DESIGN OF FUEL TANK FOUNDATIONS ON SOFT CLAY DEPOSIT

LI Allen Lunzhu[1]，YIN Guangzhi[2]，XU Jiang[2]
(1. *Key Laboratory for the Exploitation of Southwest Resources and the Environmental Hazards Control Engineering of Ministry of Education*，*Chongqing University*，*Chongqing* 400030，*China*；2. *College of Resources and Environmental Science*，*Chongqing University*，*Chongqing* 400030，*China*)

Abstract：A foundation design method and considerations for two large-sized fuel tanks on soft marine clay deposit in Attawapiskat，located in the James Bay coastal area of Northern Ontario，Canada are describes. The tanks with diameter of 29 m and height of 12 m are required for fuel supply for a diamond mine，located approximately 100 km west of Attawapiskat. Each tank has a volume capacity of 7.5 million litres；and the maximum tank pressure of 140 kPa is exerted. The design involving a mat foundation is introduced. The design criteria require adequate safety margin against potential failure(bearing pressure and edge pressure) and relatively stringent settlement(differential) limits. Other design considerations include frost effects，limited availability of granular and rockfill materials，short construction period，and spill containment. Stability assessment indicates the edge failure to be more critical than the bearing capacity failure mode，and emphasizes the need for soil improvement. The uses of geosynthetic reinforcement along the perimeter of the tank and prefabricated vertical drains(PVDs) in the foundation soil in conjunction with site preloading are expected to reinforce the foundation，accelerate consolidation and reduce post-construction settlement. The design studies confirm the feasibility of constructing a mat foundation within short time，provided that the ground improvement measures mentioned above were incorporated.

Key words：foundation engineering；fuel tank；soft clay；foundation；prefabricated vertical drain；settlement；consolidation；design

CLC number：TU 47　　**Document code**：A　　**Article ID**：1000－6915(2007)08－1561－11

大型油罐软土地基设计

李伦竹[1]，尹光志[2]，许　江[2]
(1. 重庆大学 西南资源开发及环境灾害控制工程教育部重点实验室，重庆　400030；2. 重庆大学 资源及环境科学学院，重庆　400030)

摘要：描述大型油罐软土地基设计方法。考虑建在海洋粉质黏土地基之上的油罐，位于加拿大安大略北部沿 James Bay 湖边的一个小镇 Attawapiskat。2 个油罐直径为 29 m，高 12 m，为 Attawapiskat 以西大约 100 km 处的一个钻石矿提供燃料。每一个油罐具有 7.5×10^6 L 的存储容量，并且发挥 140 kPa 的最大油罐压力。考虑软土处理和席形基础的设计，针对潜在的整体地基剪切破坏和局部地基剪切破坏设计标准要求充分的安全系数，并达到严格的不均匀沉降限制。其他设计考虑包括冻结效应、有限的粗粒和堆石料、短施工时间和燃料溢出保护。稳定性分析表明，油罐边界局部基础剪切破坏模式比整体基础破坏模式更关键，同时，强调对于软土改进的需要。地基处理利用土工合成加筋材料加强油罐外围基础，利用预制竖排水管加速地基固结沉降，并结合预压软土降低工后沉降。

Received date：2006－11－28；**Revised date**：2007－06－22

Corresponding author：LI Allen Lunzhu(1964－)，male，a professional engineer in Ontario，Canada，obtained his doctoral degree in soil mechanics from the University of Western Ontario(2002) and pursued his post-doctoral research at Queen′s University，Kingston，Ontario，Canada(2002－2004). E-mail：allenli@rogers.com

设计研究确认在短期内建造大型油罐地基的可行性，但要结合上述的软基处理技术。
关键词：基础工程；油罐；软土；地基；预制竖排水管；沉降；固结；设计

1 INTRODUCTION

A fuel tank farm in Attawapiskat is required for the development of Victor Diamond Mine Project，located approximately 100 km west of Attawapiskat in the James Bay west coastal area of Northern Ontario，Canada(Fig.1). The construction schedule requires that the fuel tank farm be operational approximately after 6 months prior to the start-up of the construction of the Victor Diamond Mine. The proposed tank farm consists of two large size tanks with a total volume capacity of 15 million litres of diesel fuel. Each tank has a diameter of 29 m and height of 12 m，with a volume capacity of 7.5 million litre(ML). They will exert a maximum base pressure of 140 kPa under full tank condition. The fuel tanks will have fixed roofs supported on the perimeter wall and internal columns.

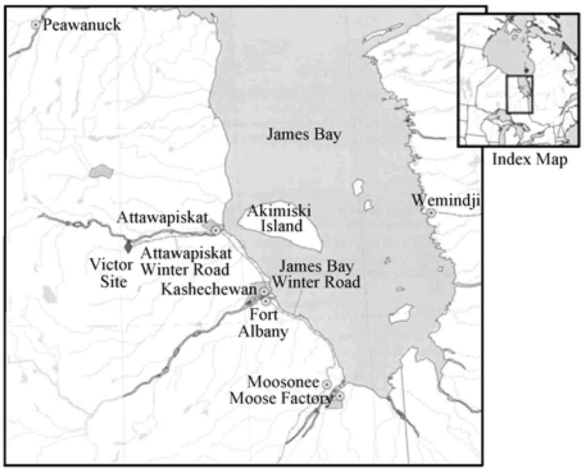


Fig.1　Locations of Victor Diamond Project and the community of Attawapiskat

The tanks will be kept least 50% full throughout the winter months to ensure fuel supply for the construction site. Since the foundation soils would frost and heave in winter，it is advantageous to keep the tank 50% full at least to have heat transfer to the ground in order to reduce vertical differential displacements of foundation due to frost. The settlement criterion allows the maximum differential settlement between the tank centre and edge to be within 25 to 75 mm over the tank radius of 14.5 m. The total settlement will be accommodated using flexible pipe connections or periodically repositioning the pipe supports.

