Challenges and potential risks associated with the construction of a rockfill dyke on a submerged tailings beach

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ABSTRACT A long dyke will be constructed in a lake with rockfill placed by end dumping method. Its crest will be at elevation 521.5 m compared to the nominal lake level at 520 m. In the first portion, its height will vary from 1.5 m on a 35.1 m thick tailings layer to 18.3 m on about 0.5 m of tailings. In the second portion, the dyke will be built on native silty clay and will vary from 18.3 to 15.2 m in height. The tailings consist in stratified layers of fine-grained soils showing lateral and vertical segregation. The tailings are compressible and may develop excess pore water pressure on loading. The tailings overlie a thin layer of relatively soft native silty clay. The till beneath the silty clay is considered incompressible compared to the material above. After construction, as the tailings and silty clay consolidate, pore water pressures will dissipate and dyke stability will improve. Since dyke stability will increase with time, the primary concern is its stability during construction. As a result of construction activities, three potential modes of failure were identified and recommendations were provided regarding the measures that should be undertaken to increase dyke stability during construction.

Introduction

Construction of a rockfill dyke on a thick layer of tailings deposited as a slurry in water up to 35 m deep presents some challenges and potential risks. Construction on tailings typically involves risks, theses risks are obviously much greater when construction takes place on submerged tailings. Even if tailings appear to be a uniform material, they often show signs of segregation when they are deposited as a slurry. Segregation will be even more prevalent when a single point of discharge is used. Coarse particles will tend to settle rapidly close to the discharge point, and finer particles may travel variable distances to progressively settle away from the discharge point in a more tranquil environment. Some very fine particle fractions may not even settle naturally in water without extremely tranquil conditions and without the use of flocculants or other chemical agents.

This paper presents potential risks and challenges associated with construction of a dyke by end dumping rockfill in water on tailings beach where the water progressively increases to depths of more than 15.0 m. The proposed methodology to reduce risks is also presented.

The proposed dyke to be built in the next few years, will be part of a tailings management project that will enclose a tailings disposal area within a lake. The dyke construction will provide a confinement for the tailings and thereby prevent the movement of tailings into the lake. The first segment of the dyke of about 1 km will be constructed away from the tailings beach and will be extended by the construction of a series of dykes in shallower waters to obtain complete confinement. The final series of dykes will be more than 15 km long. This paper addresses the challenges and issues strictly related to the construction of the first 1 km long segment. This segment presents some challenges because it will be built directly on both existing tailings and on natural sediments and in continuously increasing water depths. Given the area where this dyke will be built, material availability, and the economic environment of the project, end dumping rockfill in water at the end of the advancing front has been the chosen construction method.

The construction of the different dykes should be completed by the year 2010. The project name and client are intentionally not mentioned. The interest of this paper is only to describe the methodology that should be followed to mitigate the potential risks of failure related to the construction of a rockfill dyke by end dumping method on a submerged tailings beach.

Site description

The construction of the first segment of the series of dykes will progress from west to east with rockfill being dumped and pushed to advance along the length of the dyke (Fig. 1). Although some degree of compaction of the rockfill will be obtained by construction traffic, most of the rockfill will be compacted under its own weight in submerged conditions. It is therefore anticipated that the structure will be somewhat loose and will tend to settle as particles progressively get rearranged with time in a more compact form. The rate of this rearrangement will be at its maximum during construction and will extend well beyond the end of construction.

End-dumping will be done from the dyke crest above water using 50 tons haul trucks. The crest of the dyke will be at a nominal elevation of 521.5 m. This compares with the nominal average lake water level that is approximately at elevation 520.0 m. The lake being fairly large, the hydrological analysis showed that a 1.5 m freeboard is sufficient to protect from the wave action and the naturally varying water levels over the whole year under extreme events.

From the beginning of the dyke until approximately the midpoint, there is an almost linear transition from a 1.5 m high dyke on a 35.1 m thick layer of tailings to an 18.3 m high dyke on less than 0.5 m of tailings or natural sediments. Geotechnical site investigations have shown that tailings particle size varies progressively from coarser to finer both laterally and vertically. These investigations have also shown that from the midpoint to the end of the proposed dyke, the foundations involve fairly thin layers of native silty clay. The dyke over this segment will vary from 18.3 m to 15.2 m in height (Fig. 2).

