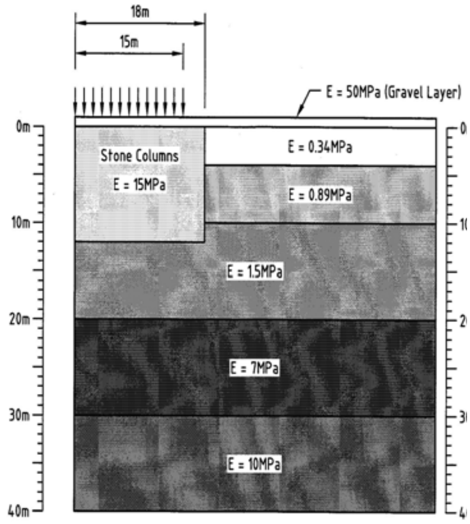

Figure 2: Analytical Model

Assessing differential settlements under a tank with soil stiffnesses varying vertically and horizontally is not simple. The following procedure was used to assess potential differential settlements:

- Assess settlements using axisymmetric and plane strain models and laterally uniform soil properties;
- Assess settlements using a plain strain model and reducing the E value under half the tank in proportion between the lowest and highest cone bearing profiles with depth;
- Pro rata axisymmetrical settlements by the ratio of plain strain settlement for the laterally uniform and the laterally variable E cases.

The methodology indicated that about 65mm of differential settlement could occur across a tank diameter, which complied with the industry standard limit of 260mm.

4 FOUNDATION CONSTRUCTION

The design replacement value for stone columns was 17% of the soil volume under the tank. This was achieved by installing 600mm columns at 1.5m centres. The column diameter was based primarily on construction requirements, as this was the most economical for the available equipment. The stone used had a maximum of two broken faces. The broken face requirement was to ensure a reasonably angular stone was used to give a high friction interlock and therefore a strong column. The column length was increased slightly from that assumed during design to 14m as the contractor could construct this depth of column without a substantial cost penalty and the extra depth provided an improved margin against unexpected settlement.

The columns were constructed by vibrating a 600mm column in to the ground with a “penny” fixed to the bottom of the tube. The tube was partly filled with water and then the gravel. A vibrating clamp was used to extract the tube leaving the gravel column in place. The volume of water was varied during the initial stages of column construction to give the maximum amount of water with easy flow of gravel out of the tube. Column production peaked at about 30 columns per day with a total of 1,146 columns were installed under two tank locations.

Four vibrating wire piezometers were installed after the stone columns were completed, two under the centre of the tanks at 4.5m and 18.5m depth and two under the edge of the tank, also at 4.5m and 18.5m depth. The shallow piezometers were intended to monitor the efficiency of the stone columns in controlling excess pore pressure in the surficial soil and the deep piezometers to monitor excess pore pressures in natural soils unaffected by ground improvement. Any residual excess pore pressure remaining after hydrotesting would also provide information on the potential for long term residual settlement.

A significant amount of ground heave occurred during stone column construction and about 2,500m³ of soft soil was removed from the site to reinstate the original ground level. The volume of soil removed compares with about 4,500m³ of gravel in the stone columns. Once the soft soil was removed the piezometers were installed and 1m of gravel was placed across the site. The 1.6m high concrete ring beam foundation was then constructed. The tank ring beam was filled with gravel so that an initial load of about 50kPa was applied to the tank foundations and 20kPa across the rest of the site. Settlement occurred under this load and 12 settlement markers were installed on the ring beam, evenly spaced around the tank circumference. Construction of the steel tank then commenced with an additional 12 settlement markers equally placed around the circumference of the tank floor plate.

5 TANK MONITORING

Piezometer monitoring started immediately after placement of the gravel blanket and settlement monitoring of the ring beam and tank floor plate of Tanks ST24 commenced once these elements were completed. The piezometers showed an immediate response to placement of the gravel blanket and ring beam weight about 52 kPa). The shallow piezometers indicated a 0.8m to 1.3m increase in piezometric pressure and the deep piezometers 1.7m to 1.8m. The piezometric pressures slowly reduced over the next five to six months and appeared to be very sensitive to construction activities and possibly rainfall.

The deep excess pressures under the centre of Tank ST24 gradually dissipated over about six months. The deep piezometer under the edge of the tank indicated an initial decrease in piezometric pressure of about 1.6m over the first month and then a slow gradual decrease over the next four months. Limited settlement monitoring was carried out over the 12 months of tank construction with approximately 200mm of settlement recorded.