The Attawapiskat is located at the borderline between discontinuous permafrost and non-permafrost zones. The site is underlain by Holocene silty clay deposited under Tyrrell Sea following the retreat of the last glaciers. Beneath a relatively thin weathered crust，the plastic marine clay is generally soft with undrained shear strength between 20 and 38 kPa. The thickness of the deposit varies across the site. The soft clay foundation is a major consideration for the construction of the fuel tank farm. Both the shallow (granular mat) and the deep(pile) foundation options that have been considered present some problems. The mat foundation requires a significant amount of granular fill materials，which are not readily available at the site；and the deep foundation requires mobilization of a pile installation equipment，crew to this remote site and a significant number of steel H piles. The design method and considerations for the granular mat foundation option are introduced.

2 SITE CONDITIONS

2.1 Quaternary Geology

Attawapiskat is located on the west coast of James Bay within the southeastern part of the Hudson Bay Lowland. The overburden strata in Attawapiskat generally comprise Pleistocene glacial till，post glacial lake sediments and Holocene marine deposits with more recent alluvial sediments and peat at the surface. Two Pleistocene till sheets，separated by proglacial sediments，were identified in the Hudson Bay Lowland with the proglacial sediments being absent between Attawapiskat River and Severn River[1]. The glacial till overlies the Attawapiskat formation of Silurian siltstone bedrock. As the last Wisconsin glacier retreated，postglacial lakes were formed in the southeastern and northwestern parts of the Hudson Lowland(including the southern part of James Bay Lowland) with stratified silt and sand deposited over the upper till. About 7 900 years ago the region was invaded by the Tyrrell sea through Hudson Strait; and the sea water covered the James Bay Lowland in a rapid manner following the lacustrine episode[2]. The Holocene deposits were formed during the regression from the early postglacial Tyrrell sea[3]. The marine unit comprises dark-grey massive silty clay with some seashells and circular peat nodules. The emerged James Bay Lowland is covered by vast and flat peatlands of peat deposits with thickness up to 3 to 4 m within the inland fens and raised bogs. The peat deposits are generally thin and discontinuous near the coast.

2.2 General Site Conditions

The community of Attawapiskat is within the cold region in Northern Ontario，Canada with an annual average temperature of –2.3 ℃ and a frost penetration depth of approximately 3.0 m. The proposed fuel tank farm is located on the north side of Attawapiskat River and in the vicinity of the northeast corner of the existing sewage lagoons(Fig.2). The site is relatively flat，heavily treed and poorly drained although it is slightly higher than the surrounding terrain.