Fig. 1. Proposed dyke alignment

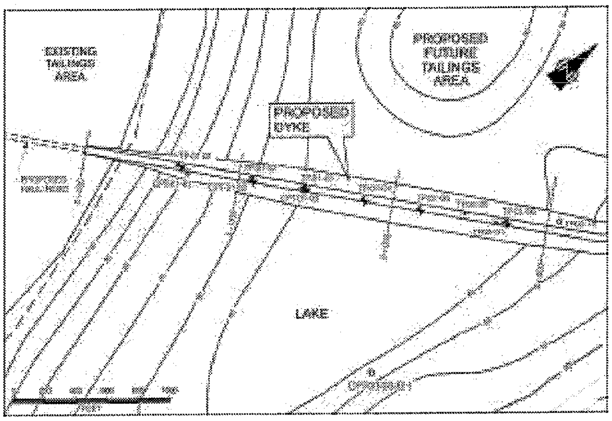
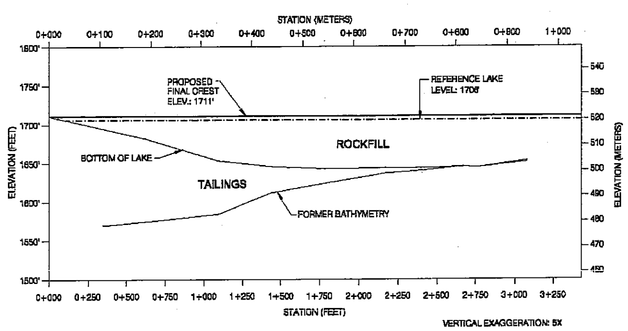


Fig. 2. Longitudinal profile of the proposed dyke



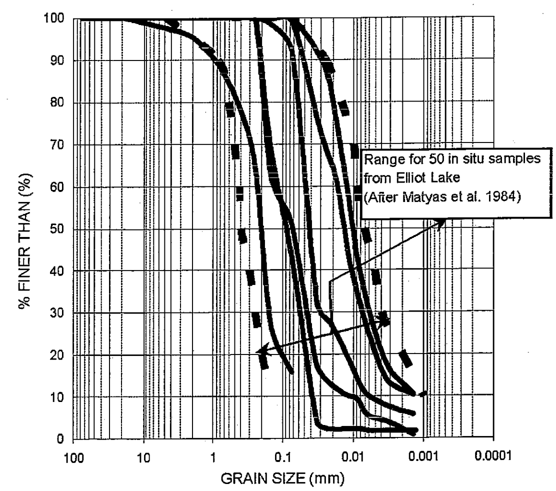
As mentioned, the tailings underlying the proposed dyke alignment have been placed hydraulically in standing water and consist of stratified layers of fine-grained soils varying from silty clay to silty sand (Fig. 3). They are considered to be normally consolidated under existing loads, compressible, and subject to the development of excess pore water pressure upon loading. The tailings overlie a relatively thin layer of soft to stiff native silty clay. A basal glacial till is also found beneath the native silty clay, and can be considered more or less competent and incompressible when compared to the tailings and native clay above it.

Pore water pressures that may be induced by the construction of the dyke will result in lower effective stresses within the tailings. This will reduce their ability to withstand the shear stresses imposed by the load induced by the construction activities; thus the stability of the dyke will be reduced. The shear strength of the native silty clay is also relatively low and may adversely affect the dyke's stability.

Theoretically, after construction, as the tailings and silty clay consolidate under the load of the dyke, pore water pressures will dissipate and there will be a corresponding increase in effective stresses which, in turn, will increase the available shear resistance of the tailings and silty clay, and thus improve the stability of the dyke.

Given that the stability of the dyke will increase with time, the primary concern is its stability during construction. The analysis conducted in this study focused primarily on the stability of the dyke during construction. A description of the methods of analyses and summaries of the results of the analysis are contained in the following sections.

Fig. 3. Grain size distribution



Potential failure modes

A number of key potential failure modes were identified during the design study based on subsurface conditions and expected dyke configuration. The following potential modes of failure are discussed in this paper:

- Shallow slumping of the front and side slopes of the dyke as the rockfill assumes its natural angle of repose in response to its own weight and the weight of construction equipment.
- Deep seated failures through the dyke and into the underlying tailings or native silty clay.
- Failure induced by static liquefaction of the tailings in response to the load of the dyke.

Shallow slumping of the front slopes of the dyke during construction

The front and side slopes of the dyke will be created by dumping and spreading rockfill from the shore and then from the top of the dyke itself. The advancing face of the rockfill will remain at its angle of repose. For the purpose of this analysis, the angle of repose of the rockfill has been estimated to vary from 38° to 40°, resulting in slope inclinations of approximately 1.5H: 1V. The factor of safety of these slopes will be marginal, nominally 1.0 on placement. The presence of construction equipment or piles of rockfill on the top of the dyke would place additional loads on the slopes thereby reducing their stability and potentially resulting in the shallow slumping of the rockfill on the slopes of the dyke. Shallow slumping of the slopes may reach the top of the dyke but is not expected to significantly encroach onto the crest or affect the overall stability of the proposed dyke.

