Use of polymer Geogrid reinforcement in the foundations of ground supported tanks for control of differential settlement

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ABSTRACT
This paper presents a case study wherein compacted granular foundation pads reinforced with polymeric geogrid were used to support pre-fabricated water storage tanks at a site in northern Manitoba, Canada. The subsurface strata at the site comprised thin layers of peat and silt underlain by sand and silt. Significant amounts of delayed settlements due to consolidation of silt and degradation of permafrost were anticipated. The differential settlements could be controlled by reinforcing the granular pad with polymer geogrids. The geogrid reinforced soil raft was to provide an environmentally friendly design as to reduce the permanent construction impact on the Keewatinoow land region. The foundation system avoids the use of large piles and concrete slabs. This allows for the easy dismantling of the foundation system and the construction site return to its natural state, after the short-term needs of the project have expired. The project consultants evaluated various alternatives for providing a safe, serviceable and environmentally friendly foundation for the tanks. The Horizontal Foundation Mat System turned out to be the simplest and most economical foundation solution. Initial observations indicate the performance of the system to be satisfactory and periodic monitoring of settlements will be carried out.

RÉSUMÉ
Le présent article décrit une étude de cas au cours de laquelle des dalles de fondation granuleuses compactées, renforcées avec de la géogrille polymère, ont été utilisées pour soutenir des réservoirs d'eau préfabriqués sur un site dans le nord du Manitoba, au Canada. De minces couches de tourbe et de limon reposant sur du sable et du limon forment les couches de subsurface du site. Des quantités importantes de tassements retardés par la consolidation du limon et la dégradation du pergélisol étaient anticipées. Les tassements différentiels peuvent être contrôlés en renforçant la dalle granuleuse avec de la géogrille polymère. Une fondation sur radier renforcé par la géogrille constitue une solution écologique qui diminue les répercussions permanentes de la construction sur le territoire de la région de Keewatinoow. Le système de fondation évite l’utilisation de grandes piles et de dalles de béton. Cette approche permet un démantèlement facile du système de fondation et le retour du terrain de construction à son état naturel une fois les besoins à court terme du projet sont répondus. Les conseillers du projet ont évalué différentes solutions pour fournir une fondation sécuritaire, fonctionnelle et écologique aux réservoirs. Après analyse, le système de fondation horizontale sur radier constitue la solution la plus simple et économique en matière de fondation. Selon les premières observations, le rendement du système est satisfaisant, et les tassements continueront de faire l'objet d'une surveillance périodique.

1 INTRODUCTION

The Keewatinoow start-up camp, which is located approximately 75 km north of Gillam, Manitoba, required two domestic water storage tanks and two fire water storage tanks. All four tanks were pre-fabricated tanks with a diameter of 7244 and height of 5126 mm and were to be directly supported on the ground (see figure 1).

Figure 1. Aerial view of the completed facility

The safe bearing capacity and settlement are two important geotechnical considerations in the design of foundations of the tank. Very low ambient temperature at the time of construction was another important factor. Many alternatives were evaluated by the design consultants of the project and finally a geosynthetic reinforced granular pad was selected as optimum solution in terms technical adequacy, ease of construction and cost. This paper briefly reviews the subsurface conditions at the site, geotechnical design considerations for tank foundations and describes salient features of the design and construction.

2 GEOTECHICAL ASSESSMENT

2.1 Subsurface Profile

The subsurface investigations carried out at the site comprised excavation of three test pits with depths ranging from 4.6 to 6.7 m, collection of representative samples and determination of grain size distribution, Atterberg limits, water content and temperature of the soil.

The test pits revealed a soil profile of peat overlying silt and sand. Soil temperatures in the test pits decreased with increasing depth from approximately 12˚C at the ground surface to 5˚C at a depth of 6.7 m (on September 12, 2012). No evidence of permafrost was observed in soil samples obtained from the test pits.