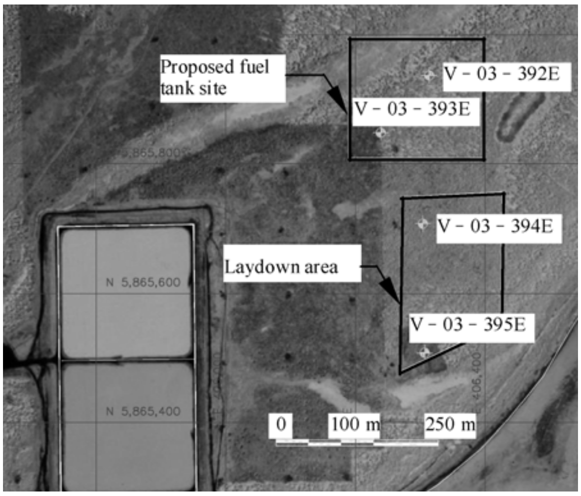


Fig.2 Borehole location plan

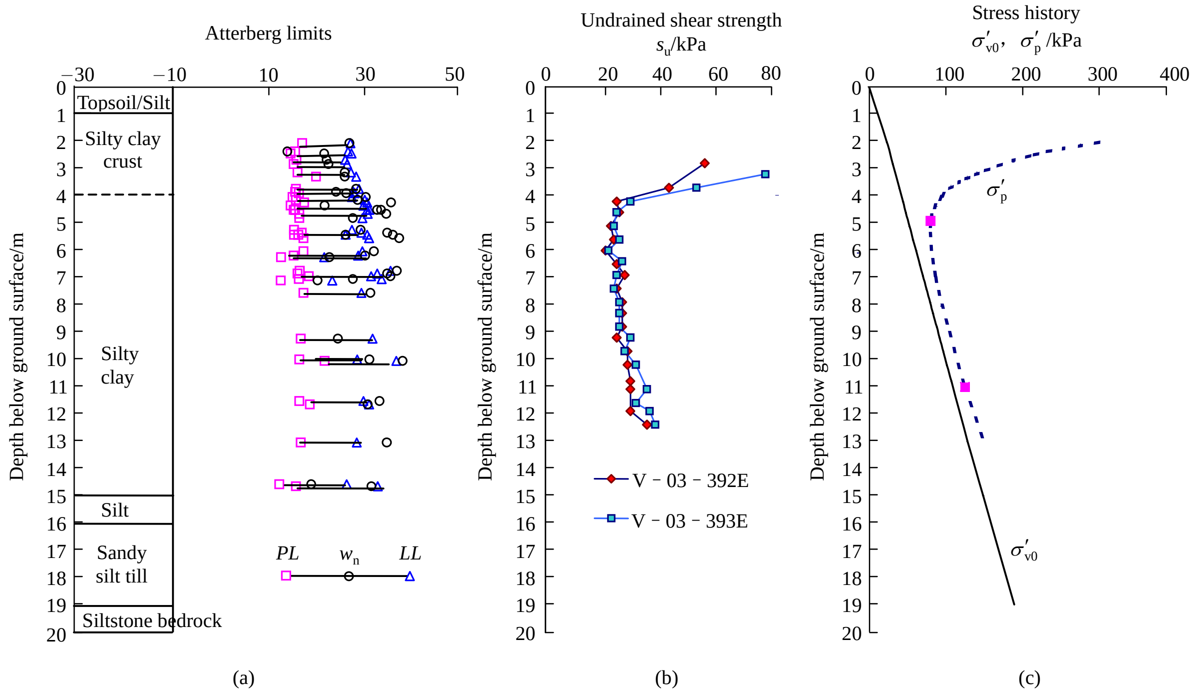
Geotechnical investigations were carried out in 2003 and 2004 to determine the subsurface conditions at the proposed locations of the fuel tank farm，laydown area and barge berth facility. Two boreholes (V－03－392E and V－03－393E) were put down at the proposed fuel tank farm area to depths of 22.9 and 25.9 m respectively. The borehole locations are shown in Fig.2. The investigation results indicated that the general soil profile consists of a relatively thin layer of organic deposit overlying 1.2 m of loose silt，about 13.5 m-thick marine silty clay，and then silt and silt till deposits.

Bedrock exists at the depth of 19.1 to 20.5 m and consists of reddish brown siltstone within the first 2 m-thick zones being highly weathered. The inferred soil stratigraphy at the fuel tank site is presented in Fig.3. The groundwater levels measured in piezometers installed in Boreholes V－03－392E and V－03－393E are 0.2 to 0.8 m below the existing ground surface.

2.3 Engineering Properties of Attawapiskat Clay

The marine clay stratum can be described as comprising three distinct layers，namely，approximately 3.0 m-thick firm to stiff crust at the top，followed by the soft to firm dark-grey main layer and a 2.0－3.0 m stiff layer at the bottom of this stratum. The 3 m-thick silty clay crust is weathered from freeze-thaw cycles. The natural moisture content of this layer varies generally from 13% to 28%. The in-situ and laboratory test results demonstrated that the silty clay has low plasticity and a firm to very stiff consistency with relatively low liquidity index. The extensive dark-grey soft clay deposit，encountered beneath the upper crust，comprises 0% to 15% sand，35% to 80% silt and 20% to 50% clay size particles，and contains traces of organics and seashells. The layer has plastic limit of 12 to 21，liquid limit of 21 to 36，plasticity index of 10 to 20，natural moisture content of 20% to 38% and a liquidity index of 0.7 to 1.5. Based on the Casagrande plasticity chart，the clay is classified as a low to medium plastic clay.

The undrained shear strength profiles obtained from Nilcon vane tests carried out at the fuel tank farm are shown in Fig.3. Nilcon vane tests were also carried out at different adjacent facility sites in Attawapiskat. All Nilcon vane data are compiled and presented in Fig.4，in which each symbol presents a borehole drilled in the different sites in Attawapiskat including



Where *PL* is plastic limit；w_n is natural water content；*LL* is liquid limit；s_u is undrained shear strength；σ'_{v0} is initial vertical effective stress；and σ'_p is preconsolidation pressure.

Fig.3　The undrained shear strength profiles obtained from Nilcon vane tests carried out at the fuel tank farm

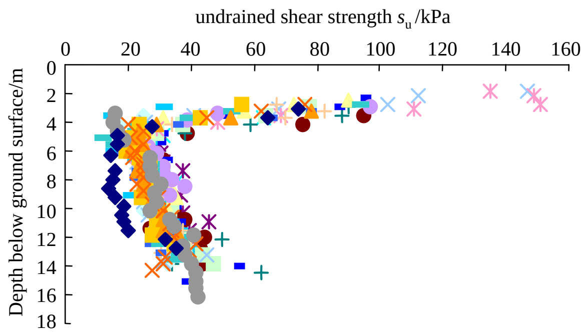


Fig.4　Field vane strength profile at the Attawapiskat site

the fuel tank farm，barge berth and laydown areas shown in Fig.2. As shown in Fig.4，the undrained shear strength of the crust layer ranges from 30 to 150 kPa；and the strength decreases rapidly with depth in the first 4 m. Undrained shear strength of the main silty clay stratum(underlying the crust layer) increases with depth from about 20 to 30 kPa at 4 to 6 m depth to about 30 to 50 kPa at 14 to 16 m depth. The sensitivity varies between 4 and 8，indicating a medium sensitivity. Based on plasticity index of generally 10 to 20，the Bjerrum correction factor for the field vane test results is considered to be approximately 1.0.

