

Geogrid Reinforced Granular Pad Foundation Resting on Loose and Soft Soils, Hamilton Harbour, Ontario

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Abstract The waterfront quays in Hamilton Harbour have been constructed by placing upfill within a perimeter steel sheet pile bulkhead. The fill materials are believed to consist of dredged material loosely placed on the original harbour bed, within the enclosure. The quayside had been used for general dock activities, including stockpiling discharged ships' cargoes. Recently, two 31 m diameter liquid storage tanks have been constructed on the site; the stored liquid applies a load of 120 kPa to the ground surface. Based on borehole data developed in an initial geotechnical investigation of the site, it was recommended that the tank loadings be supported on either stone columns or end bearing pile foundations. The cost of these foundation systems was prohibitive to the development. Additional fieldwork consisting of three soundings with a Marchetti flat plate dilatometer was carried out to better define the geotechnical properties of the soil profile. Using these data, a foundation design was prepared which consisted of construction of a geogrid reinforced granular pad. Post construction monitoring of settlement of the tank foundation shows that settlement of the foundation has been within tolerable limits.

Keywords Soft soils · In situ testing by flat dilatometer · Settlement · Foundation design with geogrid reinforcement

Introduction

The site development consisted of the design and construction of two 31 m diameter liquid storage tanks at the Emerald Street Slip, which lies on the south side of Hamilton Harbour, Ontario, Canada (Fig. 1). At this location the land has been reclaimed from the Harbour by construction of a steel sheet pile bulkhead enclosure which was then infilled on the landward side to provide a level surface for cargo handling and storage. It is believed that the infill used to raise the grade in the slip area consisted of harbour bed dredged material, which was loosely placed on the harbour bed within the perimeter bulkhead.

In the vicinity of the development site, various buildings and storage tanks have been constructed. However, the study site is situated on a previously undeveloped area of the Slip. Anecdotal evidence indicates that some of the current development site had been used previously for storage of unloaded ships cargoes consisting of construction aggregates in conical piles, which were more than 10 m high. The perimeter of such storage stockpiles was believed to lie between the north and south limits of the development site, which resulted in the soil profile beneath the southerly portion of the site area having experienced consolidation under such applied loads.

A previous geotechnical investigation of the site area was undertaken prior to undertaking the described investigation and design works. That study consisted of advancing six conventional sampled boreholes to depths of up to 26 m. Sampling and in situ testing was carried out in those boreholes by the standard penetration test method. The results of that test program illustrated the following typical soil profile:

- Ground surface to 0.5 m; pavement structure.

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Fig. 1 Site plan, Emerald Street Slip Hamilton Harbour

