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Procedures for Designing and Assessing the Firewater Storage Tank Gravel Pad Safety

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Abstract

Foundations built on weak and compressible soils have suffered catastrophic failures due to foundation instability. This study describes the procedures used in the design of the gravel pad foundation of a steel firewater storage tank. In this regard different modes of failure were used to design and estimate the factor of safety of a gravel pad foundation. In addition, tank load estimation on gravel pad foundation, recommendations concerning minimum values of factor of safety against undrained foundation failure and common construction issues are also presented. Finally different subgrade improvement techniques for these types of tanks are presented and described.

Keywords: Factor of Safety; Gravel Pad Design; Tank Foundations; Design Procedures and Challenges; Firewater Storage Tanks.

1. Introduction

Steel firewater storage tanks are frequently installed on gravel pad foundations. Bearing capacity failures in these foundations have resulted in severe damage and rupture of tanks, loss of contents, and even loss of human life. Therefore evaluating the stability of such foundations is an important aspect of their design.

Two main modes of failure have been observed in practice, base and edge shear, for such tanks [1]. Base shear involves failure of the entire tank acting as a unit, whereas edge shear involves local failure of a part of the tank perimeter and a contiguous portion of the base.

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In this study D'Orazio (1982) [2] method was used to estimate the stability of the firewater tank. This method is mainly based on estimating the tank stability on weak clays. Real sites often have sand or other soils overlaying the weak foundation clays, layers of granular soil within the weak foundation clay, or clay strength that varies significantly with depth in the foundation. Therefore accounting for such complications in a realistic manner is perhaps the most important aspect of using D'Orazio (1982) method (also presented in [3]).

It is important to know that foundation instability may develop quickly, or slowly. It often results in large non-uniform settlements and tilting of the tank, and can lead to complete rupture of the tank and loss of contents. Also either base shear or edge shear may be the critical failure mechanism, and both should be evaluated. It should be noted that thin layers near the surface have a greater effect on the edge shear factor of safety than on the base shear factor, because the edge shear mechanism is shallower. In addition (after failure) tanks have been successfully stabilized using, re-leveling or subgrade improvement techniques which are outlined at the end of this paper.

The objective of this study is to review and describe the design procedure and to presents the challenges involved the design of the gravel pad foundations for steel firewater storage tank

2. Theoretical Background

This section presents the theoretical background and the methods used for gravel pad foundation design of a proposed firewater storage tank.

2.1. Tank Load Estimation

It should be noted that the tank loads are also usually provided by the tank vendor, which should be checked and verified. A rule of thumb in tank load estimation is:

$$q = H \text{ x } 12 \text{ kN/m}^3 \quad (1)$$

where: q= pressure due to liquid in tank and weight of tank (kPa), and H= height of the tank (m).

The above load estimation only provides a rough estimation of the talk load on the gravel pad. Different standards exist that provide a more detailed and accurate breakdown and estimation of talk loads. Following shows a breakdown of the tank loads on the pad based on American Petroleum Institute (API) 2007 and Alberta Building Code (ABC) 2006 [4][5]:

2.1.1. Dead Load

Dead Load (D_L) was considered to be the weight of the tank or tank component, including any corrosion allowance unless otherwise noted.

2.1.2. Live Load

Water content can be considered as live load but mainly in the tank foundation design is considered as dead load. The reason is that the difference between the tank volume considered in design and the fabricated tank volume is insignificant so the live load coefficient would be too conservative in design. In this study the water content was assumed to be dead load and no live loads were considered.

2.1.3. Snow Load

The ground snow load (S) was determined from ABC 2006 [5] as follow:

$$S= I_s[S_s[C_bC_wC_sC_a]+S_r] \tag{2}$$

where: I_s= importance factor for snow load, S_s= 1 in 50 year ground snow load (kPa), C_b= basic Roof Snow load (kPa), C_w= wind exposure factor, C_s= slope factor, C_a= shape factor, and S_r= 1 in 50 year associated rain load (kPa).

2.1.4. Wind Load

Wind load (p) was determined from ABC 2006 [5] as follow:

$$p= I_wq_wC_eC_gC_p \tag{3}$$

where: p= specified normal pressure acting statically in a direction normal to the surface, I_w= importance factor for wind, q_w= reference factor for wind load, C_e= exposure factor, C_g= gust effect factor, and C_p= external pressure coefficient.

