Stability design of a tailings dam on a glaciolacustrine clay foundation

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ABSTRACT
As part of a tailings management mandate, new tailings must be stored in an inactive, existing tailings pond situated in Abitibi, Quebec. In order to confine the hydraulically deposited tailings, several dams will be built on soft, varved clays of glaciolacustrine origin. Field vane tests have revealed undrained shear strengths as low as 13 kPa. In order to ensure the timely delivery of the dams and to meet the design factors of safety at all times, the use of stability berms is proposed in conjunction with an observational design approach. As construction is done in stages, monitoring of pore pressures and consolidation will provide the field data required to update the numerical models and the design, if required.

RÉSUMÉ
Dans le cadre d'un mandat de gestion de résidus miniers, un ancien parc à résidus inactif est utilisé afin d'entreposer de nouveaux résidus. Afin de contenir les résidus déposés hydrauliquement, des barrages seront construits sur une fondation d'argile glacio-lacustre d'Abitibi. Des essais au scissomètre de terrain ont révélé des résistances au cisaillement non-drainé minimales de 13 kpa pour ces argiles varvées. Afin de respecter l'échancier de projet ainsi que les facteurs de sécurité choisis pour la conception, l'utilisation de bermes stabilisatrices est proposée, en parallèle avec une approche observationelle. Puisque la construction est faite par étapes, le suivi de la consolidation et des pressions interstistielles dans la fondation d'argile va permettre de recueillir des données de terrain et de mettre à jour les modèles numériques. La conception pourra ensuite être optimisée, si requis.

1 INTRODUCTION

As part of a tailings management mandate, Amec Foster Wheeler was asked to design a facility capable of storing 20 years of tailings production. The tailings management facility (TMF) will be located in the Abitibi region, in Quebec. An inactive, existing tailings pond has been selected to store the new tailings. In order to contain these hydraulically deposited tailings and to ensure adequate water management, several dams will be built. This paper presents some of the design challenges related to one of these dams, namely D1, which will be built over a soft clay foundation and partially on an existing dam.

As the service life of the TMF is about 20 years, construction has been divided into 4 phases. Stability analyses have been performed at several moments during this period in order to ensure stability at all times. The focus of this paper will be on static analyses, because these were found to be the most critical. Although consideration was given to other issues (such as settlement, liquefaction, downstream seepage collection and filter design), these aspects will not be discussed here.

Applicable stability design criteria will be presented, along with a brief discussion on the use of factors of safety for tailings dams. Design inputs will be assessed, with a focus on the geotechnical characteristics of the foundation and of the existing dam. The proposed design will then be detailed, and stability analyses will be presented. In addition, the proposed field instrumentation program will be discussed in the context of an observational design approach.

2 DESIGN CRITERIA

Applicable factors of safety for slope stability will be discussed in this section.

Tailings dam design in Quebec is regulated by Directive 019 of the "Ministère du Développement durable, de l'Environnement et de la Lutte contre les changements climatiques (MDDELCC)". Table 1, presents a summary of applicable safety factors in static conditions.

Table 1. Required factors of safety for static slope stability (MDDELCC, 2012)

Zone	Loading condition	Minimal factor of safety
Upstream and downstream	Slope stability, end of each stage of construction (short-term)	1.3 to 1.5
Upstream and downstream	Slope stability in stationary conditions (long term)	1.5

For end of construction stages, a factor of safety ranging between 1.3 and 1.5 is proposed for both upstream and downstream slopes, allowing the designer a choice. Although this is not stated in Directive 019, it is assumed that this choice should be based on such considerations as site conditions, available geotechnical information, proposed raises, consequences of failure and use of instrumentation. In the long-term, a factor of safety of 1.5 is required.

The latest Canadian Dam Association (CDA) Technical Bulletin focuses on the application of the CDA Dam Safety Guidelines for tailings dams. Table 2 presents a summary of applicable safety factors in static conditions.