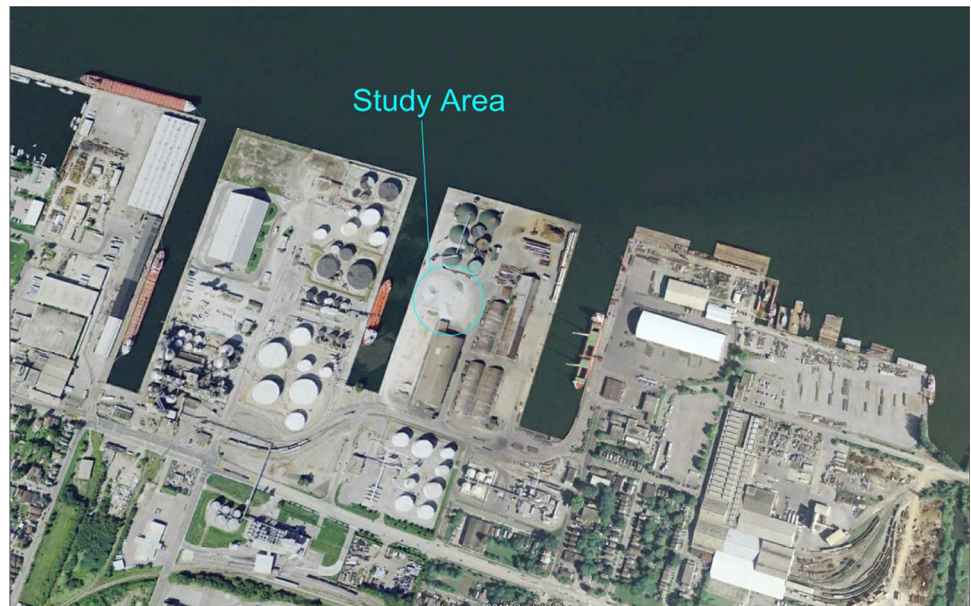
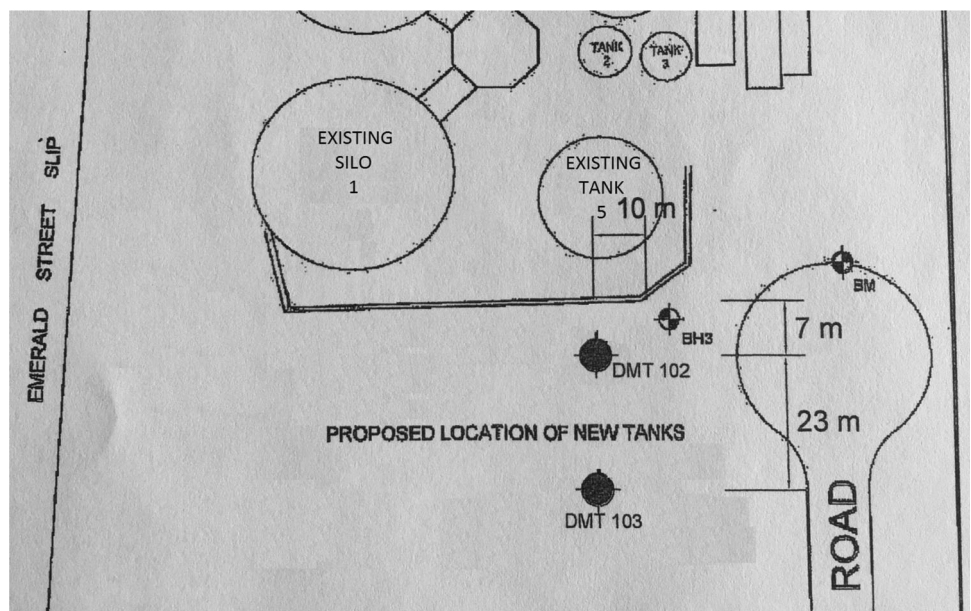
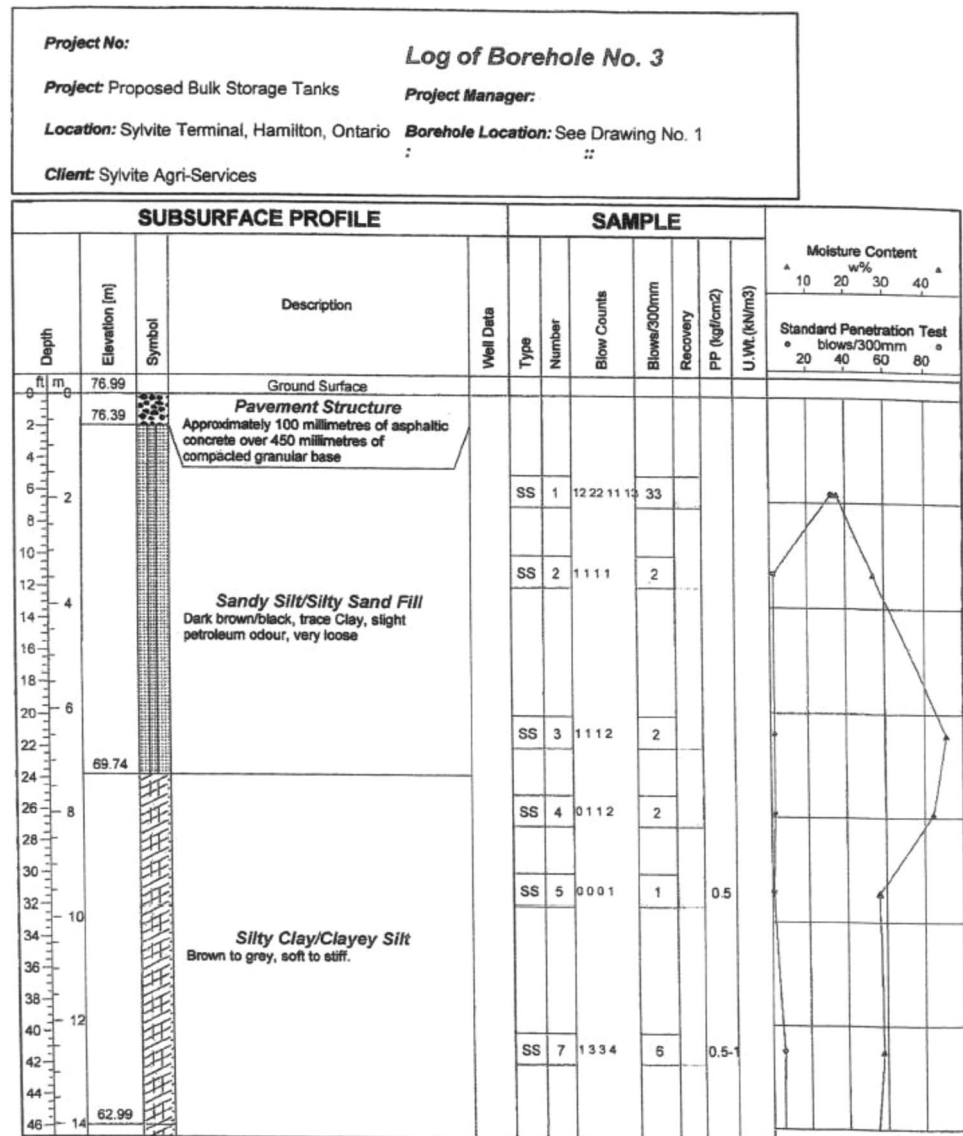


Fig. 2 Exploration location plan



- 0.5 to 7–12 m; Fill, mostly consisting of silt to sand with zones of silty clay. The layer is considered in two sections, with an approximate 2 m thick upper sub-unit of loose to compact sand and gravel with some ash lying on very loose to loose silt to silty sand. The soil condition was interpreted from standard penetration test N values of 8–55 blows/300 mm recorded in the upper sub-unit of the fill, and N values of 1–5 blows/300 mm in the lower portion of the layer.
 - 7–12 to 24 m; soft to firm becoming stiff silty clay, soil consistency represented by N values of 1–34 blows/300 mm, more commonly in the range of 3–15 blows/300 mm;
 - 24–26 m, dense to very dense clayey silt and sand, condition represented by an N value of more than 50 blows/300 mm;
 - 26 m shale bedrock.
- In one of the boreholes, a 4 m thick layer of compressed peat was found to be sandwiched between the base of the fill and the underlying sequence of mineral soil deposits; N values of 7 and 8 blows/300 mm were measured in this