2.1.5. Seismic Load

In areas where seismic loads are important, Seismic loads (E) can be determined in accordance with API 2007 section E [4].

2.2. Stability Analysis

In general, the foundations for the proposed tanks should satisfy two basic independent criteria. First, the tank loads transmitted to the foundation soils should not exceed the load carrying capacity of the foundation soils. The tank foundations should be designed to distribute the tank loads to the foundation soils, in order not to cause a bearing capacity failure. Second, total and differential tank movements resulting from settlement of the foundation soils due to the sustained loads should be within tolerable performance limits [6].

Typically for steel storage tanks, two main modes of foundation stability need to be considered: global base shear failure and local edge-shear failure. Also other modes of failure exists which will be described briefly. These mechanisms of failure are briefly described below.

2.2.1. Base Shear

Both base and edge shear modes of can be evaluated using available bearing capacity theories that take into account the thickness of the weak soil layer beneath the tank in comparison with the tank width. In base shear the entire tank acts as a single unit, with the entire base of the tank undergoing downward movement, usually accompanied by some rotation from the vertical. In this regard two approaches were taken. A typical condition is shown in Figure 1.

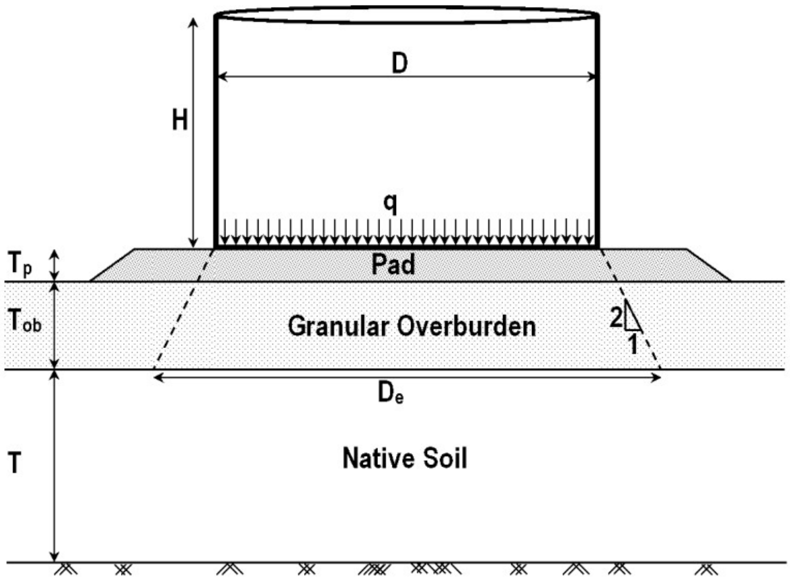

Figure 1: Typical base shear failure mechanism [3].

In order to estimate the induced vertical stress, the induced vertical stress analysis can be carried out based on 2:1 spread. The following equation can be used [2]:

$$q_e = \overbrace{q\left(\frac{D}{D_e}\right)^2}^{\text{Tank Stress}} + \overbrace{\gamma_p T_p\left(\frac{D+T_p}{D_e}\right)^2}^{\text{Gravel Pad Stress}} \tag{4}$$

where: D= tank diameter (m), D_e= $D+T_p+T_{ob}$= diameter of the loaded area of clay (m), T_p= tank pad thickness (m), q= net bearing pressure due to liquid in tank and weight of tank (kPa), q_e= net bearing pressure at top of native soil (kPa), γ_p= unit weight of pad material (kN/m^3), and γ_{ob}= unit weight of overburden material (kN/m^3).

In order to estimate the net ultimate bearing capacity for base shear in clay Duncan and D'Orazio (1984) used the following expression. The mechanism of base shear failure is very similar to the mechanism for bearing failure of a shallow footing on clay. Duncan and D'Orazio (1984) state that if the clay layer (T) is thicker than $0.7D_e$, the slip surface will probably not extend to the base of the layer. The ultimate base shear bearing capacity for clay can be expressed as:

$$q_n = S_u N_c \tag{5}$$

where: q_n= net ultimate bearing capacity for base shear of clay (kPa), S_u= c_u= CMS/2= average shear strength of clay (kPa), c_u= clay cohesitivity (kPa), CMS= ultimate compressive strength for the shear test (kPa), N_c= 6.1 when $D_e/T \leq 6$ and $4.1+D_e/(3T)$ when $D_e/T>6$= Meyerhof base shear bearing capacity factor for circular footing on deep clay layer (dimensionless), and T= thickness of clay layer (m).