The major subsurface strata as observed in the test pits are as follows:

- Peat extending to a depth between 0.1 m and 0.15 m below existing grade with water contents in the range of 20% to 131%.
- Silt was encountered beneath the peat and also beneath the sand layer in the test pits. The silt was tan to grey, soft to firm, moist, and of low plasticity. Beneath the peat layer, the silt extended to depths between 0.3 m and 0.5 m. Water contents of the silt ranged from 11% to 18%.
- Sand was encountered below the silt at depths between 0.3 m and 0.5 m in the test pits and extended up to a depth of 2.7 to 3.5 m below existing grade. The sand was tan, compact and moist and contained some gravel and infrequent cobbles. Water contents of the sand ranged from 7% to 13%.
- Silt was again observed below depths ranging from 2.7 to 3.5 m below existing grade up to the depth of exploration (4.6 – 6.7 m). The silt was low plasticity with some sand and clay and with water contents in the range of 14 – 18 %.

Ground water seepage from the sand layer was observed at depths between 1.8 and 2.7 m in all the test pits.

Although no evidence of permafrost was observed in the soil samples obtained from the test pits, the project site is located within the zone of sporadic discontinuous permafrost and consequently, isolated zones of permafrost may be present on the project site. Evidence of permafrost was observed in two boreholes drilled 200 m to 300 m east of the project site in September 2011.

2.2 Recommendations of Geotechnical Investigation Consultant

The geotechnical investigation agency had recommended the removal of superficial peat and silt layers (thickness ranges from 0.3 to 0.5 m) to expose the underlying sand layer.

The consolidation settlement within the silt could not be predicted accurately but it was estimated that it could be in the range of 40 to 100 mm. It was recommended that flexible connections should be provided on piping and connections to accommodate potential differential movement related to consolidation settlement.

Although no evidence of permafrost was observed in the test pits, ice crystals were observed in boreholes previously drilled near the project site. Settlement due to permafrost degradation may occur during the service life of the facility. However, the magnitude of settlement related to permafrost degradation cannot be determined.

2.3 Assessment of Safe Bearing Capacity

The unfactored uniform bearing pressure below the tank is 50 kPa and the unfactored perimeter line load is 10 kN/m. The tank is placed on a 0.3 m thick gravel berm which is confined within a grade band. Considering a unit weight of 20 kN/m^3 for the gravel, the bearing pressure imposed at the ground level is 56 kPa.

The subsurface profile below can be represented as a two layer case wherein a 3.0 m thick layer of sand is underlain by silt, with the possibility of ground water rising up to ground level. The sand layer can be idealized as a cohesion-less soil exhibiting a drained behavior with the following properties – effective cohesion = 0, effective angle of shearing resistance = 30° and submerged unit weight = 9 kN/m^3. The silt can be considered as a purely cohesive soil showing an undrained response with the following properties – undrained cohesion = 35 kPa, undrained angle of shearing resistance = 0 and saturated unit weight = 19 kN/m^3.

The ultimate bearing capacity was estimated using the method presented by Das (2011) for the case of a strong cohesion-less stratum underlain by a weak cohesive stratum. The calculations indicated an ultimate bearing capacity of 234 kPa and with a factor of safety of 3, the safe bearing capacity was estimated to be 78 kPa. Since the safe bearing capacity is higher than the actual bearing pressures, the risk of a shear failure of the ground is not significant.

2.4 Settlement Considerations

A good review of criteria for settlement of tanks is presented by Marr, Ramos and Lambe (1982). The major aspects to be considered are:

- Planar tilt
- Non-planar settlement
- Dish type settlement
- Localized settlement

The probable caused for non-uniform settlements include:

- Non-uniform geometry or compressibility of the soil deposit
- Non-uniform distribution of load applied to the foundation
- Uniform stress acting over a limited area of soil stratum

While there are different criteria proposed by different agencies and researchers, a reasonably conservative criterion is to limit differential settlements to 1% of the tank diameter. With this criterion, for a tank diameter of 7244 mm, the permissible differential settlement is 72 mm.

As per the recommendations of the geotechnical investigation consultant, total settlements could be in the range of 40 to 100 mm. It can be conservatively assumed that the maximum differential settlement would be limited to 75 % of total settlement. Considering a maximum total settlement of 100 mm, maximum differential settlement is unlikely to exceed 75 mm. Hence, differential settlement can be expected to be within permissible limits. Thus, if the ground conditions are reasonably uniform, a smooth dish type settlement profile can be expected, with differential settlements within permissible limits for the tank.