Fig.4 also shows an exceptionally low undrained shear strength profile measured at the location of borehole V－03－395E drilled through a 3.1 m-thick filled mound in the laydown area(see Fig.2). The strength within the main silty clay layer at this particular location ranges from about 14 to 20 kPa，which is unusually low compared to the trend observed at other test locations at Attawapiskat. This variation in undrained strength is a concern during the design study and is further investigated through additional Nilcon vane tests，laboratory tests and investigation of the history of the existing filled mound. The conclusion of the investigation suggests that the undrained shear strength anomaly is likely to be caused by over-stressing from the fill material，which was initially stockpiled up to about 6 m high. This loss of strength confirms the foundation design concerns at this site.

The ratio of undrained shear strength(lower range of values) to the existing vertical effective stress，s_u/σ'_{v0}，is in excess of 1.0 within the crust layer，and varies approximately from 0.5 to 0.3 at the transition zone between the brown crust and the grey soft to firm deposit(at depth of 4 to 7 m)，and from 0.3 to 0.2 below that. Conventional oedometer(one-dimensional consolidation) tests were carried out on selected Shelby Tube samples recovered from boreholes V－03－392E and V－03－393E. The interpreted consolidation characteristics are summarized in Table 1. Two values of the ratio of undrained shear strength to the preconsolidation pressure，s_u/σ'_p (based on results of oedometer tests，discussed hereafter)，were found to be 0.26 and 0.29.

Table 1　Summary of one-dimensional consolidation test results

Borehole and soil sample number	σ'_{vo} /kPa	σ'_p /kPa	Recompression index C_r	Compression index C_c	Consolidation coefficient C_v /(m^2 • d^{-1})
V－03－392E，TW8 (4.9 m)	50	78	0.04	0.33	0.010－0.018
V－03－393E，TW14 (11.0 m)	110	120	0.03	0.29	0.018－0.033

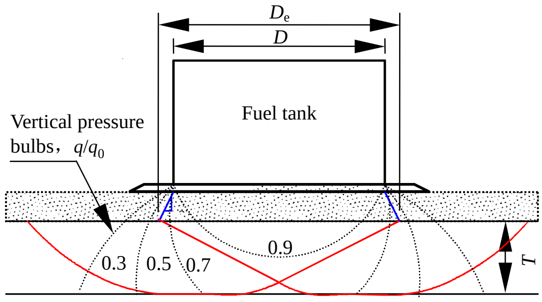
As shown in Table 1，the preconsolidation pressures for the soft clay obtained by the consolidation tests are 78 and 120 kPa at the depths of 4.9 and 11.0 m respectively. The corresponding over-consolidation ratio(*OCR*) is 1.6 at the depth of 4.9 m and 1.1 at the depth of 11.0 m. These OCR values suggest a slightly overconsolidated condition at shallow depth and a close one to normally consolidated condition near stratum bottom. Many researchers[4, 5] have suggested that all the old clay deposits，which generally consider normal consolidation，have been slightly overconsolidated because of aging and the long-term slow creep process during thousands of years of the stratum history，resulting in lower void ratio under a constant overburden pressure. This overconsolidation condition is generally not evident because of the influence of sample disturbance. The results shown in Table 1 have provided a basis to select design parameters used in the settlement and consolidation analyses for the design of fuel tank foundations.

3　DESIGN CONSIDERATIONS

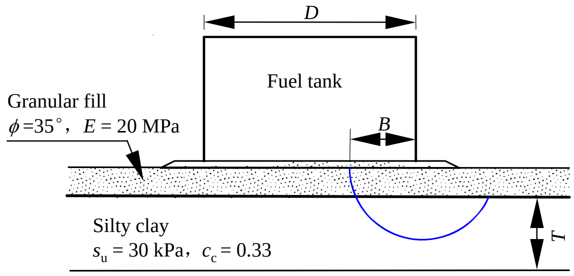
The investigation results have shown the soil underlying the fuel tank site in Attawapiskat to be weak and compressible. In the literature，there are abundance of case studies in which fuel tanks were successfully constructed on soft and compressible foundations[6~9]，provided that foundation soils were improved adequately. Foundation soils are often improved through installation of prefabricated vertical drains[6, 7] to increase the stability and reduce settlement. Case studies also indicate that the rupture of tanks may occur due to foundation instability and/or severe distortions of tanks due to differential settlement. Failure of the fuel tank may result in potential environmental damage due to fuel spill and economic loss due to tank damage and impact on the project schedule and operations. Foundation stability and consolidation settlement are the principal design considerations. The time constraint is also an important factor in the design considerations. For the Victor Project，the overall construction schedule requires that the fuel tank farm should be constructed and ready for the fuel supply within 6 months.

3.1　Foundation Stability

From case studies of tank foundation failures，two failure modes of tank foundation have been observed[7]，namely，tank base shear failure and tank edge shear failure(see Fig.5). The instability of tank base can arise from bearing capacity failure of foundation



(a) Base failure mode



(b) Edge failure mode

Fig.5　Failure modes of tank foundations [4] soils，which is similar with the failure mechanism of flexible footings on foundation soils. The edge shear failure is caused by the local slip failure of foundation under the weight of tank and fuel at the edge，which is similar with rotational slope failure mechanism. The governing failure mode will depend on the size of the tank related to the thickness of soft soil and rigidity of the shallow foundation. For a tank foundation with a relatively thin soft deposit and undrained shear strength increasing with depth，the edge shear failure is likely to be the critical one. One of the reasons for this is that the bearing capacity of the footing increases due to the increase of undrained shear strength with depth and the presence of a hard stratum within a finite depth[10].