Table 2. Required factors of safety for static slope stability (CDA, 2014)

Loading condition	Minimum factor of safety	Slope
During or at the end of construction	> 1.3 depending on risk assessment during construction	Typically downstream
Long term (steady state seepage, normal reservoir level)	1.5	Downstream

During or at the end of construction, the CDA Bulletin indicates that a factor of safety larger than 1.3 should be considered based on risk assessment during construction. Moreover, the long-term condition is characterised by steady-state seepage and normal reservoir level. In this case, the CDA recommends a factor of safety of 1.5.

Both Directive 019 and the CDA 2014 Technical Bulletin suggest that a factor of safety larger than 1.3 can sometimes be required during or at the end of construction of tailings dams.

In the case of dam D1, a minimum factor of safety of 1.5 for the downstream slope has been chosen in both short and long-term loading conditions, considering expected consolidation of the clay foundation and the fact that operation of the TMF will likely begin before long term conditions are reached. Moreover, this dam will be partially built over an existing dam, where design standards could not be validated, a higher factor of safety was chosen with regards to downstream failure during construction. Indeed, such a failure could lead to a release of existing tailings into the environment. Table 3 presents a summary of the factors of safety that have been used in this project for static conditions.

Table 3. Design factors of safety for static slope stability

Zone	Loading condition	Minimal factor of safety
Upstream	End of each stage of construction (short-term)	1.3
Downstream	End of each stage of construction (short-term)	1.5
Downstream	Slope stability in stationary conditions (long term)	1.5

3 GEOTECHNICAL SITE CONDITIONS

A geotechnical investigation campaign was conducted in order to characterise soil deposits found on site. Several field and laboratory tests were performed in order to gather the necessary information, including field vane tests, grain size analyses, liquid and plastic limits, fall cone tests and consolidations. The results of this campaign are presented in the following sections.

3.1 Site stratigraphy

Several boreholes in the area of Dam D1 revealed a superficial layer of sand or silty sand, approximately 0.6 m thick, followed by a clay deposit, approximately 8 m thick. Immediately below the clay layer, a till horizon was encountered, reaching an approximate depth of 2 m, and resting on the bedrock. This bedrock was found to be of fair to excellent quality.

It should be noted that the clays found in this region are generally varved. As such, they are composed of alternating layers of coarser (silt or sand) and finer (clay) materials that were formed when sediments were deposited at the bottom of glacier lakes, namely Lake Barlow-Ojibway (Quigley, 1980). Such varved clays were found in all boreholes performed on site.

3.2 Existing dam

As mentioned, the proposed dam will be partially built on top of an existing dam, as represented in Figure 1. This existing dam was built in several stages, and construction drawings were only available for the last raising, which occurred in the early 1980s. At the time, an inclined clay core was placed on top of a granular fill in order to raise the dam. This geometry was confirmed with boreholes drilled on the actual crest of the dam. These revealed 5 m of clay fill, followed by 6 m of granular fill, resting on a clay layer. The latter has been identified as the natural ground. The exact geometry of the original dam is not known and will be confirmed during early construction stages. Due to the fact that it has been raised in the past, it is believed that it is composed of an upstream, inclined clay core.

An existing tailings basin is situated upstream of this dam, with a water depth of approximately 7 m. At the bottom of the basin, a 4 m layer of soft tailings was encountered resting on the natural clay. Downstream of the dam is the natural ground and a water pond. Because of the proximity of this pond, the centreline of the proposed dam had to be positioned upstream of the existing dam.

3.3 Geotechnical characteristics

The geotechnical characteristics of the clay foundation are summarised in Table 4.