From a structural point of view, localized settlements could be the most harmful. Excessive deformations due to local soft spots within the ground could occur to some extent. Another factor of concern is the settlement due to degradation of permafrost, a phenomenon which is likely to be localized. It is not easy to predict the probability of occurrence and magnitude of such localized settlements.

2.5 Geotechnical Design Considerations

Based on a review of the available information on subsurface conditions at the site and the loading imposed by the tank, it was concluded that the imposed bearing pressures was less than the safe bearing capacity and for a smooth dish type settlement profile, the differential settlement is likely to be within permissible limits. However, the probability of occurrence of localized settlements of uncertain magnitudes – due to localized weak spots and also possibly due to degradation of permafrost was assessed to be potential problem, which could adversely affect the structural integrity and performance of the tanks.

3 PROPOSED SOLUTION

3.1 Geosynthetic Reinforced Granular Raft

Geosynthetic reinforcement of soils have been used extensively for a wide range of applications – reinforced soil walls and slopes, basal reinforcement of embankments on poor ground, subgrade stabilization, reinforcement of unbound aggregates in the subbase and base courses of pavements, reinforcement of rail road track ballast, reinforced foundation beds, spanning of voids in sinkhole areas etc. Although extensive amount of theoretical and experimental research has been carried out on reinforced foundation beds, the technique is being used on a far limited scale in comparison to other more popular applications. Lack of authoritative codes of practice or guidelines for design and limited number of successful published case studies could be some of the reasons why the technique is being used on a very limited scale in many parts of the world. Nevertheless, there have been many cases, where geosynthetic reinforced soil rafts or pads have been successfully used to support the foundations of structures.

Here, the there is no significant risk of a bearing failure and overall differential settlements are also expected to be within acceptable limits. The major geotechnical design consideration is to provide measures to limit the effects of potential localized settlements. The expected function of the geosynthetic reinforcement is to smooth out localized settlements and ensure a smooth dish type settlement profile. To some extent, the function is similar to void spanning in areas prone to sink holes (Karst terrain), an application where geosynthetics have been used successfully.

3.2 Selection of Geosynthetic Reinforcement

It was proposed to reinforce the granular pad with two layers of HFM-BL2PP Horizontal Foundation Mat (supplied by Titan Environmental Containment Ltd). HFM-BL2PP is manufactured out of virgin polypropylene by a unique state-of-the-art punching and drawing process. The HFM-BL2PP Horizontal Foundation Mat is produced from an extruded sheet of polypropylene that is punched and then stretched in both machine and cross-machine directions. The HFM-BL2PP thus formed is monolithic and having integral nodes. This Horizontal Foundation Mat has high tensile strength and tensile modulus, integral junctions with high junction strength, especially high tensile stiffness at low very low strains of 0.5% and 1.0% , high flexural stiffness, high tensile stiffness and aperture stability thereby ensuring a high level of positive interlock with soil and aggregate.

The Horizontal Foundation Mat reinforcement provides a high level of confinement to the soil and hence overall stiffness of the granular pad is greatly enhanced. The tensile strength of the Horizontal Foundation Mat enables it to span over localized weak spots thereby redistributing stresses and smoothing out uneven settlements. The Horizontal Foundation Mat reinforced granular pad acts as a flexible raft, which appreciably reduces any localized uneven differential settlement.

3.3 Grade Band, Gravel Pad and Erosion Protection

The tanks would be placed on a compacted gravel berm contained in a retainer ring called a Grade Band, which is fixed to bolt channels embedded into ground. A ¼" thick by 18" wide laser cut and perfectly fitted steel bearing plate would be positioned on the level gravel pad so that the tank walls will sit directly in the middle of the plate. This would help to distribute the weight of the tank walls and roof to the granular foundation below. An 8oz non-woven geotextile TE-8(supplied by Titan Environmental Containment Ltd) would be placed above the bearing plate to cover the entire surface of the gravel berm. A layer of fiber board is placed over the geotextile to serve as cushion and protection for the coated steel floor plates.

One of the major purposes of providing the gravel berm with geotextile is to prevent the erosion caused by the rain water running off the roof of the tank (see figure 2). If the gravel berm is exposed, the aggregate particles would be washed away and an erosion ditch will form over the years around the tank walls. As a result of this slow erosion process, the walls of the tank will sink farther and farther into the trench, inside floor plates will suffer extreme distortion and eventually floor seams could fail. The level gravel pad which is confined within the grade band and completely covered with geotextile provides adequate protection against erosion and prevents such failure of the tank. The clean gravel also provides a non-organic and sterile finish around the tank to discourage unwanted growth of vegetation and possible pests.

