BANK EROSION CONTROL AT FREDERICKHOUSE DAM, ONTARIO

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ABSTRACT

Strong turbulence from discharge at the Frederickhouse Lake Control Dam, combined with the channel geometry, created significant erosion at the toe of the left bank downstream of the structure. Undercutting of the 28 m high slope led to several small failures and eventually one mass slope movement involving 10,000 m^3.

Monitoring of slope regression following the mass movement and a detailed geotechnical investigation identified the need to rehabilitate the downstream area to arrest the erosion. The nature of geology, soil stratigraphy and the mechanics of the discharge channel, which lacked energy dissipation controls, presented a number of challenges in arriving at the final design concepts. The slope rehabilitation work was carried out in 2014.

The paper describes the results of detailed laboratory and in-situ testing to derive the soil parameters for slope stability modeling. Also described is a relatively complex model using Flow-3D to assess the impacts of turbulent flow on the design strategy for the slope stabilizing berm and analysis to support the selection of the armourstone and riprap.

RÉSUMÉ

Un fort écoulement turbulent en aval du barrage de Frederickhouse Lake, de concert avec la géométrie du chenal d'évacuation, a causé une érosion importante en rive gauche au pied aval de la structure. L'affouillement d'une pente de 28 m de hauteur a provoqué de multiples petites ruptures de talus et un glissement de pente massif d'environ 10 000 m^3.

Après le glissement, la surveillance de la régression du talus ainsi qu'une investigation géotechnique détaillée ont identifié la nécessité de travaux en aval de l'ouvrage afin d'arrêter l'érosion. La géologie, la stratigraphie des sols et les paramètres mécaniques du canal de décharge, sans dissipation d'énergie, ont présenté de nombreux défis avant d'arriver à la conception du design final. Les travaux de stabilisation des berges ont été réalisés en 2014.

Cette publication présente les résultats détaillés des essais réalisés in situ et en laboratoire afin d'obtenir les paramètres géotechniques requis par le modèle d'analyse de stabilité des pentes. Ce document décrit aussi un modèle d'écoulement relativement complexe utilisant Flow-3D afin d'évaluer les impacts de l'écoulement turbulent sur la stratégie de conception de la berme de stabilisation du talus ainsi que l'analyse requise pour la sélection du roc de blindage et du riprap.

1 INTRODUCTION

Frederickhouse Lake Control Dam, a 21 m high and 140 m long concrete dam constructed in 1937 and 1938 in Northern Ontario, controls the water level in Frederickhouse Lake which stores water for power generation at Abitibi Canyon Generating Station.

The dam's downstream channel is divided by a 60 m long bedrock outcrop on which a training crib mounted to an elevation of 865 ft (263.6 m) during original construction. In addition, there were 30 m long timber cribs with rockfill to elevation of 865 ft (263.6 m) built (refer to B in Figure 1), next to that, a 40 m long rip-rap of a 1:1.5 (V:H) slope was constructed (refer to B and C in Figure 1) in the south bank (left bank). These features were intended to provide erosion protection along the south bank.

At some point since completion of this dam, the training cribs on the 60 m long bedrock outcrop were washed way. The south bank experienced serious erosion: half of the crib was damaged (A to B), 40 m rip-rap slope was flattened and its downstream native soil bank was washed away by up to 8-10 m (refer to D and E). South bank slope is 28 m high. The slope toe erosion mainly occurred along a 60 m long bank between C and F. Point F is a location where no significant erosion occurs. Point G, minor erosion, is located at OPG property boundary.

Slope toe erosion at south bank led slope failure. In spring of 1995 (Paulen 2002), heavy and prolonged rainfall combined with unusually rapid snowmelt, triggered a large landslide, up to 8000 m^3 to 10000 m^3, at south bank (refer to C to E).