By knowing the ultimate bearing capacity for base shear and the stress on top of the native soil the factor of safety (FOS) for bearing pressure can be calculated using the following equation:

$$FOS_b = \frac{q_n}{q_e} \quad (6)$$

If the term $\gamma_{ob}T_{ob}$ in both the nominator and the denominator is added, the FOS_b expression will be based on the gross load. Duncan and D'Orazio (1984) state that although including the term $\gamma_{ob}T_{ob}$ in both the nominator and the denominator have a fully logical basis, they have the effect of biasing the value of FOS_b toward unity. Therefore in the analyses they performed, the FOS_b based on net quantities were used (Eq. 6).

2.2.2. Edge Shear

Edge shear is the most common mode of bearing failure for storage tanks supported on shallow foundations [3]. In this mode, the near surface soils shear, allowing a small section of the tank to distort, and subsequently rupture. D'Orazio (1982) presented the following edge shear approach based on Figure 2.

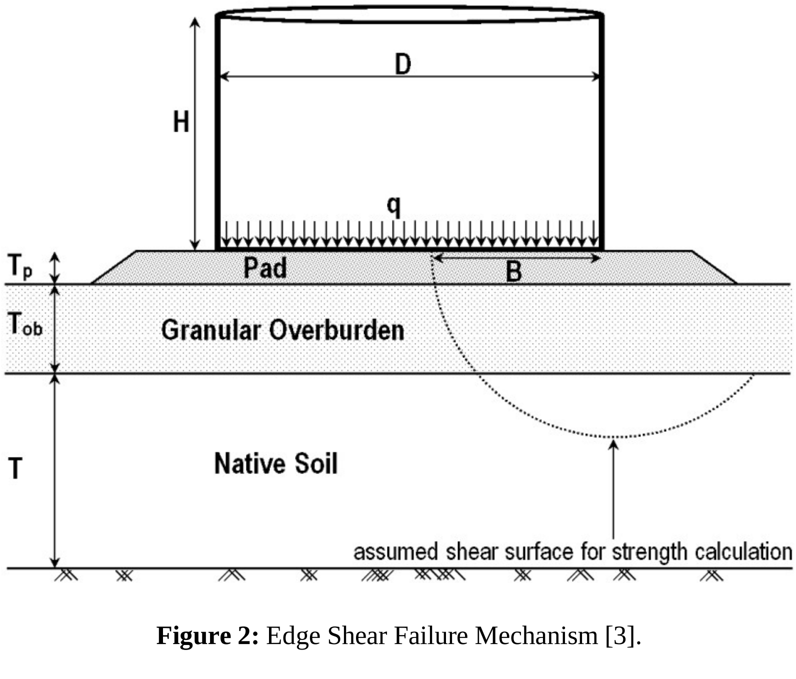

Figure 2: Edge Shear Failure Mechanism [3].

In analyzing edge shear stability the net load was calculated using the expression:

$$q_{app} = q + \gamma_p T_p \quad (7)$$

where: q_{app}= applied net load at top of clay layer.

The ultimate bearing capacity for the edge shear mode can be calculated using the following expression:

$$q_{ult} = S_u N_c \tag{8a}$$

Please note that Eq. (8a) is the corrected form of equation in Duncan and D'Orazio 1984 (*Loc. Cit.*). Where the bearing capacity factor for edge shear (N_c) is [7]:

$$N_c = 5.2 + \frac{B}{D} \tag{8b}$$

where: q_{ult} = ultimate bearing capacity for edge shear mode, S_u= average undrained shear strength, and B= width of segment involved in edge shear failure.

D'Orazio (1982) suggested that the weighting factors shown to be used to calculate the average value of S_u. Eq. 9 below evaluates the average shear strength and calculates the edge shear factor of safety.

$$FOS_e = \frac{q_{ult}}{q_{app}} \tag{9}$$

The above method is an iterative approach and by changing the B value the lowest FOS_e will be estimated. When the lowest FOS_e is achieved that is the B value.

2.2.3. Tank Settlement

Various forms of settlements could take place in a firewater storage tank. Generally, the settlement of the tank foundations may be regarded as consisting of two separate components of settlement, immediate settlement and consolidation settlement (S_T=S_i+S_c) [8].