Table 4. Geotechnical characteristics of the clay foundation

Parameter	Value
Density (kN/m^3)	15
Undrained shear strength - C_u (kPa)	13 to 33
Plasticity Index (%)	22 to 26
Liquidity Index	0.93 to 0.94
Compression index - C_c	0.73 to 1.71
Recompression index - C_r	0.04 to 0.08
Effective preconsolidation stress - σ'_p (kPa)	58 to 88
Overconsolidation ratio (OCR)	1.30 to 1.42
Coefficient of consolidation - C_v (m^2/s)	1.35E-09 to 7.23E-09

The undrained shear strengths presented in Table 4 were measured using vane shear tests in two boreholes downstream of the existing dam. In the Year 1 stability analyses, the lowest values (13 and 16 kPa) found in each downstream borehole were used, except under the existing dam (see Figure 2 and Figure 3).

At this time, no vane shear tests have been performed on the clay located below the existing dam, which has been in place for more than 30 years. Increases in undrained shear strength are expected due to consolidation under the weight of 11 m of embankment. A numerical assessment of this increase has been performed using a ratio of undrained shear strength to overburden stress (C_u/σ'_p) of 0.22 (Lacasse et al., 1977; Leroueil et al., 1983). Adding this increase to an initial strength of 13 kPa yielded values of 25 and 20 kPa under the different portions of the existing dam.

An undrained shear strength of 25 kPa was also measured using laboratory fall cone tests. Nonetheless, these values will be confirmed through additional vane shear and triaxial tests.

It should be noted that the coefficient of consolidation (C_v), measured during consolidation tests, corresponds to normally consolidated conditions.

3.4 Material properties

The mechanical properties of the various materials used in stability analyses are presented in Table 5. The foundation bedrock has been modelled as impenetrable, and thus was assigned no parameters. Common values were used for granular soils (till, sand and gravel, rockfill, tailings and soft tailings), based on engineering experience and results from the literature (Holtz and Kovacs, 1981).

Cohesion values for the undrained (U) natural clays are based on field vane tests and numerical calculations. The cohesion of the undrained (U) compacted clay will be validated through additional field vane tests.

Finally, the friction angles for the drained (D) clays are based on engineering experience and sources in the literature (Holtz and Kovacs, 1981; Leroueil et al., 1983).

Table 5. Mechanical properties of materials

Material	Unit weight (kN/m^3)	Cohesion (kPa)	Friction angle (°)	Color
Bedrock	N/A	N/A	N/A	
Till	20	0	38	
Sand and gravel	19	0	35	
Rockfill	20	0	40	
Tailings	15	0	28	
Soft tailings	15	0	20	
Compacted clay (U)	18	50	0	
Natural clay (U)	17	13-28.8	0	
Compacted clay (D)	18	0	28	
Natureal clay (D)	17	0	25	

4 DESIGN APPROACH

An observational design approach has been used in this project, with a focus on the stability of the soft clay foundation. The first construction phase has been stretched over 3 years in order to allow for the consolidation of the foundation, to observe its behaviour and to optimise the design, if possible. As such, work during the first year of construction is limited. Nonetheless, in order to reach design factors of safety at all times and to ensure the timely delivery of the TMF, the use of stability berms was favoured. Total stress stability analyses were performed, using the results of field vane readings. Conservative estimates of strength gains in the clay foundation due to consolidation were accounted for in stability analyses, based on the previous ratio of undrained shear strength to overburden stress (C_u/σ'_p). Using the coefficient of consolidation (C_v), the increase in consolidation with time was evaluated.

As part of the observational design approach, these calculations will be validated by field measurements during and after construction. Monitoring of Dam D1 will be performed as soon as construction starts using vibrating-wire piezometers, settlement plates and surveying. The objective is to obtain field data that will be used to update the design models, and also to allow monitoring of pore pressures during construction.

Additional clay samples will be collected for triaxial testing before construction starts, along with piezometer installation. Unconsolidated-undrained (UU) and Consolidated-Undrained (CU) tests will be performed, with the latter providing the parameters required to perform effective stress stability analyses. In turn, these will be used to determine what pore pressures are tolerable in the field.