Fig. 3 Borehole log



soil lens. A plan showing the locations of geotechnical explorations is given in Fig. 2. A copy of a log sheet for a representative borehole is given in Fig. 3. On the basis of the determined soil profile and interpreted soil condition data from the sampled borehole investigation, the original geotechnical engineer concluded that the soil profile underlying the site could not support the tank loads and that each of the two storage tanks should be founded on an array of end bearing piles which would be embedded in the very dense basal soil stratum or on bedrock. An alternative recommendation was given that the condition of the underlying soils could be improved sufficiently to support the tanks by installation of a set of stone columns. The estimated cost of providing such a foundation was approximately \$500,000.00, above the cost of a conventional shallow foundation, which compromised the financial viability of the

project. Therefore, the owner determined that an independent review of the subsurface data be carried out with a view to developing a design which provides foundation support at shallow depth in the soil profile.

Some details of this case study were previously presented by Alston et al. [1].

Geotechnical Investigation of In Situ Soil Properties

It is well known that steel storage tanks are settlement tolerant [2–4]. Also, as noted in Sect. 1, anecdotal evidence indicated that previously, the southerly portion of the site had been loaded with stockpiles of aggregate materials to a height of up to about 10 m. Thus, past performance of the ground in the general area of the site had shown that the subsurface profile was able to sustain significant surface

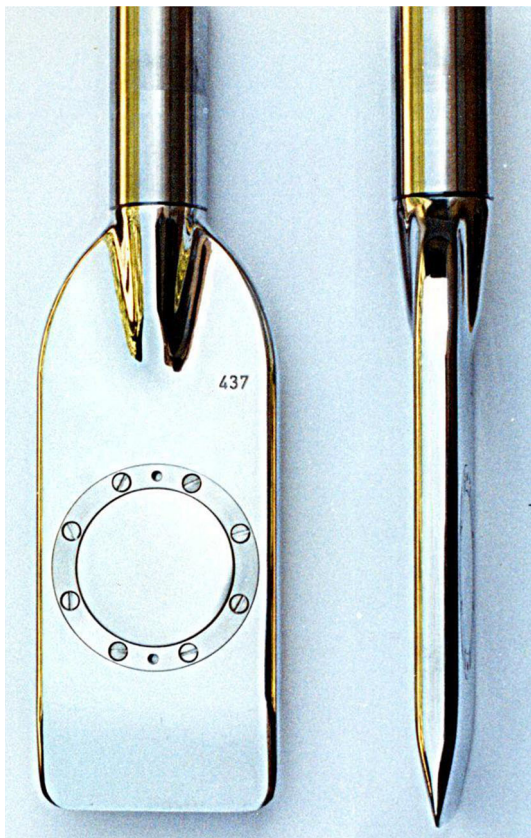


Fig. 4 Illustration of test blade Marchetti flat plate dilatometer

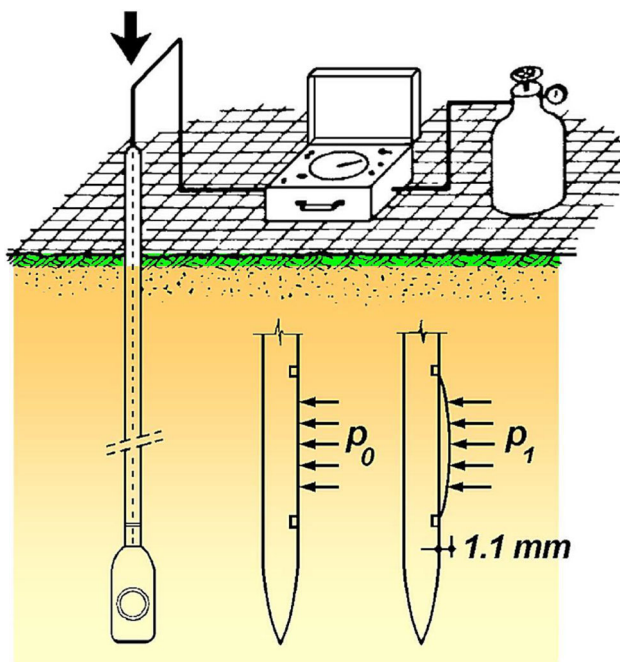


Fig. 5 Illustration of field arrangement Marchetti flat plate dilatometer

loadings without experiencing a catastrophic shear failure. On this basis, an additional geotechnical investigation of the site [5] was undertaken to better define the in situ properties of the soil profile. The second phase of geotechnical work was designed to complement the available borehole data with a more sophisticated program of in situ testing. This work unit consisted of advancing three soundings using a Marchetti flat plate dilatometer (DMT) to provide interpreted measures of in situ soil properties [6]. A picture of a dilatometer blade is shown in Fig. 4 [7] and a diagrammatic illustration of the test arrangement is shown in Fig. 5 [7]. The soundings were advanced under the northerly part of the development site in which the anecdotal evidence indicated that no previous significant loading had been applied to the site area, as well as in the more central and southern portions of the site which are believed to have been previously subject to significant loading by aggregate stockpiles. The results of the northerly sounding (DMT 102) and the southerly sounding (DMT 103) are given in Figs. 6 and 7, respectively. Reference to the results of the soundings illustrates the interpreted behavioural character of the overlying fill materials as well as the inferred engineering properties of the various soil materials which were contacted in the soundings. Reference to the reported in situ test results also illustrates the beneficial effect of previous loading of the site by stockpiled aggregates on soil parameters such as undrained shear strength and constrained modulus. Summaries of the soil parameters selected for stability analysis and settlement predictions are shown in Table 1.