The most critical potential failure mechanism during construction is expected to be deep seated failures through the dyke and into the underlying tailings or native soil. The potential for this type of failure along the proposed dyke alignment was analysed as described in the following sections.

Analytical approach

Stability analysis was conducted for six representative cases located along the proposed dyke alignment. For each case, a cross-section was produced through the center of the dyke, parallel to the alignment. The cases are summarised in Table 1.

The locations of the cases were selected based on:

- Being representative of the subsurface conditions along the proposed dyke alignment; and
- Proximity to standard boreholes and/or piezo-cone penetration tests conducted during the geotechnical investigations.

Table 1. Stability analysis cases

Case	Station (m)	Approximate height of dyke	Approximate thickness of tailings
1	0+315	16.5 m	21.9 m
2	0+480	21.9 m	10.4 m
3	0+625	20.1 m	6.4 m
4	0+180	7.9 m	36.6 m
5	0+425	21.9 m	11.0 m
6	0+840	21.9 m	1.0 m

The subsurface stratigraphy for each case was interpreted from the results of the geotechnical investigations.

The following assumptions were made for analytical purposes:

- The rockfill will be a cohesionless and free-draining material not subject to excess pore water pressure;
- The tailings are essentially frictional in nature, based on their grain size distribution, their low plasticity and the observed piezo-cone penetration test response as encountered during the geotechnical investigations;
- Some deeper zones within the tailings will be subject to the development of excess pore water pressures on loading based on grain size distribution and piezo-cone penetration tests;
- The tailings and native silty clay underlying the proposed dyke alignment are normally consolidated and contain no excess pore water pressure under pre-loading conditions;
- The rate of pore water pressure dissipation within the tailings is expected to be fairly high;
- Based on its plasticity and consistency, the strength of the native silty clay can be approximated by its undrained shear strength, as determined by vane shear testing or estimated from the existing effective vertical stress;
- Given that the glacial till that underlies tailings or native silty clay at all borehole locations is competent, critical potential failure surfaces will not pass through the glacial till;
- The dyke will be constructed relatively rapidly which would not allow for complete consolidation or complete dissipation of excess pore water pressure;
- Excess pore water pressure developed in the tailings will be a function of the distribution of stress below the dyke, thus limited excess pore water pressure will develop in the tailings beyond the toe of the dyke;
- The subsurface stratigraphy can be linearly interpolated between the boreholes;
- There will be no excess pore water pressure developed in the upper 1.5 m of the tailings due to: 1) the rockfill providing a freely draining boundary on the top of the tailings; 2) the tailings being loaded progressively as the front slope of the dyke advances; 3) low pore water pressure response observed by the piezo-cone penetration tests in the upper portion of the tailings; and 4) displacement of the upper few feet of tailings by the placement of rockfill.