When soil is loaded by a structure, deformations will occur. Vertical deformation at the existing ground surface resulting from the structure load is termed as settlement. In the design of engineered structures, the amount of settlement and the rate at which the structure will settle are two aspects that are of interest [9][10]. The total settlement of soil area being loaded has three components. These components are immediate settlement, consolidation settlement, and secondary settlement. The immediate settlement also referred to as distortion settlement is estimated using elastic theory. Consolidation settlement is time dependent and is a process that occurs in saturated fine grained soils with a low coefficient of permeability. The settlement rate is dependent on the rate of drainage of the pore water. Secondary compression occurs at a constant effective stress with no subsequent changes in the pore water pressure. GeoStudio software (2007) [11] was used to estimate the settlement on the tank in this study.

2.3. Factors of Safety (FOS)

The procedures for calculating FOS is described above. Duncan and D'Orazio (1984) state mention the minimum acceptable values of safety factor against base shear and edge shear thus depend in large part on the degree of certainty or uncertainty with which the undrained shear strength of the foundation can be evaluated. In cases where the strength of the foundation clay can be evaluated with minimal uncertainty, and the consequences of failure do not involve risk to life or catastrophic financial loss, factors of safety as low as 1.3 are acceptable. In cases where the foundation strength evaluations involve greater uncertainty, and where the consequences of failure are severe, larger safety factors should be used.

For tank foundation design in this study, minimum target factors of safety of 2.0 against base shear failure and 1.5 against edge shear failure were adopted (based on the recommendation of the geotechnical investigation and [12]). Although factors of safety of less than 2 are indicative of some yielding of foundation soils, and somewhat larger settlements as compared to designs where factors of safety are more in line with traditional values of 2.5 or 3. Since the strength of the foundation soils generally increase with depth at this site, edge shear effect will govern the gravel pad design.

3. Study Area

The study area is located within the City of Edmonton in Canada. The site is an active oil terminal with existing facilities at the site including storage tanks within secondary containment plots, buried facilities and pipe racks. Access to the site is available via public highways. The proposed expansion will include construction of several large storage tanks with different capacities. Figure 3 illustrates the proposed location of Firewater Tank in this study.

A geotechnical investigation was carried out for the site and several boreholes were drilled across the site and underneath the tanks.

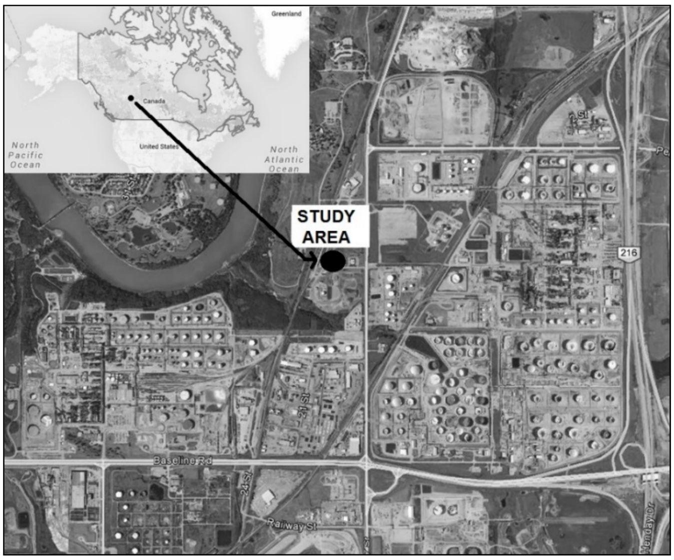

Figure 3: Study Area.

The review of the boreholes at site indicated that there may be weak zones in the upper 2 to 3 meters where improvements of the subgrade support would be required. With conventional construction this would entail excavation of these weaker soils and their replacement with an engineered fill. These weaker soils would include fill soils that typically had been placed in an uncontrolled manner and where the Standard Penetration Test 'N' values of less than 10 were noted. Figure 4 shows the soil profile at site. The suggested undrained shear strength (S_u) of clay in the upper 5 m was between 60-120 kPa, which is consistent with the category of stiff to very stiff clay.