Hydrotesting of Tank ST24 commenced on the 10 November. Piezometer monitoring results are summarised in Figures 3 to 4, and settlement monitoring in Figure 5. The tank was filled in five height increments each of 3.2m of water with each filling taking one to two days to complete. Because of this, piezometric pressures did not peak for two to four days after the start of filling.
There was a significantly different response between the deep and shallow piezometers, and between piezometers under the centre and edge of the tank. All piezometers showed a limited response to the initial 3.2m fill with shallow piezometers indicating about a 0.8m increase in head and the deep piezometers 1.1m. The shallow piezometers then indicated a 1.2m increase in head for the next three cycles with a 0.9m head for the last increment (Figure 3). Classical stress distribution under a circular load indicates that the stress under the edge of the tank should be 45% of that under the centre. Given that the piezometric response of both shallow piezometers

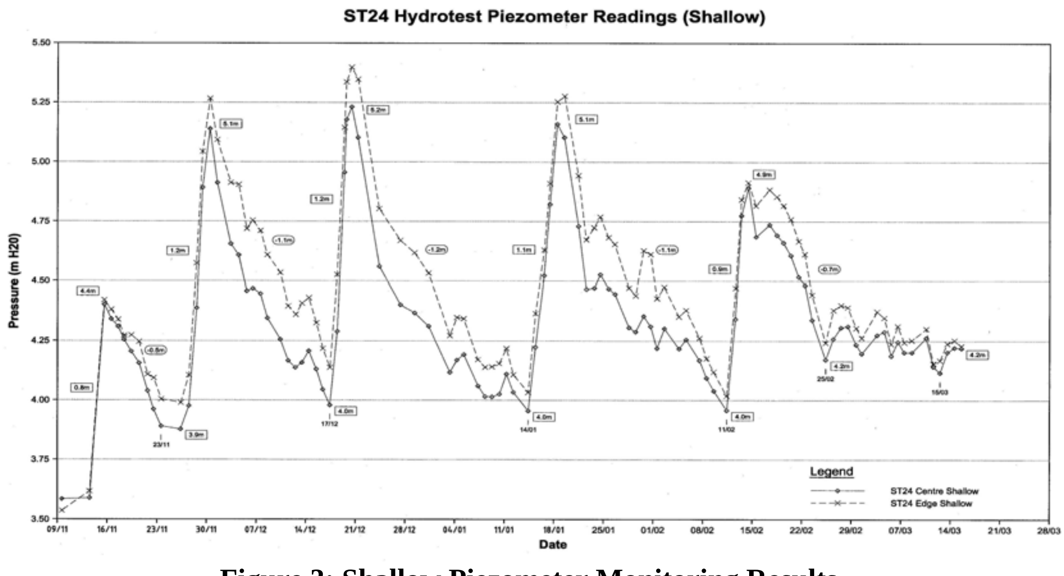

Figure 3: Shallow Piezometer Monitoring Results

was almost identical, we infer that the stone columns limited the increase in piezometric pressures to the maximum 1.2m measured.

The excess piezometric pressures measured during the first four stages of loading largely dissipated within the period between fillings. Stage 1 dissipation was slightly less than the piezometric pressure increase but there was only a 14 day period between the Stage 1 and Stage 2 fillings, which may have not allowed sufficient time for pressures to dissipate. The Stage 5 filling was different to the previous stages as the piezometric pressures rose less (0.9m vs 1.2m) and pressures fell quickly by 0.7m. They then oscillated about 4.2m over the next two to three weeks. At the time of preparing this paper pressures were still about 600mm higher than the precommissioning levels.

The deep piezometers (Figure 4) also showed a limited response to the first loading stage. The reasons for this are not clear but the ground was significantly disturbed during stone column installation and possibly stone column installation effectively preloaded the soils immediately under the reinforced block. The second fill stage resulted in a 2.3m pressure increase under the centre of the tank with a 2.7m and 2.4m increase for Stages 4 and 5 respectively. Pore pressure dissipation was much slower in the deep soils below the stone columns and typically about 20% to 30% of the piezometric increase dissipated during the first two loading cycles and 50% to 70% in the next three cycles.

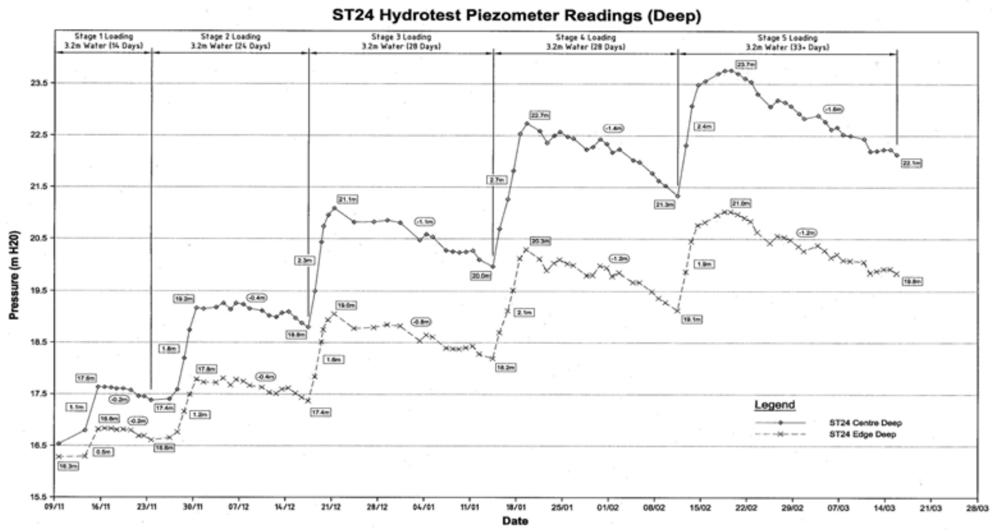

Figure 4: Deep Piezometer Monitoring Results

The coefficient of consolidation, as indicated by the slope of the settlement versus square root of time plots decreased from Stage 1 to Stage 4 and increased for Stage 5. Because of this, and the overall shape of the piezometric pressure plots for all four piezometers, we infer that the soil may have been close to failure under the final load increment.