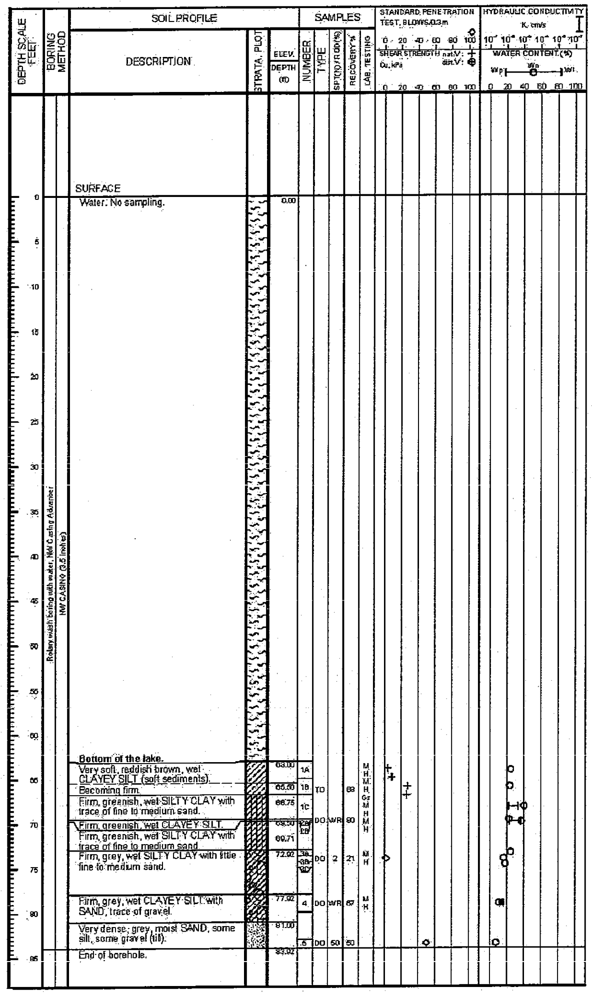
Geotechnical investigations

The properties of the rockfill, tailings, and native silty clay used in the analysis were based on the results of the geotechnical investigations and laboratory testing. The geotechnical investigations included standard geotechnical boreholes, in-situ shear strength measurement profiles and piezo-cone penetration tests (CPT). For the standard geotechnical boreholes, samples were typically collected in a continuous manner in the upper 3.0 meters of soils or tailings, at 1.5 meter intervals beyond the first 3.0 m of soil or tailings and 3.0 meter intervals beyond 12.0 m of soil and tailings. Typical data of the standard geotechnical boreholes are plotted in Fig. 4 and 5 for the tailings and the native soils respectively. The CPT is a very good tool to evaluate the variation of the soils. It measures the tip resistance to penetration, side or sleeve friction of the soil on the penetrometer, and pore water pressure response as the penetrometer is pushed into the soil at a constant rate of 2 cm/s. Typical results from CPT are plotted in Fig. 6. Typically, it was found that the upper 1.5 m of tailings did not develop excess pore water pressure during testing. However, the tailings below this level developed highly erratic excess pore water pressures, ranging from none to moderate indicative of the strongly segregated nature of the tailings vertically. The higher excess pressure measured below approximately elevation 489.2 m in the last 1.5 m or so penetrated by the CPT, is interpreted to be related to the presence of native silty clay.

Rockfill

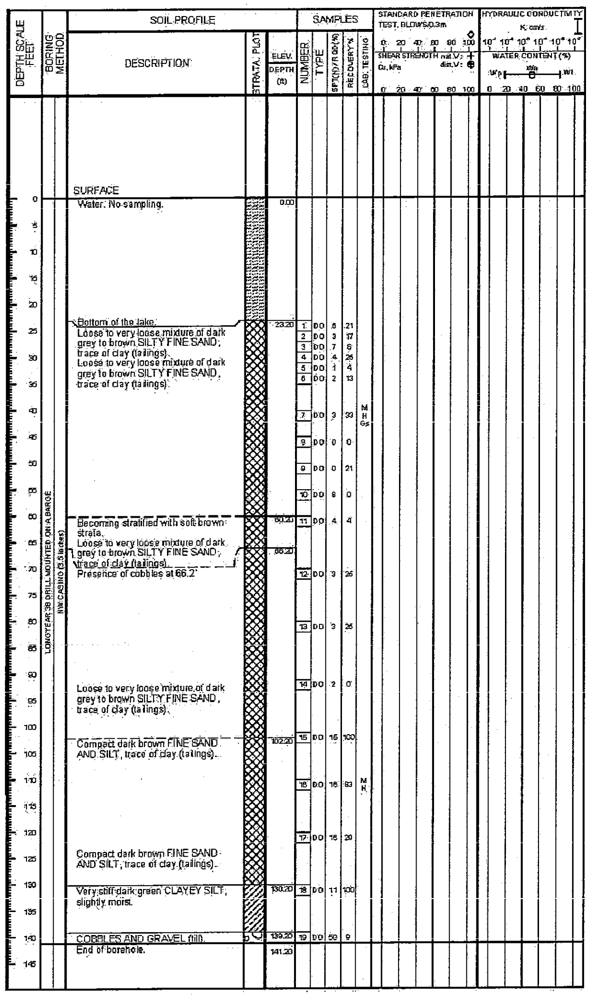
Based on available information and engineering experience on similar materials, it was estimated that the rockfill to be used for the dyke construction will have a bulk unit weight of 19 kN/m^3 and an internal friction angle of 38° at high stress levels. For shallow failure zones with low stress levels, an angle of 40° was considered.

Fig. 4. Typical borehole record for tailings



to 1.0 for excess pore water pressure equal to the magnitude of the applied vertical load caused by the dyke.

Fig. 5. Typical borehole record for native soils



Tailings

Based on the results of the geotechnical investigations and for analytical purposes, the tailings have been divided into two separate layers: a 1.5 m thick upper layer and a lower layer of variable thickness. Based on its grain size distribution and the results of the geotechnical investigations, the upper layer of tailings was assigned a bulk unit weight of 19 kN/m^3, an internal friction angle of 33°, and no cohesion. The lower layer of fine grained tailings was assigned a bulk unit weight of 19 kN/m^3 and an internal friction angle of 30° and no cohesion. Additionally, based on the piezo-cone penetration test results, the lower layer of tailings was found to be susceptible to the development of excess pore water pressure on loading.