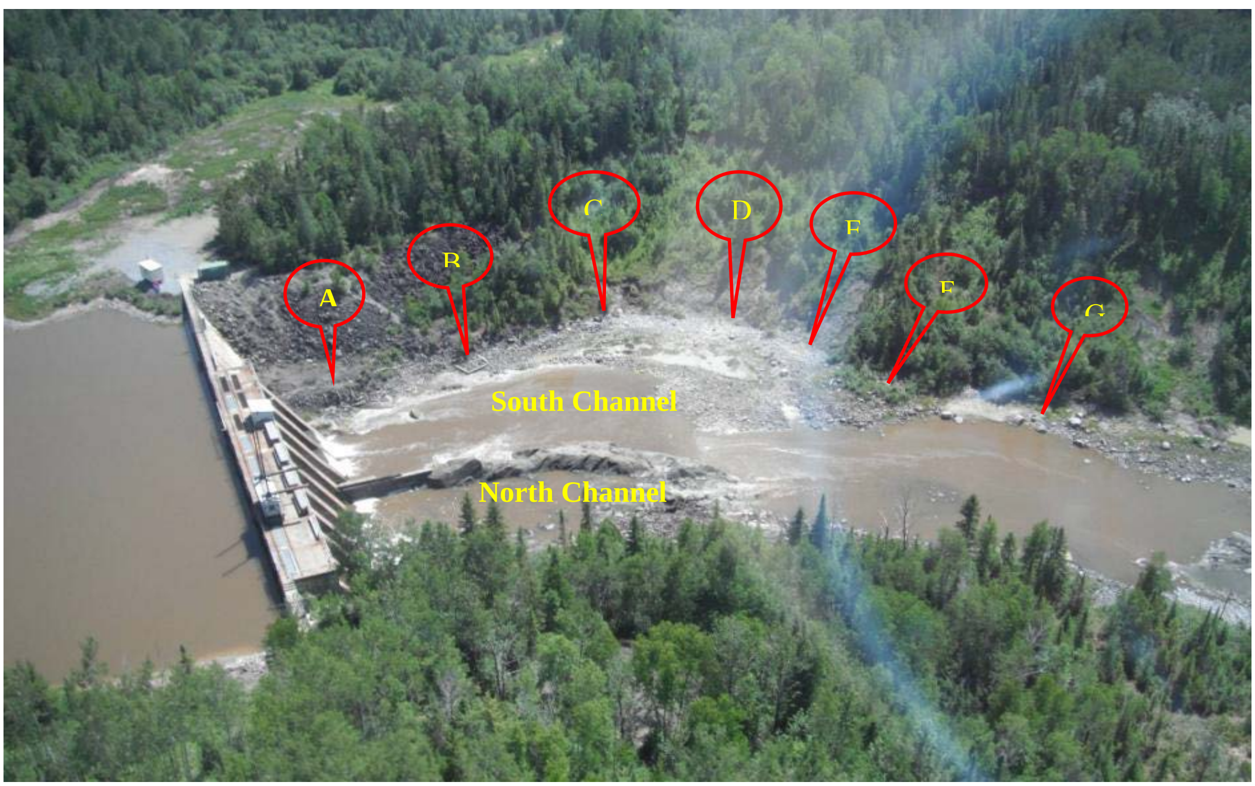

Figure 1: Frederickhouse dam and its downstream channel on 7 July 2010

The nature of geology, soil stratigraphy and the mechanics of the discharge channel, which lacked energy dissipation controls, presented a number of challenges in arriving at the final design concepts. The paper describes the results of detailed laboratory and in-situ testing to derive the soil parameters for slope stability modeling. A Flow-3D was used to assess the impacts of turbulent flow on the design strategy for the slope stabilizing berm and analysis to support the selection of the armourstone and riprap. The slope rehabilitation work was carried out in 2014.

2 DESIGN CONSIDERATIONS

2.1 Geological Setting and Slope Stability

2.1.1 Geological Setting

Over the bedrock, Precambrian granitic gneiss (metasediment), which exists below El.259 m, the south bank slope is comprised of three geological units: Matheson Till (6.6 m), Lake Ojibway Formation (8.5 m) and Cochrane Formation (7 m). Matheson Till is silty-sand compact basal till overlain by clay-rich, clast-poor Lake Ojibway and Cochrane deposits of glaciolacustrine sediments consisting of over 40% clay, which has higher plasticity index and high expansion activity values.

According to regional quaternary geology studies (Boissonneau 1966 and McClenaghan 1992), the Matheson till was deposited during the Late Wisconsinan glacial period, approximately between 18000 and 10000 years ago, during that period, an ice mass advance in a southeasterly direction. Postglacial uplift was up to 12 m, indicating a huge thickness of the ice sheet.