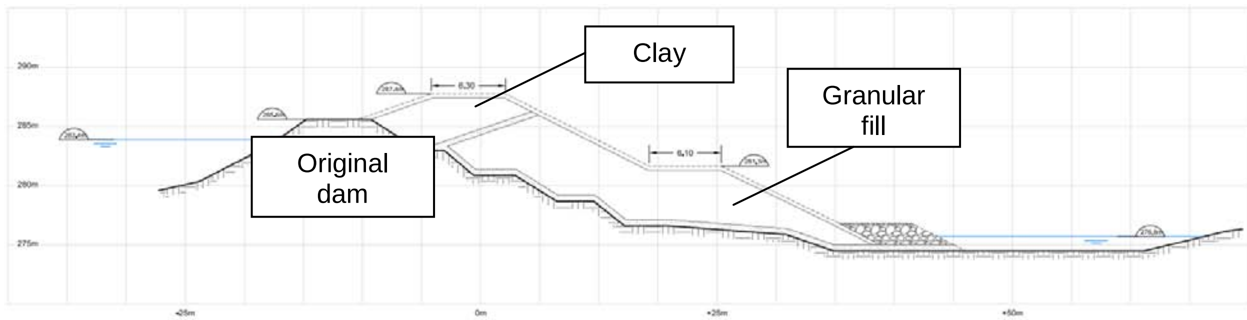

Figure 1. Cross-section of existing dyke, showing known geometry

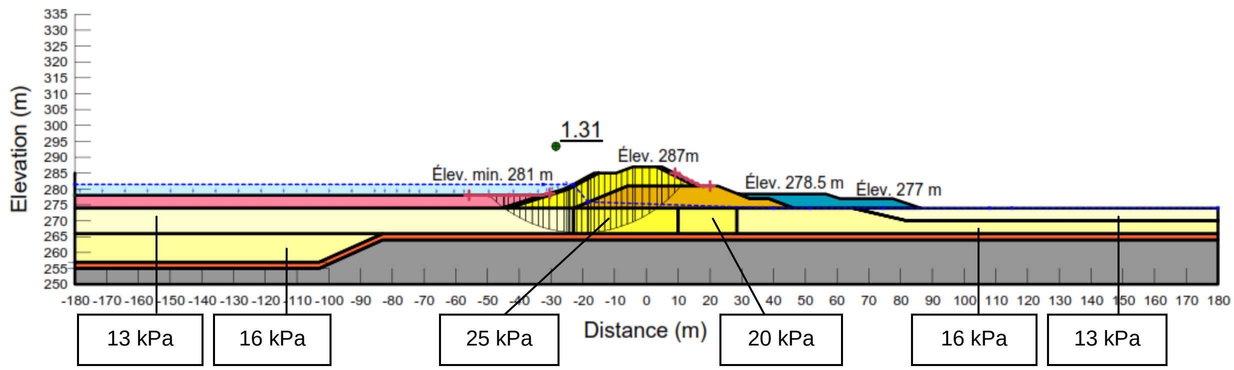

Figure 2. Year 1, undrained, upstream stability analysis

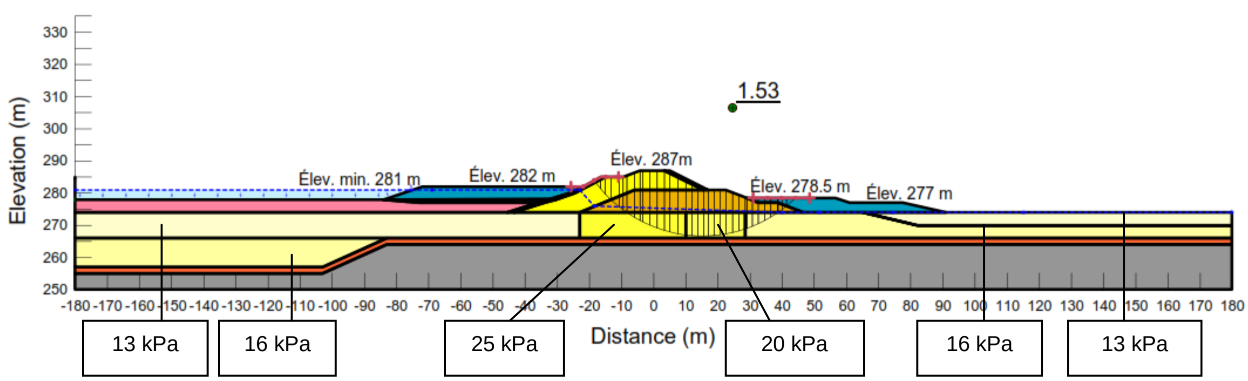

Figure 3. Year 1, undrained, downstream stability analysis

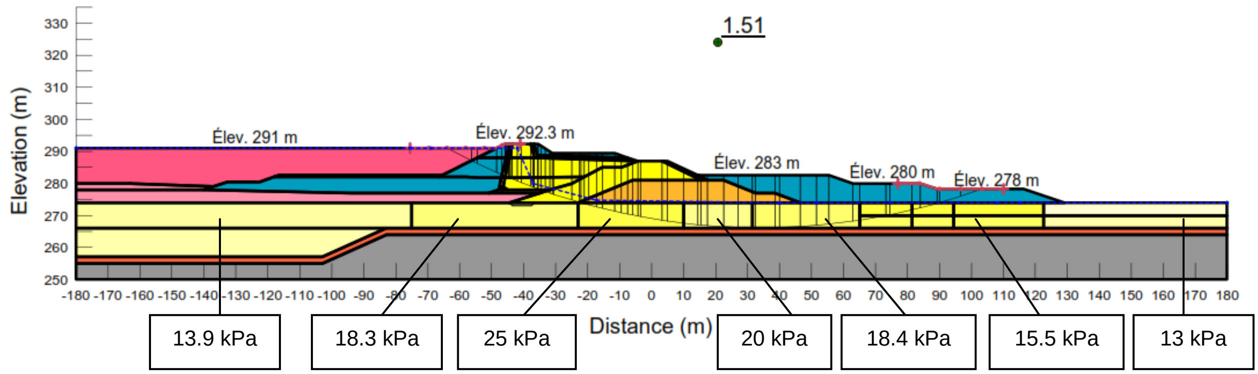

Figure 4. End of Phase 1, undrained, downstream stability analysis

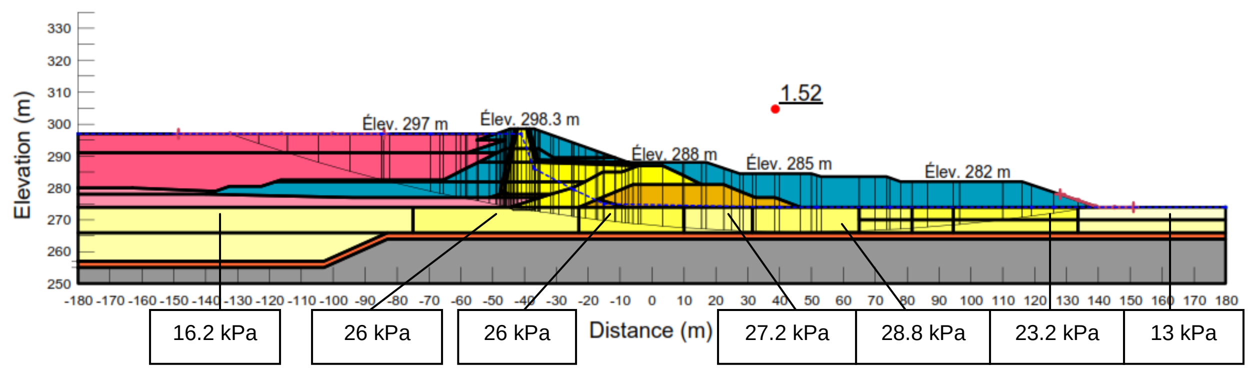

Figure 5. End-of-life, undrained, downstream stability analysis

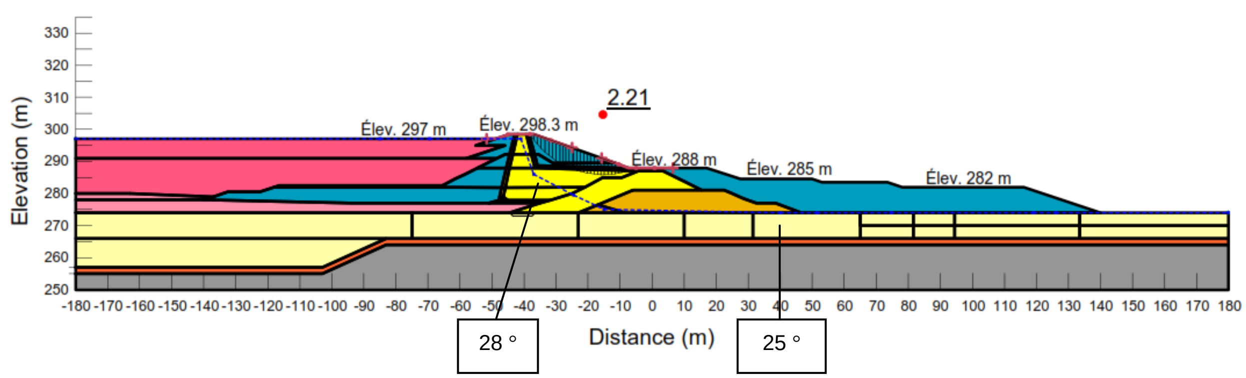

Figure 6. Long-term, drained, downstream stability analysis

5 DESIGN AND STABILITY ANALYSES

Dam D1 will consist of a central clay core, supported by rock-fill shoulders and berms. Filter graded material will be used between the clay and the rock-fill. Following the first phase of operation, the overall downstream slope will be 9H : 1V.