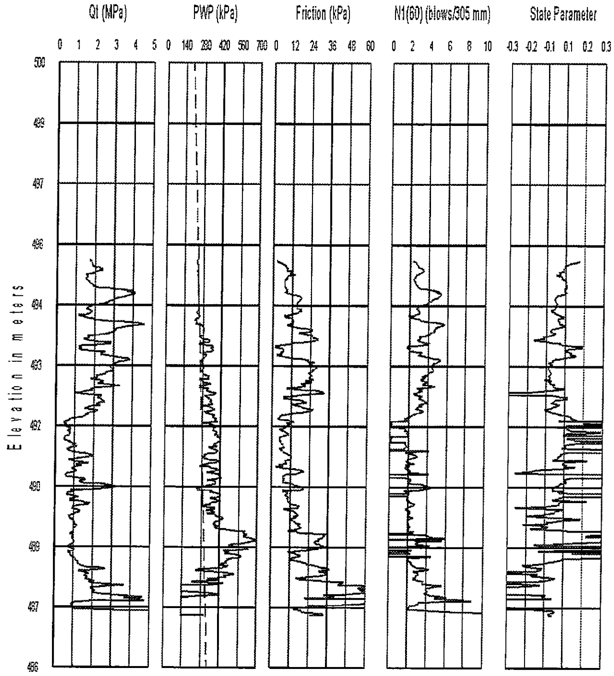
Excess pore water pressure Δu that could develop in the tailings as a result of dyke construction was represented by the pore pressure parameter $B=\Delta u/\Delta\sigma_v$ where $\Delta\sigma_v$ is the load increment associated with dyke construction. In the stability analysis, Δu was represented by the traditional r_u approach where $r_u = u/\gamma h$ where u = total pore water pressure $u_o + \Delta u$ and γh is the total stress at a given point. The analysis involved varying the pore pressure parameter B from 0.0 for no excess pore water pressure development

The results of the CPT's, particularly the PWP profiles, show that the tailings are layered with coarser and finer grained material. The finer portions are reflected by low to moderate excess pore water pressures, while the coarser layers are reflected by no to low excess pore water pressures. The maximum thickness of finer tailings is of the order of 1.5 m. Given this layered structure, and considering that even fine-grained tailings consolidate quickly, it is likely that the induced excess pore water pressure caused by dyke construction will be moderately low.

Using the data provided by Matyas et al. (1984), it was estimated that the B value would likely not exceed 0.3 in the tailings. For example, examination of measured excess pore water pressure induced by a 5 m high test embankment constructed on a 11 m thick tailings deposit at Elliot Lake showed B values of typically approximately 0.25 in fine-grained silt tailings and very little excess pore water pressure was measured in sandy (sand, some silt) tailings, with maximum B values of about 0.03. The silty tailings of Elliot Lake are considered to be reasonably representative of fine-grained tailings (Fig. 1).

For analytical purposes, the upper 1.5 m of tailings were assumed to be free of excess pore water pressures and the remainder of the tailings were assumed to have a uniform development of excess pore water pressure characterised by a B value varying from 0.0 to 1.0.

Fig. 6. Typical piezo- cone penetration test results



Native silty clay

The native silty clay underlying the tailings was determined by field and laboratory testing to have a bulk unit weight of approximately 15 kN/m^3 and an average undrained shear strength of approximately 24 kPa. This undrained shear strength correlates well with the expected strength of the clay given its stress history and existing state of effective stress. The undrained shear strength of the silty clay was measured by means of in situ vane shear tests and laboratory vane shear tests on Shelby tube samples.

Summary of properties used in analysis

Table 2 summarises the properties used in the dyke stability analysis. However, in the course of the analysis certain properties such as the undrained shear strength, the thickness of the silty clay layer, and the pore water pressure ratio within the lower portion of the tailings were varied to provide an evaluation of the sensitivity of the dyke stability to the variation of these parameters.

Analysis and results

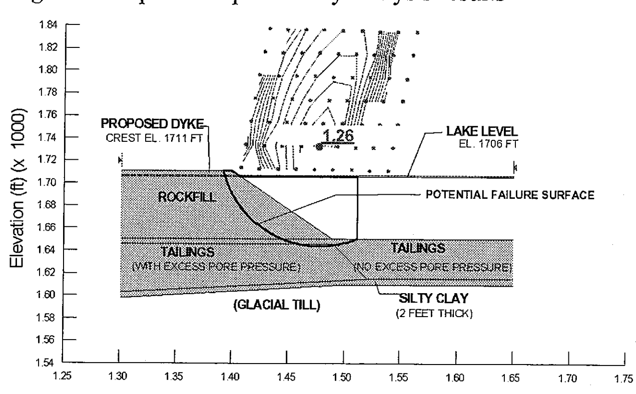
The overall stability of the dyke was analysed at the six representative sections using version 4.0 of SLOPE/W, a commercially available software program from Geo-Slope International Ltd. The Morgenstern-Price method was selected to model the internal forces of the potential areas of failure because it is widely recognised as both representative and accurate. Analysis was conducted for each case while varying key variables or parameters to determine their relative effect on the dyke stability. Both circular and wedge failure mechanisms were analyzed. Even if it is commonly recognized that the stability of a rockfill advancing face over the surface of a submerged tailings beach is controlled by the double wedge failure mechanism, the analyses have shown that the circular failure mechanism was the most critical in this case. An example of a cross-section used in the analysis is presented on Fig. 7 and the analysis results of the six cases are shown in Fig. 8.