As the ice front melted northward out of this region between 10,000 and 9000 years ago, it released large volume of water into a series of immense ice-dammed lakes, such as Lake Agassiz (Thorleifson). East part of Lake Agassiz is called Lake Ojibway where Lake Ojibway and Cochrane formation were deposited.

In general, Matheson till, dense moraine, is formed in late Pleistocene Period; Lake Ojibway and Cochrane formation, fine-grained deposit, are deposited in the later Holocene period. This geological feature determines geotechnical properties of the slope soils. Matheson till is relatively compact and stiff; overlain clay is sensitive to water content; soft when wet and hard when dry.

The 1995 landslide (Paulen 2002) was mainly caused by clay softening after heavy rainfall and snow melting, a perched water table that overlies the impermeable clayey sediments affects the natural pore pressure and induces changes in shear strength in that abnormally wet season. In addition, significant toe erosion also contributed the slope failure. The failure produced approximately 8,000 to 10,000 m^3 of clayey silt debris.

Since the massive landslide in 1995, the exposed slope is very unstable and continues to fail on a yearly basis. Erosion is continuing at slope toe due to operation of Frederickhouse dam. Surveys in 2001 and 2008 at the same section show a combination of erosion at the toe of the slope along with flattening on the slope surface.

2.1.2 Slope Stability

Although current slope state is globally stable (Figure 1), the surface and shallow failures have been occurring, in particular, continuation the erosion of Matheson till (Figure 2) during the high flood seasons could lead to further slope failure.

Figure 2: Ojibway silt/Matheson till at 266 m. Till is eroded in high flows.

A site investigation including borehole drilling, cone penetration testing (CPT) and soil strength laboratory testing was carried out and soil properties are recommended as in Table 1.

Table 1: Slope stability geotechnical parameters

Geological Unit	Stratum	c' (kPa)	Φ' (degrees)	Unit Weight (kN/m^3)
Cochrane	Brown Silty Clay	1.0	28	17.0
	Grey/Brown Silty Clay to Clay	5.0	23.9	17.0
Lake Ojibway	Grey, Varved, Silty Clay to Clay	1.5	32	17.0
	Silt	0	33	17.5
Matheson Till	Sand and Gravel	0	33	19.0
	Sandy Silt	0	35	19.0
Precambrian	Bedrock	0	45	20.0

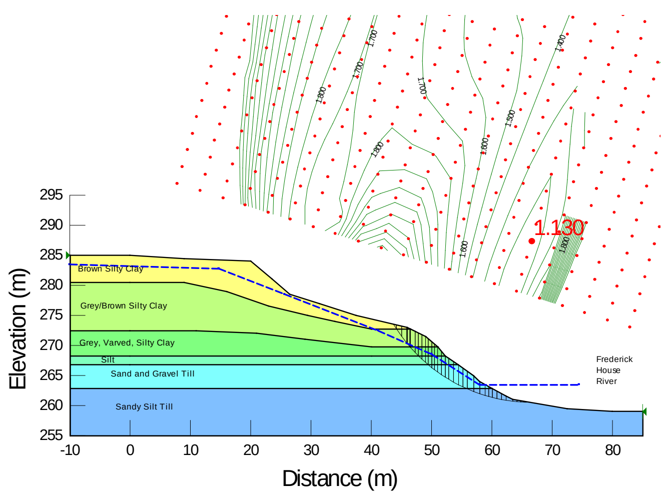

Figure 3: Global failure without a berm

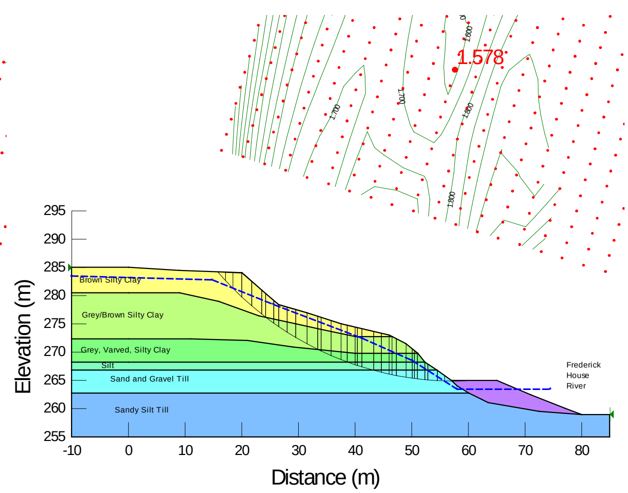

Figure 4: Global failure with a berm

The above parameters are used to model current slope state which has a Factor of Safety (FOS) are 1.02 (surface failure) and 1.13 (global failure in Figure 3), respectively. That reasonably well describes the present state of the slope.