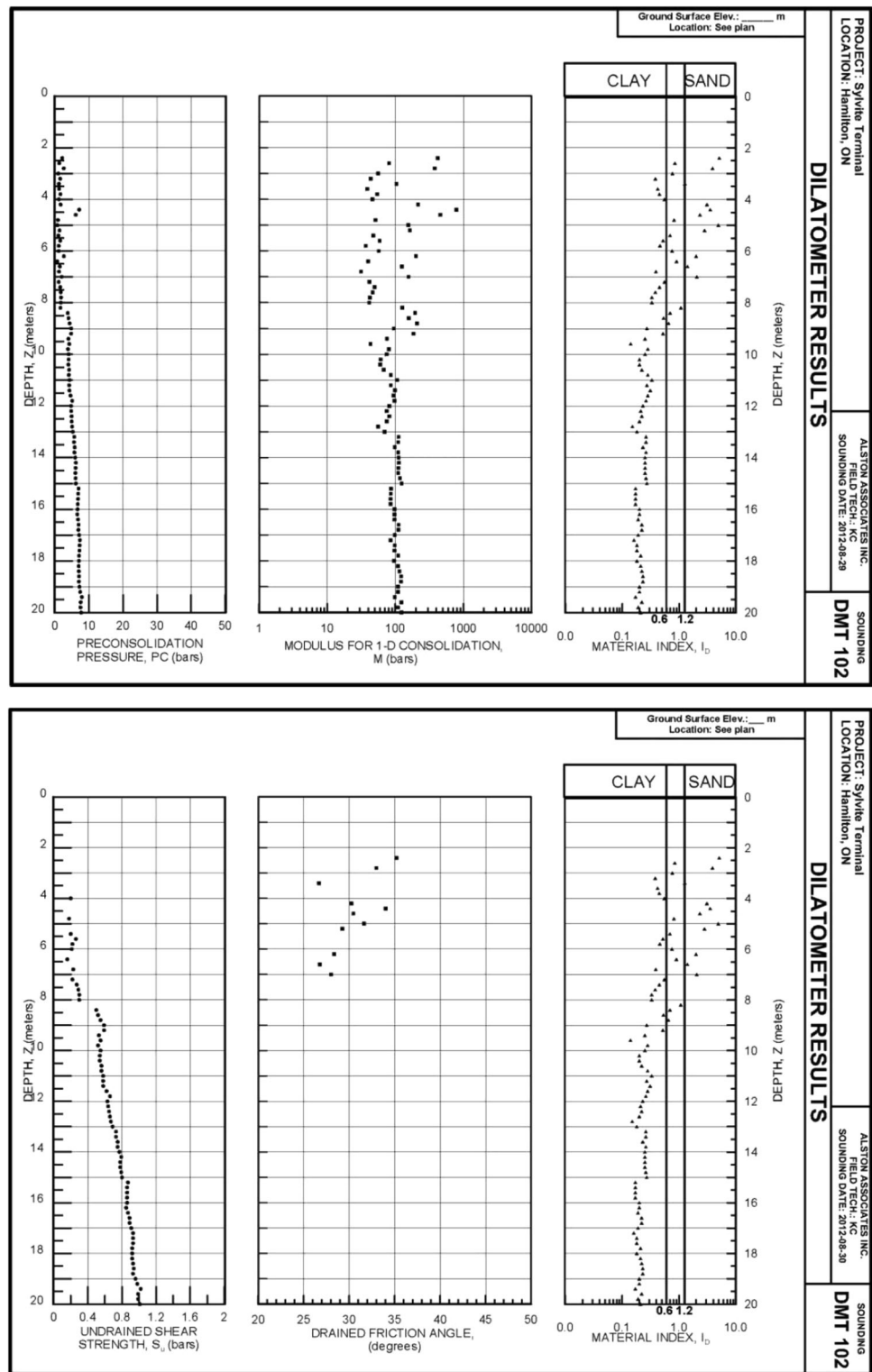
Published literature [8, 9] shows that good confidence can be placed on the results of DMT soundings with regard to settlement predictions.

Geotechnical Design

The results of the subsurface investigations of the site showed that the geotechnical design of surface supported structures would be primarily controlled by the properties of the relatively weak surficial layer of fill and the upper soft to firm zone of the underlying silty clay soil deposit. These soil layers could be expected to experience significant consolidation as a result of load application. The shear strength characteristics of the respective soil layers limit the intensity of foundation load which can be applied to these layers to a modest level, unless the soil materials were to be improved or mechanical appurtenances designed to increase tensile strength were included in foundation design.