Table 2. Material properties used in analysis

Material	Properties	
Rockfill	γ'	19 kN/m^3
	ϕ'	38 $^{\circ}$
Upper Layer of Tailings	γ	19 kN/m^3
	ϕ'	33 $^{\circ}$
Lower Layer of Tailings	γ	19 kN/m^3
	ϕ'	30 $^{\circ}$
	$\Delta u/\sigma_v$	Varies
Native Silty Clay	γ	15 kN/m^3
	Cu	24 kPa

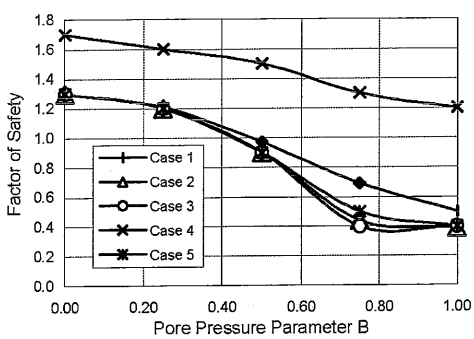
Cases 1 through 5 were analysed assuming that the pore pressure parameter, B, varied from 0.0 to 1.0. As shown on Fig. 8, the stability of the dyke is highly dependent on the magnitude of excess pore water pressure developed in the tailings. For an estimated pore pressure parameter B of 0.3, the factor of safety of the dyke during construction varies between 1.2 and 1.6. With respect to Case 6, no excess pore water pressures were assumed in the tailings due to the thickness of the tailings layers at this location. The factor of safety for this case is approximately 1.3.

Fig. 7. Example of slope stability analysis results



At the location of Case 6, where the dyke will be constructed directly over native silty clay, the thickness and undrained shear strength, Cu, of the silty clay were varied to assess the impact of the strength of the clay on the factor of safety. The effect of varying the strength of the silty clay is shown on Fig. 9. The factor of safety ranges from 1.2 with a shear strength Cu of 12 kPa to 1.4 with a shear strength Cu of 25 kPa. Silty clay strengths greater than 25 kPa do not result in an increase in the factor of safety because a potential failure through the silty clay is no longer the critical mode of failure above this strength. Fig. 10 also presents the effects of varying the thickness of the native clay layer. The stability of the dyke decreases with increasing thickness of the silty clay layer to a point where the dyke stability is marginal when the layer is approximately 3.7 m thick. However, it is noted that the silty clay layer encountered in the vicinity of Case 6 was only 0.6 m thick. The likelihood of clay thickness approaching 3.7 m is considered to be very low given the findings of the geotechnical investigations.

Fig. 8. Effect of variation in pore pressure parameter B



At the location of Case 3 the proposed dyke will be approximately 20 m in height and located on roughly 6 m of tailings over a 0.5 m thick layer of silty clay. Under excess pore water pressures based on an assumed pore pressure parameter B of 0.3, the factor of safety of the dyke is approximately 1.2. Assuming that the silty clay layer at this location is 1.5 m thick and that its undrained shear strength Cu varies between 12 and 26 kPa results in factors of safety varying between 1.0 and 1.1.

Slight variations in the factors of safety with respect to the wide variation in the assumed undrained shear strength of the silty clay at the locations of Cases 3 and 6 indicate that the shear strength of the silty clay is not a significant factor in the stability of the dyke. However, as described above for Case 6, the thickness of the silty clay layer is a significant factor in the stability of the dyke.

Static liquefaction of tailings

The results of the field geotechnical investigations show that the tailings are variable in thickness, composition and density/consistency. The granular (sandy/silty) tailings are generally very loose to loose, while the finer grained (clayey) tailings are generally soft. Given these conditions, the potential for static liquefaction needs to be addressed, particularly for the more sandy material.

Static liquefaction is defined as a condition where monotonic stress increments in a soil results in the development of excess pore water pressure leading to a substantial reduction in effective stress and associated substantial loss in strength. This, in turn, can lead to flow, particularly in loose saturated sand slopes. There are no dynamic or cyclic effects, such as those induced by earthquakes, associated with static liquefaction.