Use of granular mat was considered to spread the load from the tank and diesel fuel over a large area on the soft foundation soil. However，the excessive fill above the original ground surface can reduce the factor of safety against base failure due to the increased pressure on the soft clay from the weight of the fill materials. The thickness of the fill was determined based on the following factors：(1) the thickness of the organics and loose/soft deleterious surficial soils；and (2) the thickness required for base and edge stability considerations. At the fuel tank site，the average thickness of the organic and loose silt is about 1.2 m. Therefore，after removal of the surficial soils，the granular fill would be placed directly on the silty clay crust with a thickness of 1.2 m. The required thickness of fill above the original ground was determined based on the following design approaches.

For the base stability，the granular fill should be thick enough so that the bearing capacity of the multi-layer soil foundation system(i.e. granular fill layer，native crust and the underlying soft clay) is adequate to sustain tank pressures. A fill thickness of 1.7 m，with 0.5 m above the original ground and 1.2 m below the original ground，was considered in the design. Geosynthetic reinforcement was considered to reinforce the granular mat and improve the edge stability of the tank foundation. The tensile force mobilized in the geosynthetic reinforcement can also provide additional resistance to the rotational slip of the foundation soils at the tank edge. The factors of safety against the base failure and edge failure modes were calculated according to the design approach proposed by J. M. Duncan and T. B. D′Orazio[7]. In this approach，which is based on observed behaviours of tank foundations，the factor of safety against base failure was calculated using a 2(vertical)∶1(horizontal) stress spread in the granular mat(as shown in Fig.5) and the ultimate bearing capacity of a uniform soft clay deposit with average undrained shear strength. To examine the effects of the increase of undrained shear strength with depth and finite depth of the hard layer，the bearing capacity of granular mat foundation was also calculated using the bearing capacity charts summarized by R. K. Rowe and K. L. Soderman[10]. In addition，the method proposed by G. G. Meyerhof and A. M. Hanna[11] for a thick sand layer on a weak clay foundation under a circular footing was also used to determine the ultimate bearing capacity and account for the potential punching failure of the two-layer foundation system. In analyzing the factor of safety against edge failure，the approach proposed by J. M. Duncan and T. B. D′Orazio[7] was modified to incorporate the tensile force mobilized in geosynthetic reinforcement.

J. M. Duncan and T. B. D′Orazio[7] recommended that the minimum factor of safety should be 1.3 or greater for tank foundations with the minimal uncertainty and the failure not involving a risk to life or catastrophically economic loss. Considering the proposed tanks with potential risks，the factor of safety of 2.0 was selected as a criterion to determine the allowable bearing capacity of soil. The calculated factors of safety for the 29 m(diameter)×12 m(high) tanks founded on granular fill foundations was 1.3 to 1.4 for the base failure mode and 1.3 for the tank edge failure mode as shown in Table 2. In view of these low factors of safety，an alternative fuel tank size，with a diameter of 29 m and a height of 6 m(which are readily available from the manufacture) was examined. These smaller tanks exerted an average base pressure of 85 kPa in accordance with the manufacture specifications. The calculated factors of safety for the 6 m-high tank ranged from 2.1 to 2.2 for the base failure mode and 2.0 for the edge failure mode(shown in Table 2). This indicated a significant improvement

Table 2　Factors of safety for base and edge failure modes

Researchers	Base failure mode		Edge failure mode	
	12 m-high tank	6 m-high tank	12 m-high tank	6 m-high tank
J. M. Duncan and T. B. D′Orazio[7]	1.3	2.1	1.3	2.0
R. K. Rowe and K. L. Soderman[10]	1.4	2.2	–	–
G.G.Meyerhof and A. M. Hanna[11]	1.3	2.0	–	–

over the 12 m-high tank case. The design analyses suggested that only 6 m-high tanks founded on granular fill foundations would have adequate safety margin against foundation failure with soil improvement using geosynthetic reinforcement alone.

However，the smaller tanks only have half of the fuel volume capacity of the larger fuel tanks. If the smaller tanks were to be used，four tanks would be required in order to provide the required quantities of diesel fuel. The large tank is more desirable since it needs less expensive to be constructed and requires less acreage. Therefore，as discussed in the later section in this paper，consideration was given to ground improvement to increase the allowable bearing pressures of foundation soils through preloading the drainage-enhanced soft clay using prefabricated vertical drains.

3.2　Settlements

Steel tanks are generally flexible and ductile and therefore able to tolerate relatively large settlements without significant overstress on the tank structure[8, 9, 12]. The pattern of tank settlement can be idealized into four types，i.e. (1) planar settlement；(2) nonuniform perimeter-centre settlement and uniform perimeter settlement；(3) non-planar settlement due to nonuniform perimeter settlement；and (4) localized settlement. The planar settlement involves only rigid body displacements(i.e. tilt)，which generally does not cause significant problems to the tank. The nonuniform perimeter-centre settlement and uniform perimeter settlement have disc shapes and are the results of nonuniform settlement of foundation soil under the vertical pressure stress “bulb” below the tank as illustrated in Fig.5. Nonuniform perimeter settlement is caused by the significant variation of foundation soil conditions along the tank perimeter. The localized settlement is due to presence of localized highly compressible soil. For a particular size of steel tank and soil conditions，the settlement is generally dominated by one of the settlement modes. Nonuniform perimeter settlement and localized soil yield can result in significant lateral displacement and radial distortion of tank shell，which can lead to malfunction or rupture of tank[6, 12]. A tank with a floating roof is generally more sensitive to nonuniform perimeter settlement than the tank with a fixed roof，which tends to settle more uniformly[8]. The lateral deflection of the tank shell and non-planar distortion are generally calculated based on settlement measurements around the perimeter of the tank after it is erected and filled with water during pressure testing.