In order to assess the feasibility of adopting a design based on a tank base resting on grade and shallow foundations to support the tank wall loads, an estimate of

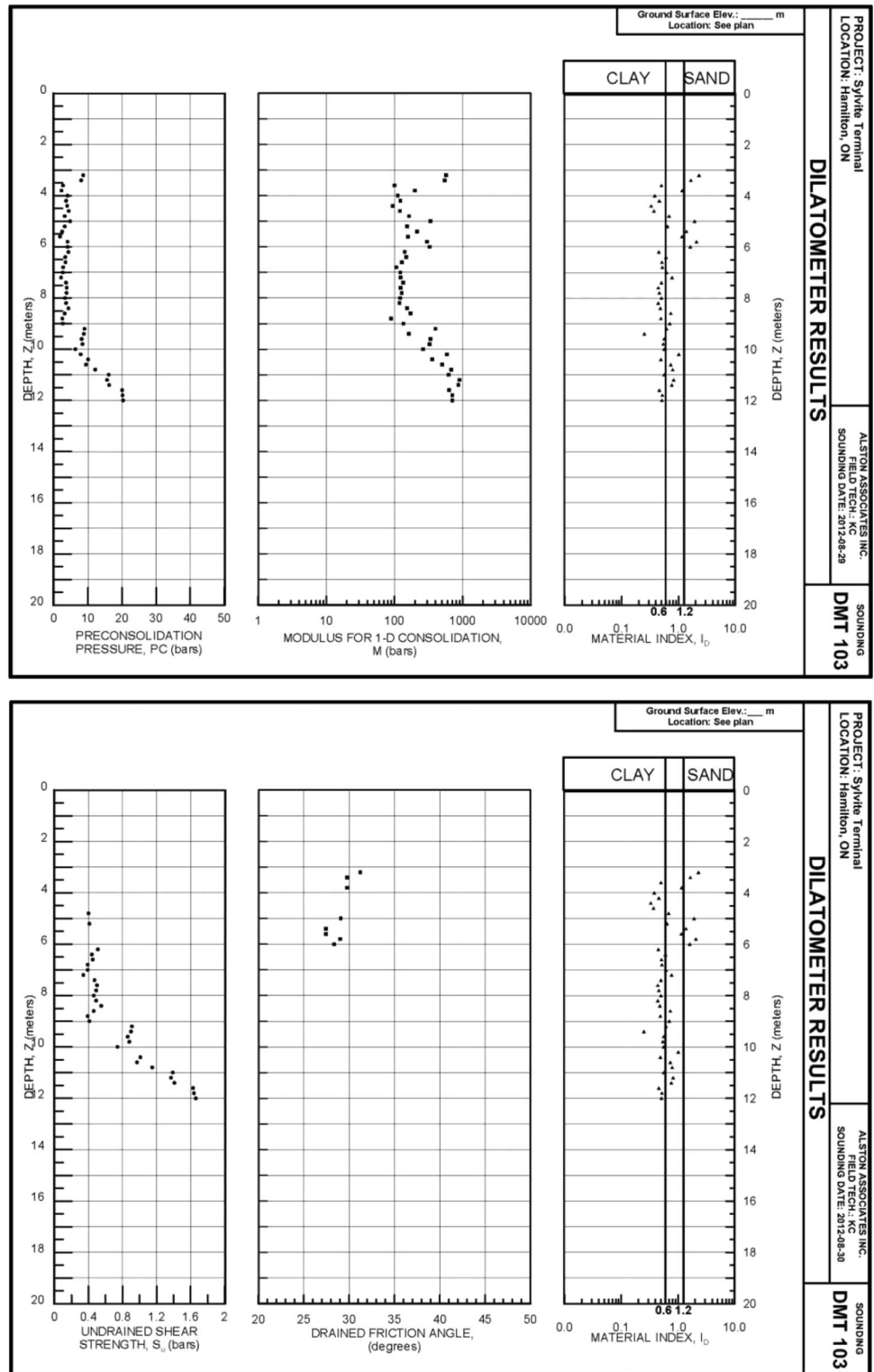
Fig. 6 Flat dilatometer sounding 102 results



probable settlement was made to determine if this could meet the project’s operational criteria. Analysis was carried out using the GEO5 program “Settlement”, verified by hand calculations. The analysis of the probable consolidation settlement of loaded tank foundations was carried

out using the values of constrained (deformation) modulus of the soil materials which were interpreted from the DMT results. The analysis gave estimated settlements of about 55 mm at the south (probably preloaded) limit of the tanks, 130 mm at the tank centre and about 80 mm at the northern

Fig. 7 Flat dilatometer sounding 103 results



(unloaded) limit of the tank footprints. These values of settlement are within the criteria given in the referenced technical literature and were confirmed as acceptable with the Owner for the project in hand. A graphical summary of the results of the settlement analysis is given in Fig. 8.

The values of settlement generated by the computer program were verified by hand calculation for the northern limit of the tank wall. The interpreted values of overconsolidation ratio from DMT information are generally less than 2 in the upper soil units and increase to more than 4

Table 1 Geotechnical properties of soil units

Soil unit	Approximate depth range (south) (m)	Approximate depth range (north) (m)	Angle of internal friction (°)	Undrained shear strength (south) (kPa)	Undrained shear strength (north) (kPa)	Constrained modulus (south) (MPa)	Constrained modulus (north) (MPa)
Silty clay fill	–	0–4	–	–	20	–	4.5
Silt and sand fill	0–9	4–5.5	27	–	–	20	20
Firm silty clay	–	5.5–10	–	–	28	–	10
Stiff silty clay	9+	10+	–	70	70	10	10

below a depth of 8.5 m; on that basis it was expected that little consolidation would occur below that depth. The estimated settlement at the check position was as shown in Table 2.