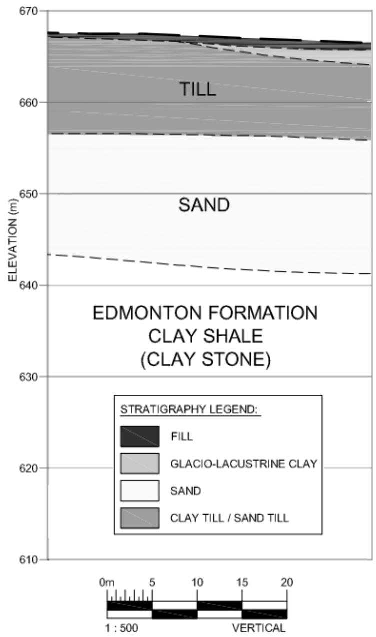

Figure 4: Soil Profile at Site.

Also based on the weighted average undrained shear strength and the weighted average N-value of the subsurface soils, as well as the depth of the bedrock, the site was categorized as Class 'D' in accordance with the National Building Code of Canada (NBCC) 2010 [13].

The firewater tank in this study was supplied by a third party vendor. The tank was 12.2m high and 13.0m wide with 1:120 base slope with a ¼" (6mm) annular ring all around. The water content was considered to be to the top of the tank (see Figure 5).

4. Results

Based on conditions encountered throughout the proposed site, and based on the recommendation receiving from the tank vendor, the proposed tanks can be supported on a shallow foundation consisting of crushed stone pads. For this study the tank load on the foundation was estimated using the following three methods:

- Load provided by the tank vendor;
- The rule of thumb estimation (Eq. 1); and
- API 2007 and Alberta Building Code (ABC) 2006 [4][5].

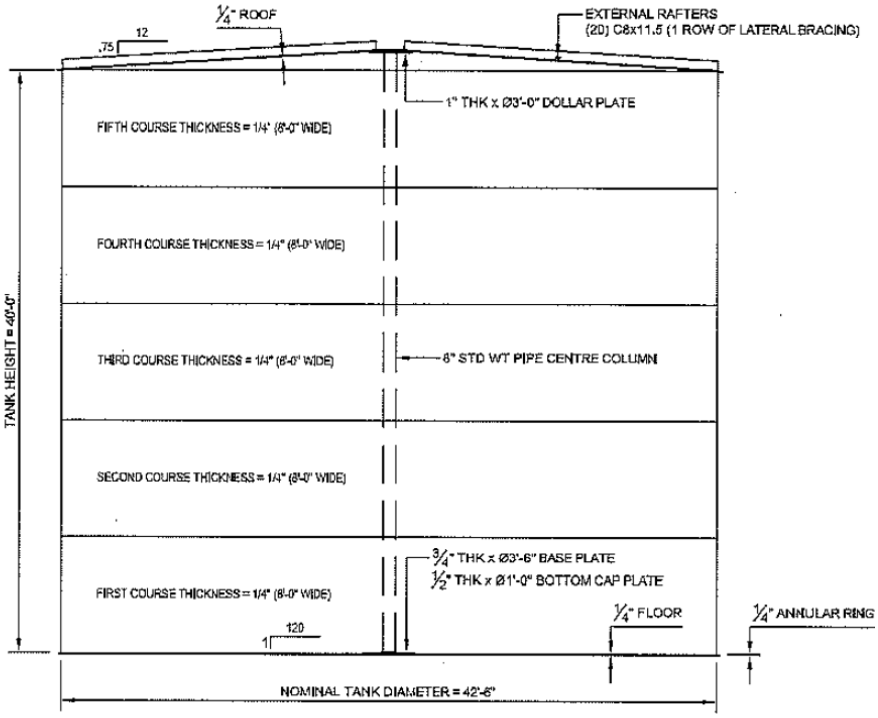

Figure 5: Tank Dimension and Size Provided by the Vendor.

For the design of the tank foundation, the most stringent of all 3 were uses. The ground was considered to be sub-excavated 2.5m to remove the unsuitable and weak soil, prepare and inspect the subgrade and fill with 40mm minus crushed gravel compacted to 100% standard proctor maximum dry density (SPMDD). Then overlay with 0.8m of 20mm minus crushed gravel compacted to 100% SPMDD for the tank foundation. The principal factors governing stability and the calculated factors of safety for base shear and edge shear were 2.80 and 1.59 respectively.

It is important to note that the most critical condition for the soil usually occurs immediately after construction, which represents undrained conditions, when the undrained shear strength is basically equal to the cohesion (c). Therefore, it is expected for the factor of safety to increase with time. Figure 6 shows the design details of the gravel pad in this study.