The potential for static liquefaction is controlled by the stress-strain behaviour of soils. Static liquefaction can only happen in soils that show contractive behaviour during shear. The soil volume must tend to decrease or contract when loaded or subjected to a stress increment. As the soil mass tends to contract, excess pore water pressure in the voids can develop if the pore water pressures do not get a chance to dissipate prior to the application of the next stress increment. If the rate of loading is rapid relative to the rate of consolidation, excess pore water pressures can steadily build up leading to a loss of shear strength. The stress-strain behaviour of contractive soils is characterised by a post-peak reduction in strength or a strain-softening behaviour. After peak strength is reached, with an additional strain or deformation, the strength reduces until it reaches the residual strength value; no further decrease in strength despite increasing strain. Flow slides induced by static liquefaction continue to develop and spread until the applied shear stress becomes equal to or less than the available or operational shear strength.

Fig. 9. Effect of variation in strength of native soils

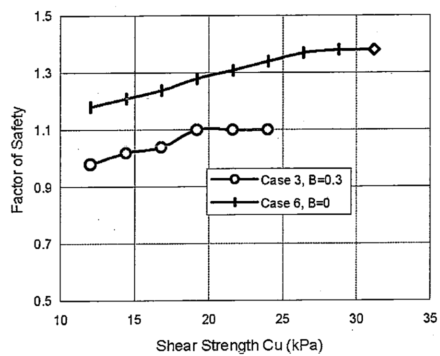
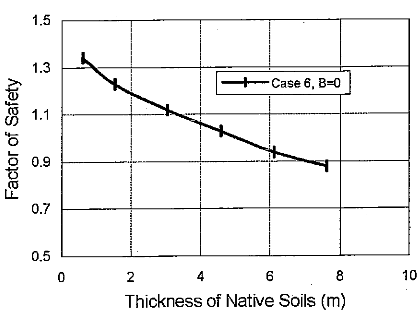


Fig. 10. Effect of variation in the thickness of native soils



The assessment of whether a soil, including tailings, shows contractive behaviour can be carried out by laboratory testing, preferably on undisturbed samples, or through correlations with in-situ tests such as the piezo-cone penetration tests. For loose saturated sandy material, the use of the piezo-cone penetration tests is widely accepted in the current state-of-the-practice as a suitable, if not preferred, method for evaluating static liquefaction potential (Robertson & Campanella, 1985).

The state parameter interpreted from the cone penetration test results is used to assess if the tailings materials are likely to be susceptible to static liquefaction. This assessment is made on the basis of evaluating if the CPT response is indicative of contractive or dilative behaviour. The distinction between contractive vs. dilatant behaviour is defined by a state parameter. A positive value of state corresponds to contractive behaviour, while dilatant behaviour corresponds to a negative value (Robertson, 1990, Plewes et al., 1992, Been & Jefferies, 1992 Robertson et al., 2000).

A dilatant soil is characterised by a stress-strain response that shows no drop in strength with increasing strain, strength steadily increases until its critical state is reached. The excess pore water pressure induced during shear is negative. For such soils, static liquefaction is not possible.

The piezo-cone penetration test records provide the interpretation of state parameter. The values of the variables required for the interpretation are summarised on the piezo-cone penetration test sheets and were based on the database published in the technical literature. It is noted that the tailings investigated are very fine-grained, with a substantial proportion of the tailings passing the No. 200 sieve. The high fines (silt and clay) content was taken into account in the selection of the interpretation of state parameter.

The interpreted state parameter is shown on the piezo-cone penetration records. It is seen that with the exception of a few thin zones, the state parameter is typically smaller than –0.05. This indicates that the tailings would likely exhibit dilatant behaviour. A zone of positive state exists within the upper 1.5 m of tailings at some locations. This is considered to be of no significance because this material will likely be displaced by rockfill during dyke construction.

At some locations, positive state parameters were found for zones in which excess pore water pressure developed during the piezo-cone advance. These zones are likely to be coherent clayey materials that are generally less susceptible to static liquefaction. Further, the interpretation of state parameter from the piezo-cone penetration tests in such fine-grained material is uncertain and may not be appropriate.

It is generally accepted that fine-grained clayey/silty material is much more resistant to liquefaction than fine sands with a low fines content and at low confining pressure. Given this and the above discussion, it is considered that the potential or likelihood for static liquefaction of the tailings as a result of rockfill placement is low. Supporting evidence for the low probability of occurrence for static liquefaction is provided by the results of the Canadian Liquefaction Experiment (CANLEX) project, in which a condition of static liquefaction could not be induced by the rapid construction of a test embankment on a tailings material considered to be susceptible to static liquefaction (Robertson et al., 2000). In the field trial conducted by the CANLEX Project at Syncrude's J-Pit, 8 m of tailings were rapidly placed over approximately 12 m of loose beach below water tailings in the deliberate attempt to cause a static liquefaction failure. The rapid loading under this experiment failed to trigger liquefaction in the underlying loose sand unit (List, 1995). Loading rates in excess of those tested in the CANLEX Project are not feasible from an operational perspective (List et al., 1996).