This gives an accumulated consolidation settlement of 72 mm, which is comparable with, but less conservative than, the computed value of 80 mm.

The settlement criteria being met, the design challenge was to include tensile reinforcement in the design which would upgrade the support characteristics of the underlying relatively weak soils to secure the foundations against shear failure in the foundation soils with adequate factors of safety, to meet contemporary standards.

The borehole and sounding data show that the near surface fill materials underlying the site consist of a variety of dominantly granular material (sand, gravel, ash, cinders present in varying proportions). The results of grain size distribution tests carried out on samples of the upper sub-unit of the fill show that these materials could be selectively excavated and mostly re-used and engineered to provide a granular pad with consistent soil properties to support tank foundations, provided that the pad is suitably reinforced. Analysis showed that inclusion of geosynthetic reinforcement in the engineered granular fill pad would add sufficient tensile strength to the fill pad to improve the design factors of safety with respect to both local and global stability, to satisfy conventional foundation design criteria. The selected reinforcement consisted of including two layers of polyester geogrid each with an ultimate tensile strength of 68 kN/m in the primary direction (Mirafi 5XT) in the engineered fill pad; the main direction of the selected uniaxial geogrid was to be laid radially towards the centre of the tank, and perpendicular to the tank wall. The lower level of geogrid was positioned at the base of the fill pad and the second layer 300 mm above the base. The design thickness of the engineered granular base is 1.5 m. The adopted analytical method to assess global stability was based on the assumption that a beneficial horizontal reinforcement force can be included in a simplified Bishop analysis [10–12]. A summary of the results of the global stability analyses which show that the critical failure surface, with respect to global stability, is given in Fig. 9. This figure also illustrates the differences in the properties of the

soil profile between the south and north sides of the tanks. The factor of safety of the critical failure surface with respect to global stability for the selected foundation design was determined to be 1.54.

The beneficial effect of the geogrid reinforcement on increasing the bearing capacity of the granular pad with regard to the perimeter wall foundations was evaluated using methodology given by FHWA [13]. This was adequate to meet design criteria.

Construction of the Foundation

Construction of the foundation consisted of removing the existing site surface layer of asphaltic concrete pavement as well as the immediately underlying zones of the upper fill layer to a depth of 1.5 m below the base of the tank foundation. The excavation phase was followed by dense compaction of the material exposed in the base of the excavation to its maximum practical density, which was carried out using a smooth drum roller with a static load of 10 tonnes under full time engineering control. Satisfactory compaction of the base was assured by visual engineering inspection and in situ testing with a hand operated static penetrometer. The base layer of geogrid reinforcement was laid in a radial pattern on the floor of the excavation beneath the footprint of the tanks and extending outside the footprint of the wall footing for a width of 3 m. A plan of the layout of the geogrid sheets is given in Fig. 10, and an image of the emplaced geogrid is given in Fig. 11. A 300 mm thick layer of the local, excavated granular fill adjudged suitable by the Engineer's on-site representative was laid on the base geogrid and compacted to 100 % of the material's standard Proctor maximum dry density. The second layer of the geogrid was laid on the emplaced lift of compacted fill to the same pattern and dimensions as the lower sheets of reinforcement. Above the reinforcement, the grade was raised to the design elevations using local suitable granular fill which was compacted to 100 % of material's standard Proctor maximum dry density.

Construction of the reinforced concrete tank wall foundations and tank base proceeded on top of the geogrid reinforced, densely compacted pad of engineered fill.