A proposed stabilization rip-rap berm (Figure 4) increases the Factor of Safety (FOS) of global failure to 1.6 which meets requirements of highway and OPG dam safety standards.

2.2 Flow Pattern and Toe Erosion

2.2.1 Field Observation

During project initiation stage, the author happened to be on site and observe a high flow pattern on August 15, 2013. According to log operation sheet, the flows of August 15, 2013 varied from 141.8 to 274.10 cms with an average of 181.8 cms. During on-site observation, the flow rate is 221.7 when 6 logs were removed out from Sluiceway # 2, 3, 4 and 5, respectively; 4 logs was out from Sluiceway #6.

Figure 5: Flow pattern looking toward dam

Figure 6: Flow pattern at slope toe

Figure 7: Water level marks and erosion after high flows

Flow in the downstream channel was very turbulent (Figure 5 and 6), including features such as hydraulic jumps, flash overtop bedrock crops, reverse flow, vortexes, and sediment suspended along the slope toe.

Based on a topographic map and compared with on-site observations on October 9 (Figure 7), the water level is visually estimated to be at 261.5 m for a flow of 221. 7cms on August 15, 2013.

There was erosion evidence along the toe of slope, such as new fallen trees, fresh soil scarps. Existing riprap also experienced erosion and movement. Historical placement (1938) of angular riprap of 0.6 m mean diameter along a 40 m segment (B in Figure 7), has been eroded by flow. Rocks over 1m (D in Figure 7) didn't show signs of movement, suggesting 1m diameter armourstone should resist the erosion.

2.2.2 Design Flow

In 2010 Dam Safety Periodic Review (DSPR), Frederickhouse Dam is assessed in accordance with the accepted engineering standards in both the Ontario Power Generation's (OPG) Dam Safety Assessment Program and the Draft version of the Ontario Provincial Dam Safety Guideline (ODSG, 2009). The ICC classification under OPG guideline is HIGH and the HPC under the Ontario guideline is VERY HIGH.

According to 2010 DSPR, the 50, 100 year return period floods are 402 and 453 cms, respectively. Historically, there were six (6) flow rates higher than 300 cms since 1950, the highest is 435.7 cms.

A flow of 400 cms is selected as design flow for this slope erosion project. 400 cms is approximately 90% of 100 year return period flood (453 cms) and close to 402 cms of a 50 year flood.

This design flow of 400 cms occurs in summer season (Victoria Day weekend to Thanksgiving Day) which mainly requires up to 8 logs removed from Sluiceway # 2, 3, 4, 5 and 6.

In winter period, the all underwater logs in Sluiceway #8 and 9 are removed and 2 or 3 logs out from Sluiceway #2 to 6. Historic highest winter flow was 226.9 cms on November 27, 2003. 200 cms is selected as winter design flow.

2.2.3 Flow-3D Modeling

Based on 2010 LiDAR data, 2013 bathymetric survey data, sluiceway configuration, a Flow-3D model was set up. The model setup was calibrated using the flow pattern in downstream channel at a flow on August 15, 2013.

The simulated water levels at Point A, B, C, D and E are less than 0.1 m in discrepancy comparing with that visually estimated on that time. However, in the north channel the simulated water levels are lower than visual estimates, which is due to no air entrainment and cavitations being simulated in the Flow-3D model. Significant air entrainment was visible at high flow. Even with this discrepancy, this Flow-3D model setup is calibrated and believed to be a good tool to simulate flow pattern and design options at south channel.

Design flows both summer and winter can be reached by various log operation scenarios. A summer design flow of 400 cms is simulated on a configuration of 8 logs out from Sluiceway #2 to 6 at a reservoir water level of 274.10m.