For the fuel tank in Attawapiskat，the underlying soft clay is of generally uniform engineering properties even though the thickness of the deposit varies. For proposed tanks with fixed roofs to be constructed on granular mat foundations with a minimum factor of safety of 2.0，the second settlement pattern is considered to be predominant one. For the second settlement pattern，the settlement criteria shown in Table 3 have been developed empirically in the literature. Based on field performance experience，fuel tanks typically can tolerate a centre-to-edge differential settlement of 0.010 to 0.025 times of the tank diameter. However，the design criteria in Table 3 were considered to be applicable only to tanks under moderate climate and not conservative for the tanks in Attawapiskat due to the possible effect of the cold temperatures on the ductility of the steel. It was considered that the temperature of the diesel fuel would tend to follow but slightly lag behind air temperatures so that the steel could become as cold as −40 ℃ or lower. However，this issue was not solved. For the proposed tanks，the maximum allowable differential settlement given by manufacturer corresponds to approximately 0.003 times of the tank diameter. This stringent criterion was set primarily due to the cold climate.

Table 3　Differential settlement criteria

Researchers	Maximum tolerable $\rho_{centre}-\rho_{edge}$
J. M. Duncan et al.[12]	0.025D(Settlement profile shape A) 0.015D(Settlement profile shape B) 0.005D(Settlement profile shape C)
P. Rosenberg and N. L. Journeaux[9]	0.011D
Other researchers summarized by W. A. Marr et al.[8]	(0.010 – 0.020)D

Note：ρ_{centre} is tank centre settlement；ρ_{edge} is tank edge settlement；D is tank diameter.

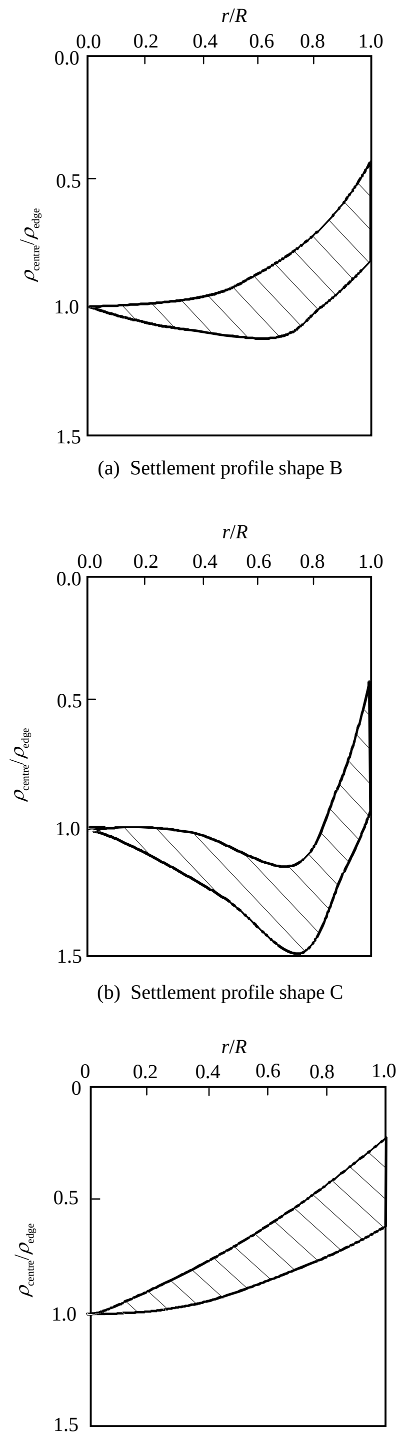
J. M. Duncan et al.[12] established a program to estimate tank settlement and to determine limits of tank tolerable settlement based on the settlement magnitude and shape observed for 31 tanks constructed on compressible foundation soils. Based on this approach，it was found that the settlement profile shape depends on the thickness of the clay layer in relation to tank diameter，and the factor of safety against foundation failure as shown in Table 4 and Fig.6.

Table 4　Factors of safety against foundation failure

Undrained F_{min}	D_e/T	Settlement profile shape
＞1.1	＜4	A
	＞4	B
＜1.1	–	C

Note：D_e is effective tank diameter；T is the thickness of foundation clay.

For the proposed tank，the calculated factor of safety is greater than 1.1；and the ratio of the effective diameter of the tank D_e to the thickness of the foundation clay T is greater than 4，which implies that the settlement profile shape B is applicable. Settlements of the 12 m- and 6 m-high tanks(comprising immediate，primarily consolidation and secondary compression settlements) with the other tanks located at least 20 m farther apart were estimated，which were considerably high. The immediate settlement was calculated based on the conventional elastic theory using the elastic moduli of the granular fill and clay soil；the consolidation settlement was calculated using one-dimensional settlement analysis in terms of recompression index C_r and compression index C_c of the clay；the secondary compression settlement was calculated using secondary compression index C_α, which was estimated assuming a value of 0.06 for the ratio C_α / C_c.



(a) Settlement profile shape B

(b) Settlement profile shape C

(c) Settlement profile shape A

Where R is tank radius；r is distance between the point of interest and the centre of tank.