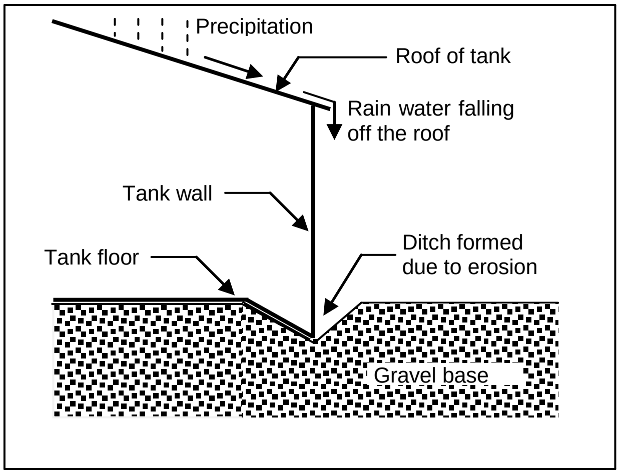

Figure 2. Mechanism of erosion around the tank

3.4 Proposed Layout of the Reinforced Raft

The layout which was adopted is shown in figure 3.

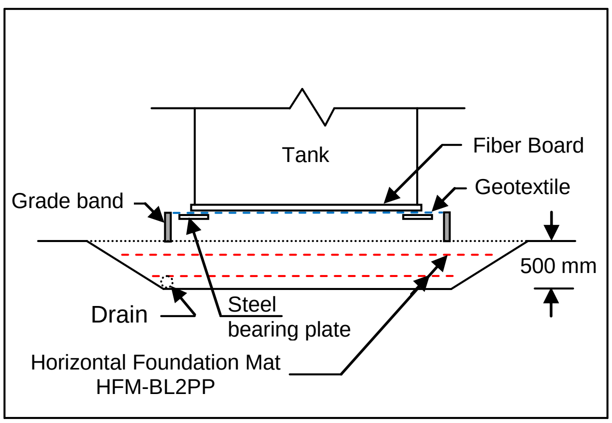

Figure 3. Schematic c/s of reinforced granular raft

The tanks are supported on a 300 mm thick gravel pad which is confined within a steel retainer ring (grade band) which is bolted to channels embedded into the ground. A fibre board and non-woven geotextile are placed between the tank bottom and top of gravel pad. The existing soils are excavated up to a depth of 500 mm below existing grade and replaced with compacted granular fill, which is reinforced with two layers of HFM-BL2PP Horizontal Foundation Mats. The bottom mat is placed at a depth of 350 mm below existing grade grade and the top mat is placed at a depth of 150 mm below existing grade. Adjacent panels or rolls were overlapped by 450 mm.

3.5 Construction

The ground was excavated to the required depth and diameter. The bottom of excavation and made level and rolled. Select granular fill was placed and compacted to a thickness of 150 mm and density corresponding to 98 % of standard Proctor maximum dry density. The first layer of HFM-BL2PP was laid without wrinkles and folds. Adjacent rolls were overlapped by 450 mm. Sand-filled bags were placed along the edges of the mat to hold it in position (figure 4). Granular fill was spread over the mats and compacted to a thickness of 200 mm. The second layer of HFM-BL2PP was laid and 150 mm thick layer of fill was compacted over it. No, special equipment or expertise was required for the installation of mats. The installation was easy and not affected by the extreme weather conditions.

The grade band, gravel pad, non-woven geotextile and fibre board were installed over the reinforced granular raft (figure 5).

Figure 4. Laying of Horizontal Foundation Mat HFM-BL2PP

Figure 5. Construction of grade band completed and steel bearing plate installed.

4 PERFORMANCE

The proposed scheme was used to improve the foundation of all four tanks at the site. The construction was completed towards the end of 2012. No indications of shear failure or excessive settlements so far. However, the real function of the reinforcement is to control excessive localized settlements in the long-term. A plan for periodic monitoring of settlements is in place. It is hoped that long-term settlement measurements would offer further evidence to show the effectiveness of this system.

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