The piezometric response of the deep piezometer under the edge of the tank was similar in shape to the centre of the tank. However, the piezometric pressure increase increased from about 45% of that for the deep centre piezometer during the first load cycle up to 79% for the final load increment. Classical stress distribution theory indicates that the stress increase under the edge of the tank should be approximately 50% of that at the centre. It is presumed that the stone columns acted as a reinforced block and transferred a greater proportion of the stress increase to deep soils than assessed by elastic theory, which assumes that the load is applied at the surface.

The settlement monitoring results (Figure 5) indicate a maximum of 350mm of settlement has occurred during hydrotesting with a rate of 1mm per day occurring when this paper was written. The total settlement that has occurred under the weight of the gravel fill, tank foundation and water load is in the order of 550mm. This is in the same order of magnitude of the original predictions (300mm to 500mm after eight weeks of hydrotesting). The original prediction however, was based on an applied load of 160 kPa whereas the current load is in the order of

210 kPa. Differential settlements in the order of 130mm have occurred more or less across the tank diameter, which is twice that originally predicted. The reasons for the magnitude of the differential settlement are not clear, as the pattern differential settlement does not match any previous site usage, which could have preloaded half of the tank footprint. It is postulated that the slight tilting represents natural variation in either, or both of, the reclamation fill and

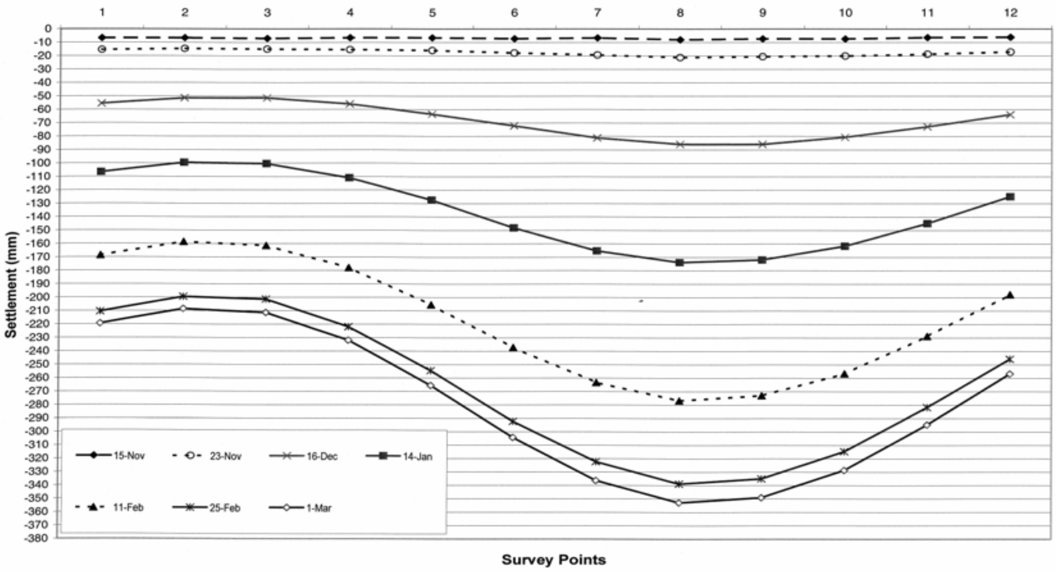

Figure 5: Settlement Monitoring Results

underlying soft settlements.

6 SUMMARY AND CONCLUSIONS

Extensive investigations under the proposed development area confirmed the presence of very deep deposits of soft soils. Stone columns were installed to stiffen the site subsoils down to 14m depth. Monitoring included four piezometers and 24 settlement monitoring markers were installed as tank foundations were completed. The piezometers indicated significant changes in piezometric pressures as the tank foundations were completed and settlement monitoring indicate that up to about 200mm of settlement had occurred prior to tank commissioning. About 350mm of settlement had occurred during hydrotesting at the time this paper was prepared, or 550mm in total under the combined weight of 16m of water and 2.6m of gravel foundations. This is similar to the 500mm to 600mm predicted.

The stone columns successfully controlled piezometric pressures in the surficial soils with minimal excess pore pressures 28 days after filling. There has been a 7.2m rise in piezometric pressures at 4.5m below the bottom of the stone columns under the centre of the tank. Approximately 1.6m of this peak has dissipated and the excess pore pressure under the tank at depth is currently about 5.6m.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

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