Fig.6　Fuel tank base settlement profiles(modified from J. M. Duncan et al.[12])

The total settlement under the tank centre was estimated to be 730 mm and 540 mm for the 12 m- and 6 m-high tanks respectively. The corresponding differential settlement from tank centre to edge was estimated to be about 450 mm for the 12 m-high tank and 340 mm for the 6 m-high tank，which are significantly greater than the maximum allowable differential settlement of 75 mm given by the manufacturer. These predicted differential settlements were not acceptable even though the ratios of differential settlement to the tank diameter for both tanks were within the criteria suggested in Table 3.

3.3　Foundation Improvement

As shown in the previous sections，the stability and settlement analyses indicate that，with geosynthetic reinforcement alone，the tank foundation will be marginally stable；and the tank will experience considerable settlement. The majority of settlement is due to consolidation of the soft clay stratum over a prolonged period. To increase the bearing capacity of the foundation soil and reduce the tank settlement，a ground improvement technique employing installation of prefabricated vertical drains in combination with subsequent site preloading was considered.

The ground improvement has to be achieved within about 6 months to meet the construction schedule. The average degree of consolidation prior to construction of the tanks was required to be 90 percent or greater in order to meet the post-construction settlement requirements. Due to time constraints of the project，it requires relatively accurate prediction of the accelerated consolidation of the drainage-enhanced foundation soil. For vertical drain design，the following factors are required to be considered：(1) smear effect due to vertical drain installation；(2) discharge capacity of the vertical drain；(3) dissipation of the excess pore pressures in both horizontal and vertical directions；(4) time-dependent loading conditions；and (5) the significant change in consolidation coefficient when the effective stress exceeds the preconsolidation pressure. In the above factors，the consideration of time-dependent loading conditions is of importance for construction schedule.

The approach proposed by A. L. Li and R. K. Rowe[13] for consideration of the factors mentioned above was used for design of the vertical drain system. To obtain the design parameters，the consolidation coefficient of the clay deposit was determined in both vertical and horizontal directions using the oedometer test data. All other design parameters were determined from field tests in combination with literature research data. Based on the design analyses，Nilex vertical drains arranged in a triangular pattern with a space of 1.2 m and extending approximately 20 m to the bottom of the silty clay stratum were considered to be necessary. A drainage blanket about 0.5 m thick connected with lateral French drains would be required to direct the released pore water into the perimeter diversion ditches. It was estimated that about 90 percent of consolidation would take place within a four-month preloading period.

The fill height and slope for the preloading fill were determined based on the tank pressures and stability condition of the surcharge embankment. The preloading surcharge of 7.5 and 5.0 m of granular fill(i.e. 6.3 and 3.8 m above the original ground) was considered for the 12 m- and 6 m-high tanks respectively(see Fig.7). In case of the 12 m-high tank，the two-stage preloading(4.5 m of fill for the first stage and additional 3.0 m for the second stage) was recommended to achieve adequate consolidation without overstress in the foundation. The surcharge fill would have side slopes of (2－3)∶1(horizontal∶vertical). During construction，settlement and pore pressure changes would be monitored with strategically located instruments. In addition，Nilcon vane tests would be carried out to compare soil strength profiles before and after completion of the preloading operations.

The total settlements for the improved foundation stratum under the centres of the tank were estimated to be reduced to about 140 and 100 mm for the 12 m- and 6 m-high tanks respectively. The corresponding differential settlement from centre to edge was estimated to be about 70 and 55 mm for the 12 m- and 6 m-high tanks respectively. The comparison of potential tank settlement for different tanks on the native foundation clay with and without improvement

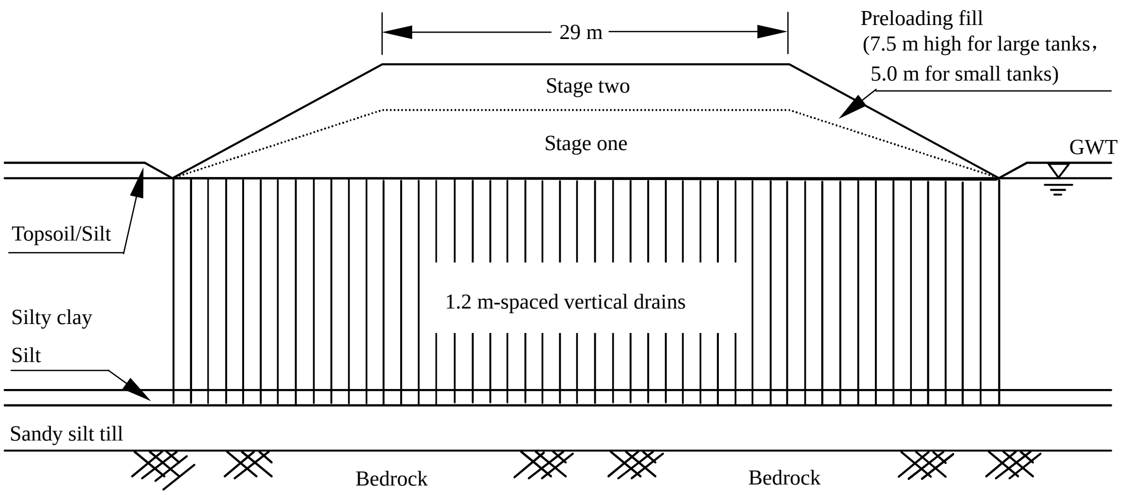


Fig.7　Schematic illustration of the preloading fill embankment on the foundation with vertical drains

was shown in Table 5. The settlement analyses suggested that，with foundation improvement，both kinds of tanks will meet the design requirements for the maximum allowable differential settlement. However，only the 6 m-high tank satisfied both the factor of safety and settlement requirements under the conservative assumption of no strength gain during the preloading period. In review of the clay index properties，it was anticipated that the clay would have some degree of strength gain after consolidation. The undrained shear strength of the foundation soil after consolidation was estimated using the SHANSEP method[14]. For the 12 m-high tank，the calculated factors of safety were 2.2 and 2.0 against overall bearing capacity and edge failures respectively. The factors of safety given above would be confirmed by conducting Nilcon vane tests to compare the soil strength profiles before and after completion of preloading.