Fig. 8 Results of settlement analysis

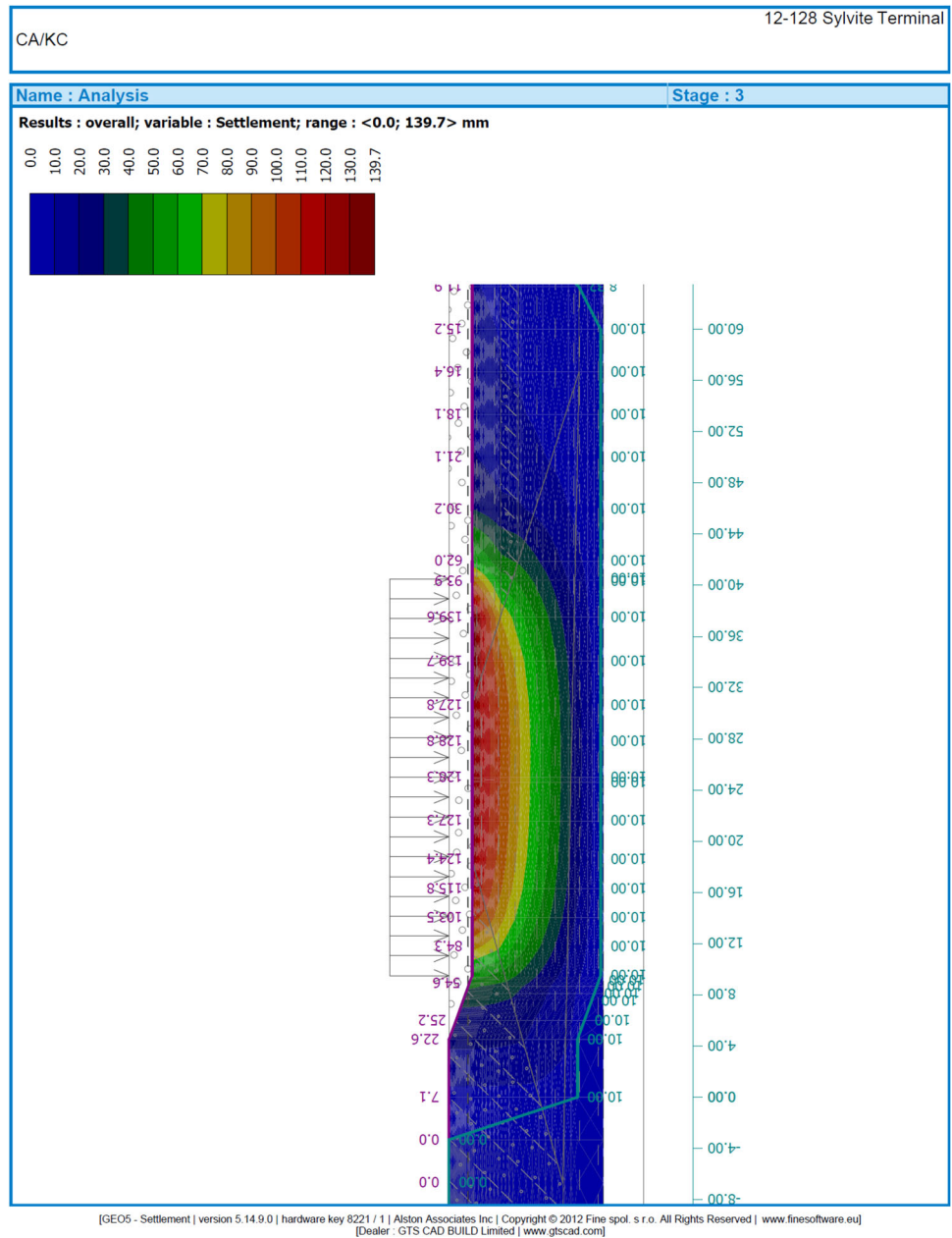


Table 2 Estimated settlement in design calculations

Depth range	Average increase in vertical stress (kPa)	Constrained modulus (MPa)	Consolidation settlement (mm)
0–4	60	4.5	53
4–5.5	55	20.0	4
5.5–8.5	50	10.0	15

Performance

Reference to the results of the settlement analysis indicates that the predicted consolidation settlement of the supporting soil profile at the southern limit of the tank wall

foundation was approximately 55 mm, and about 85 mm at the northern limit. The difference between the two estimated settlement values is attributed to the result of the effects of previous loading of the site by aggregate stockpiles. Actual settlement values were measured on the