The proposed construction sequence is presented in the following sections along with corresponding stability analyses. The factors of safety obtained in static stability analyses are presented in Table 6, for upstream (US) and downstream (DS) slopes.

It should be noted that short-term, undrained loading conditions were the most critical at all construction stages, and thus long-term conditions are only presented using the final height of the dam. Moreover, no layers have been modelled in the clay foundation despite its varved nature. Back-analysis of failures in varved clays has shown that circular, clay-like failures surfaces are adequate to represent this type of soil (Lacasse et al., 1977).

Table 6. Results of static stability analyses

Year	Slope	Conditions	Crest elevation (m)	Factor of safety
1	US	Undrained	287.0	1.31
1	DS	Undrained	287.0	1.53
2	US	Undrained	288.0	1.30
2	DS	Undrained	288.0	1.65
3	US	Undrained	292.3	1.42
3	DS	Undrained	292.3	1.63
End of phase 1	DS	Undrained	292.3	1.51
End-of-life	DS	Undrained	298.3	1.52
Long term	DS	Drained	298.3	2.21

5.1 Year 1

Minimal work is planned for Year 1, as more geotechnical information is required before starting further stages. Thus, part of the downstream berm will be built first in order to reach a factor of safety of 1.53 for the downstream slope of the existing dam, as illustrated in Figure 3. Once this is done, the water level in the existing basin will be slowly lowered, but not below elevation 281 m. This is necessary to maintain a factor of safety of 1.31 in the upstream direction (see Figure 2). Then part of the upstream berm will put in place in the existing basin, starting from the shore and up to elevation 282 m. With both slopes of the existing dam secured, the upstream berm will be used as a platform to perform additional geotechnical investigations.