Table 5　Comparisons of tank potential settlement for tanks on different foundations　mm

Tank option	Tank on existing clay foundation		Tank on improved clay foundation	
	Centre total settlement	Differential settlement	Centre total settlement	Differential settlement
6 m-high tank	540	340	100	55
12 m-high tank	730	450	140	75

3.4　Other Design Considerations

The results of preliminary thermal modelling for the fuel tanks at the Victor site suggested that permafrost will be developed under tanks when fuel temperatures were assumed to follow the air temperature changes. Hence，in order to decelerate the permafrost front into the frost susceptible native soils under the granular mat foundations，a 125 mm-thick continuous Styrofoam HI insulation would be installed under the tanks. The insulation would have an embedment depth of about 0.3 to 0.5 m and extend about 3.6 m horizontally beyond the tank diameter exterior below finished grade. For spill containment，a geomembrane liner embedded within the fill material would be installed over the footprint of the fuel tank farm.

4　CONCLUSIONS

The geotechnical investigation carried out in Attawapiskat，Northern Ontario，Canada，indicated that the site is underlain by soft marine clay with a relatively stiff and thin crust. The principal concern for the proposed fuel tanks for the Victor Project is the marine silty clay stratum，which has a low to medium plasticity and is slightly overconsolidated with medium sensitivity. The foundation design study has shown that (1) the native clay is too weak to support the tanks and the settlement would be excessive；and (2) the combination of the use of prefabricated vertical drains and subsequent preloading of the foundation clay stratum can effectively improve the soft foundation within the time constraint. It is concluded that the granular fill foundations on soft clay deposits using the proposed preloading technique are feasible for relatively high fuel tanks； and the anticipated differential settlement between the tank centre and edge is within the allowable maximum differential settlement.

References(参考文献)：

[1] MCDONALD B C. Glacial and interglacial stratigraphy，Hudson Bay Lowland[C]// HOOD P J，HOBSON G D，NORRIS A W，et al ed. Proceedings of the Earth Science Symposium on Hudson Bay，Geological Survey of Canada. [S.l.]：[s.n.]，1968：53－68，78－99.

[2] CRAIG B G. Late-glacial and Postglacial history of the Hudson Bay Region[C]// HOOD P J，HOBSON G D，NORRIS A W，et al ed. Proceedings of the Earth Science Symposium on Hudson Bay，Geological Survey of Canada. [S.l.]：[s.n.]，1968：63－77.

[3] MARTINI I P，PROTZ R，GRINHA M D，et al. Studies of coastal sediments，soils，and biota—James Bay，Ontario，Canada[R]. Guelph：Department of Land Resource Science，University of Guelph，1979.

[4] BJERRUM L. Embankments on soft ground[C]// Proceedings of ASCE Specialty Conference on Earth and Earth-Supported Structures (Vol.2). Lafayette：Purdue University，1972：1－54.

[5] SCHMERTMANN J H. The mechanical aging of soils[J]. Journal of the Geotechnical Engineering Division，ASCE，1991，117(9)：1 288－1 330.

[6] BERARDI R，LANCELLOTTA R. Yielding from field behavior and its influence on oil tank settlements[J]. Journal of Geotechnical and Geoenvironmental Engineering，2002，128(5)：404－415.

[7] DUNCAN J M，D′ORAZIO T B. Stability of steel oil storage tanks[J]. Journal of Geotechnical Engineering，1984，110(9)：1 219－1 238.

[8] MARR W A，RAMOS J A，LAMBE T W. Criteria for settlement of tanks[J]. Journal of the Geotechnical Engineering Division，ASCE，1982，108(8)：1 017－1 039.

[9] ROSENBERG P，JOURNEAUX N L. Settlement limitations for cylindrical steel storage tanks[J]. Canadian Geotechnical Journal，1982，18(2)：232－238.

[10] ROWE R K，SODERMAN K L. Stabilization of very soft soils using high strength geosynthetics：the role of finite element analyses[J]. Geotextiles and Geomembranes，1987，6(1/3)：53－80.

[11] MEYERHOF G G，HANNA A M. Ultimate bearing capacity of foundation on layered soils under inclined load[J]. Canadian Geotechnical Journal，1978，15(4)：565－572.

[12] DUNCAN J M，D′ORAZIO T B，MYERS P. Settlement of tanks on clay[C]// YEUNG T Y，FÉLIO G Y ed. Vertical and Horizontal Deformations of Foundations and Embankments，Proceedings of Settlement′94. [S.l.]：[s.n.]，1994：92－106.

[13] LI A L，ROWE R K. Combined effects of reinforcement and prefabricated vertical drains on embankment performance[J]. Canadian Geotechnical Journal，2001，38(6)：1 266－1 282.

[14] LADD C C，FOOTT R. New design procedure for stability of soft clays[J]. Journal of the Geotechnical Engineering Division，ASCE，1974，100(7)：763－786.