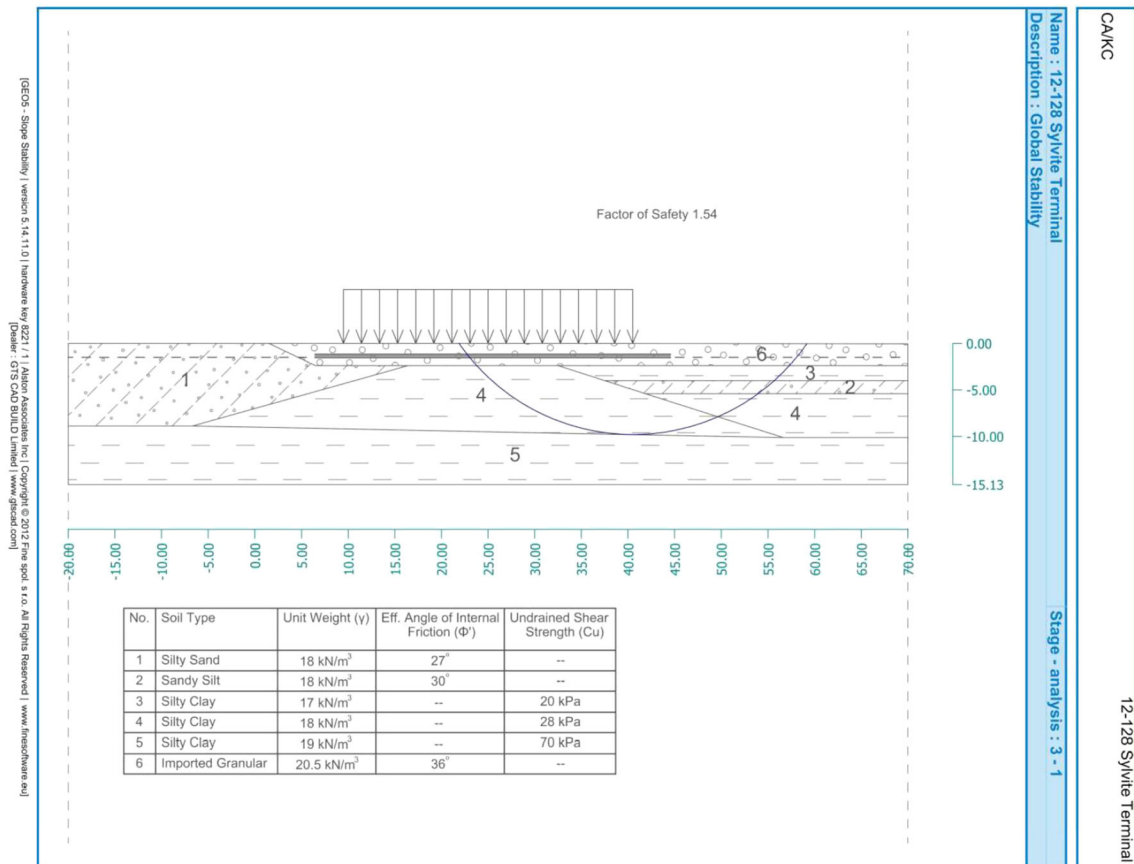


Fig. 9 Global stability analysis

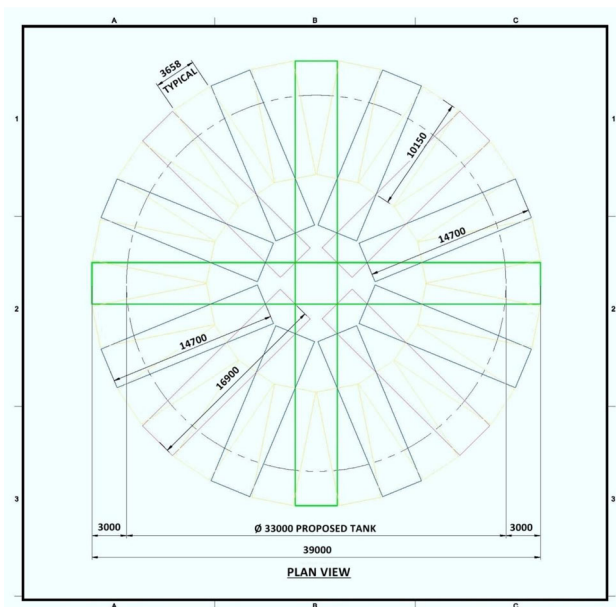


Fig. 10 Plan view of geogrid layout

foundation wall of one of the tanks and those results are given in Table 3.

The measured values show settlements of about 70 and 80 % of the theoretical predictions for the north and south sides of the tank, respectively. The underlying soils consist of loose (fill) or soft to firm (silty clay) materials and some ongoing creep settlement is expected. The actual settlement of the wall foundations falls within a reasonable proximity of the predicted values.

The cost premium of the adopted reinforced soil pad foundation system constructed on the site compared with a conventional footing foundation under the tank walls and a granular base beneath the tank floor was about \$200,000.00 for the two tank foundations. The estimated cost premium of a foundation comprising either a system of end bearing piles or stone columns was more than \$500,000.00 above the cost of a footing foundation. Thus, the selection of the reinforced pad foundation resulted in a cost saving of about 60 % compared to the initially recommended foundation system.

An image of the completed tanks are given in Fig. 12.

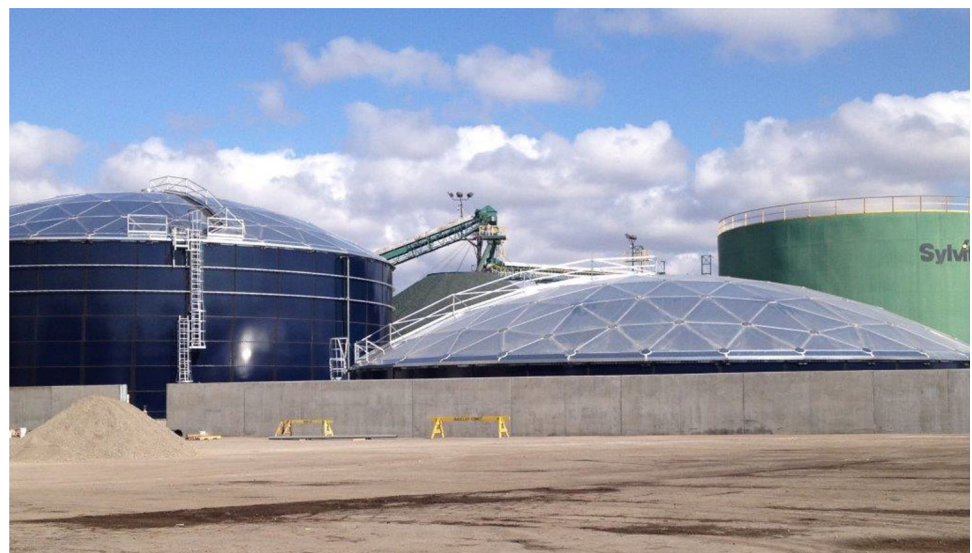


Fig. 11 Geogrid reinforcement on excavation base

Table 3 Settlement of perimeter ring beam foundation

Time from first load application	Predicted settlement		Measured settlement	
	North side (mm)	South side (mm)	North side (mm)	South side (mm)
1 week	–	–	9	9
1 month	–	–	25	23
1 year	80	50	54	41

Fig. 12 Completed tanks



Conclusions

- (1) Characterization of strength and deformation parameters of the supporting soil strata by an in situ test method (Marchetti flat dilatometer) has resulted in good characterization of the soil properties.
- (2) Construction of an engineered pad of reinforced granular fill above layers of relatively weak, compressible fill and native soils resulted in the foundation experiencing only moderate differential settlement, which is well within tolerable limits.
- (3) Design carried out using conventional limit equilibrium methods which account for the beneficial effect

of geosynthetic reinforcement, and using conventional one dimensional consolidation theory are shown to adequately describe the site and result in a foundation performance which is satisfactory in service.

- (4) The adoption of a foundation design utilizing the tensile strength of geogrid reinforcement enabled a cost savings of about \$300,000.00 on a previously estimated cost premium of about \$500,000.00 for a foundation supported by either end bearing piles or stone columns.

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