As mentioned previously, the exact core configuration of the existing dam is not known. It will thus be confirmed at this stage. In addition, field vane measurements will be taken in the upstream foundation and directly below the existing dam. These will validate some of the assumptions made regarding undrained shear strength parameters in these zones. Finally, vibrating wire piezometers will be installed in order to monitor the variation of pore-pressures during further construction stages.

5.2 Year 2

The first step proposed for Year 2 is to extend the downstream berm in order to maintain a factor of safety of 1.5 while raising the dam. Once this is completed, the existing water basin will be emptied, providing room to extend the upstream berm. This is necessary in order to maintain an upstream factor of safety of 1.3 while raising the dam. In order to connect to the existing core, the upstream berm placed in Year 1 will need to be excavated at the upstream toe. Excavation will be performed one section at a time and then filled in order to ensure the upstream stability. The core and upstream embankment will then be raised to elevation 288 m, while maintaining a downstream factor of safety of 1.65. It should be noted that the option to start the tailings deposition one year ahead of time (in Year 2) has been studied and meets stability requirements. As with further stages of operation, a 1 m freeboard should be maintained between the crest of the dam and the tailings discharge point.

5.3 Year 3

During Year 3, the dam will reach its final configuration for Phase 1. Upstream and downstream berms will be completed and the dam will be raised to elevation 292.3 m. Factors of safety are kept above requirements, reaching 1.42 and 1.63 for upstream and downstream slope stability.

5.4 End of Phase 1 of operation

At the end of Phase 1, tailings will reach elevation 291 m in the basin. The proposed stability berm allows a factor of safety of 1.51 to be maintained at this stage, as illustrated in Figure 4.

5.5 End-of-life of TMF

Detailed steps of construction for Phases 2 and 3 will be designed using the same approach, in order to ensure that the design stability criteria are met at all times.

The end-of-life of the TMF is presented here, with the crest and tailings reaching final elevation at 298.3 m and 297 m respectively. The downstream berm is extended in order to reach a factor of safety of 1.52 in undrained conditions, as shown in Figure 5. For long-term fully drained conditions, a factor of safety of 2.21 is obtained, as shown in Figure 6.

6 CONCLUSION

The objective of this paper was to present how a tailings dam built on soft clay was designed in order to ensure adequate stability and the timely delivery of the TMF. An observational design approach was used, and stability berms were designed to provide a factor of safety of at least 1.5 for the downstream slope, in both undrained and drained conditions and for every step of the project. During construction, the upstream slope stability will be ensured by keeping water in the existing basin at a specified level.

Because this dam will be built using a staged construction approach, increases in shear strength in the clay foundation were accounted for using consolidation theory and available field data. These were then used in the total stress analyses presented here. Additional field vane and triaxial tests will be performed during the first year of construction in order to validate these values.

As part of the observational design approach, Dam D1 will be monitored and field data will be collected. In turn, stability models will be updated, and, if possible, the design of the dam will be optimised.

REFERENCES

Canadian Dam Association (CDA), Technical Bulletin: Application of Dam Safety Guidelines to Mining Dams, *Canadian Dam Association*, 2014.

Holtz, R.D and Kovacs, W.D. 1981, *Introduction à la géotechnique*, Presses internationales Polytechniques, Montréal, Québec, Canada.

Lacasse, S., Ladd, C. and Barsvary, A. 1977, Undrained behavior of embankments on New Liskeard varved clay, *Canadian Geotechnical Journal*, 14 (3): 367 - 388.

Leroueil, S., Tavenas, F. and Le Bihan, J-P. 1983, Propriétés caractéristiques des argiles de l'est du Canada, *Canadian Geotechnical Journal*, 20 (4): 681 - 705.

Ministère du Développement durable, de l'Environnement et de la Lutte contre les changements climatiques (MDDELCC), Directive 019, *Gouvernement du Québec*, 2012.

Quigley, R. M. 1980, Geology, mineralogy and geochemistry of Canadian soft soils: a geotechnical perspective, *Canadian Geotechnical Journal*, 17 (2): 261 - 285.