GeoStudio software (2007) [12] was used to estimate the settlement on the tank in this study and the total long term settlement of the foundation was estimated to be less than 300mm.

In this design, the traditional ground improvement (removal of the unsuitable soil beneath the foundation) was carried out, since other ground improvement techniques were found to be uneconomical. A summary of other ground improvement techniques that can be conducted for the site is given in Table 1.

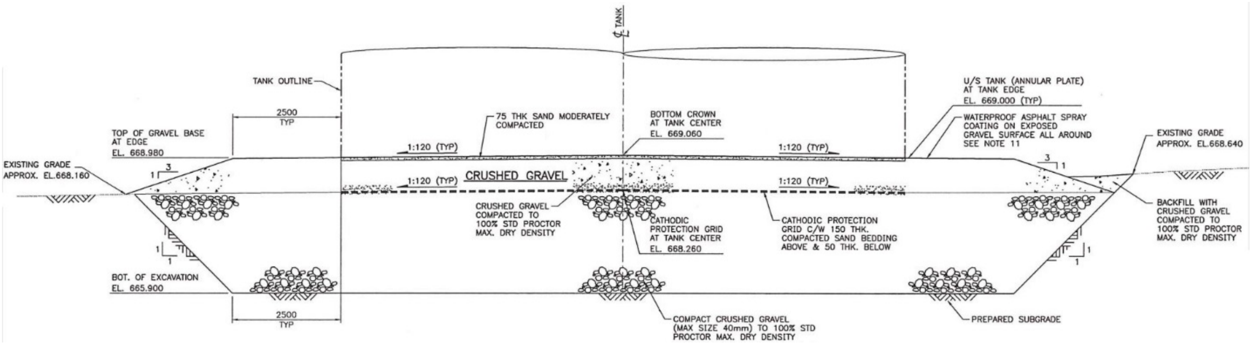

Figure 6: Firewater Tank Gravel Pad Foundation Design.

Table 1: Available Soft Ground Improvement Techniques and Applicability to the Site in this Study.

TYPE	TECHNIQUE	SOIL TYPE	APPROPRIATE FOR SITE
COMPACTION (COMPACT OR DENSIFY THE SOIL)	Dynamic Compaction	Granular Soil	×
	Vibro Compaction	Granular Soil	×
	Compaction Grouting	Granular Soil	×
	Surcharge with Prefabricated Vertical Drains	Fine Grain Soils	✓
	Blast-Densification and Vacuum-Induced Consolidation	N/A	×
REINFORCEMENT (REINFORCING THE SOIL)	Stone Columns	Sand Silt Clay Gravel Etc.	✓
	Vibro Concrete Columns	Organic	✓
	Soil Nailing	Slopes, Cohesive	×
	Micropiles	Any Type	✓
	Fracture Grouting	Any Type	✓
	Fibres and Biotechnical	N/A	×
FIXATION (BIND THE SOIL)	Permeation Grouting	Sand Granular	×
	Jet Grouting	Wider Range of Soils	✓
	Soil Mixing (Dry & Wet)	Soft Soils	✓
	Freezing and Vitrification	N/A	×
OTHER INNOVATIVE SOFT GROUND IMPROVEMENT TECHNIQUES	Rammed Aggregate Piers	N/A	×
	Reinforced Soil Foundations	N/A	×

Following summarizes some of the challenges involved in gravel pad foundation design and construction:

- Not possible to compact the gravel 100% SPMDD, especially in winter.
- Maintaining the foundation side slopes (when no asphalt pavement or any other type of protection is used).
- Sloping the foundation base to match the tanks base slope.
- Proof rolling of subgrade, to achieve the desired subgrade loading, not possible due to access to the base of the foundation. In this case a picket pen may be used.
- Winter construction and material handling.

5. Summary and Conclusions

The design procedure of a gravel pad foundation for a firewater tank was presented in the current study. The study is mainly limited to the Northern Alberta Oil & Gas plants in Canada where the standards presented herein are applicable. In this study for constructing the gravel pad foundation the traditional method of complete removal of unsuitable soil was used. Other techniques of the subgrade improvement were presented but were found to be uneconomical due to the size of the gravel pad. Finally, the common challenges for gravel pad foundation construction were presented and shown. In order to further investigate the performance and to the ensure safety of the pad it is recommended to conduct regular maintenance and install instruments to monitor settlement.

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