Mitigation measures

To attain an acceptable level of safety during construction, six remedial measures or alternatives were identified:

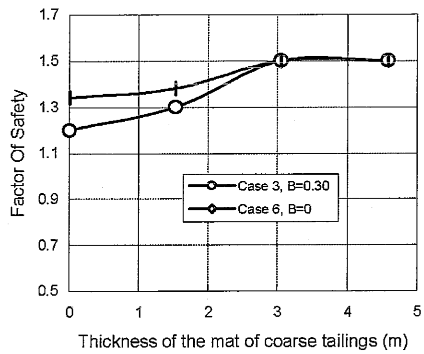
- Control of the rate of advancement of construction to allow for partial consolidation of the existing tailings and silty clay under the front slope of the dyke resulting in a partial dissipation of the excess pore water pressure;
- Relocation of the alignment of the proposed dyke in shallow water where the subsurface conditions are more favourable;
- Preloading the existing tailings and silty clay layers by placing a mat of coarse tailings along the alignment and allowing it to consolidate before construction of the dyke. This could be accomplished by re-directing normal tailings disposal to the east side of the tailings;
- Dredging the tailings and silty clay from the lakebed along the proposed dyke alignment. This option is considered difficult to execute at this location;
- Changing the construction method: instead of placing the rockfill by conventional end-dumping, use an open-bottom barge to place the rockfill so that flatter slopes could be achieved, and
- An instrumented trial dyke was proposed to assess on the actual build-up of the excess pore water pressure during construction.

The analysis of these six options reveals that the most feasible solution in terms of practicality and cost effectiveness is the preloading of existing materials with a mat of coarse tailings; allow the existing tailings to consolidate under the load of the freshly placed tailings, and then constructing the dyke on top of these tailings. This alternative would strengthen the tailings through consolidation and thus increase the dyke stability during construction. It may be possible to place a tailings blanket simply by redirecting the normal tailings slurry disposal to the proposed dyke alignment. To be effective, preloading requires that the tailings mat be in place for a sufficient period of time before construction for the existing tailings and silty clay to substantially consolidate. Given the relatively rapid rate of consolidation expected in the existing tailings and silty clay, a period of at least three months should be sufficient to obtain a reasonable degree of consolidation. However, the use of longer periods would incrementally increase the stability because a higher degree of consolidation would be obtained before construction. This option has been evaluated for Cases 3 and 6.

As shown on Fig. 11, placing a mat of coarse tailings on the existing tailings has an appreciable effect on the overall stability of the dyke. For Case 3, the factor of safety without a mat of tailings is 1.2, with 1.5 m thick mat of tailings, it increases to 1.3, and with a 3.0 m thick mat of tailings, it is 1.5. The effect of the mat of coarse tailings is less pronounced in Case 6. A 1.5 m thick tailings mat marginally increases the factor of safety and a 3.0 m thick mat increases the factor of safety to 1.5. However, these results show that an even thicker mat of coarse tailings does not increase the factor of safety further than 1.5 because the critical potential failure surfaces will not pass through the native silty clay. For a mat of coarse tailings thicker than 3.0 m, the failure mechanisms are no longer controlled by the shear strength of the native silty clay. The potential critical surface passes through the tailings layer. For a mat of tailings with a thickness less than 3.0 m, the failure mechanisms are controlled by the shear strength of the native silty clay and the potential critical surface passes through the native silty clay layer.

This alternative was recommended and implemented because, in addition to the reduction of the potential risks of failure associated to the dyke construction, this solution has a significant impact on cost because of the substantial reduction of the amount of rockfill required for the dyke construction.

Fig. 11. Effect of variation in thickness of a mat of coarse tailings



In addition, during the dyke construction, a monitoring program that would include vibrating wire piezometers should be implemented to monitor the pore water pressure in the existing tailings especially in the first 300 m of the dyke. The piezometers should be spaced at 75 to 100-meter intervals and should be constructed to measure the pore water pressure in the approximate center of the tailings layer. The piezometers should be read on a daily basis until the pore water pressure in the tailings is approximately hydrostatic. The collected data would allow evaluating the stability of the dyke and adjusting the rate of advance or methods of construction as necessary to maintain adequate stability. In fact, this instrumented first portion of the dyke would play the role of a trial dyke to assess on the actual build-up of the excess pore water pressure during construction. The collected data would help to adjust the rate of construction if required.

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