The simulated summer design flow pattern is shown in Figure 8 and its hydraulic conditions in the downstream channel are summarised in Table 2.

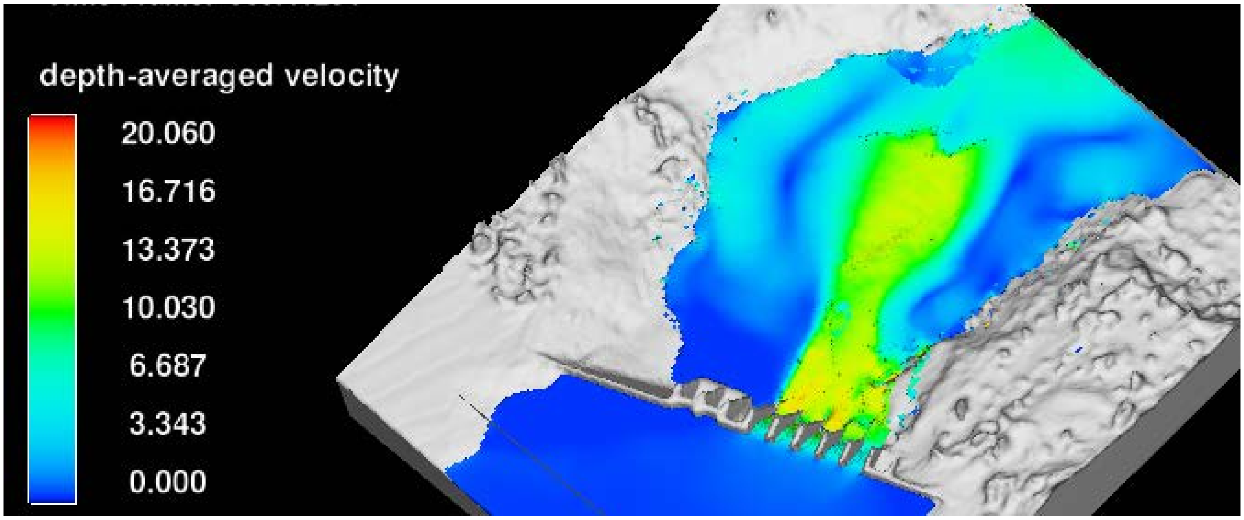

Photo 8: Depth-average velocity pattern of design flow (400 cms)

Table 2: Hydraulic conditions of slope toe at design flow (400 cms)

Chainage (m)	Existing Condition	Water Surface Elevation (m)	Depth-averaged Velocity (m/s)	Water Depth (m)
0 to 60	Rip-rap slope of 1V:2H	262.2 to 262.9	0.1 to 3.5	2.1 to 3.0
60 to 110	Steep till cliff	262.8 to 264.0	3.5 to 6.3	2.9 to 3.4
110 to 130	Flat till slope with boulder and vegetation, approximately slope 1V:3H	264.0 to 265.6	1.5 to 3.5	1.9 to 2.5
130 to 160	Some cracks at top of the slope hill, some erosion	262.4 to 265.5	2.4 to 8.5	2.2 to 3.6

The Flow-3D model also simulated flow patterns affected by a proposed berm at toe area along the south bank slope and indicates the flow would be improved and reduced flow erosion capability.

2.3 Armour Stone Size and Berm Design

2.3.1 Mean Diameter of Armour Stone

Hydraulic Engineering Centre (HEC) of US Army Corps of Engineers publications –Design of Riprap Revetment (USACE, 1989) recommends following equation for median diameter size (D_{50}) of riprap design:

$$D_{50} = 0.007\, V^3/({d_{avg}}^{0.5} K_1^{1.5}) \quad (1)$$

Where:
V = Average section velocity
d_{avg} = Average section water depth
K_1 = The bank slope correction term, this is defined as

$$K_1 = [\,1 - (\sin \emptyset^2 \,/\, \sin \theta^2)]^{0.5} \quad (2)$$

Where:
$\theta =$ The bank angle with the horizontal; and
$\emptyset =$ The riprap material angle of repose.

Rip-rap is crushed ledge rock or very angular native rocks; its angle of repose is 41 degree according to Appendix C of HEC (USACE, 1989). A 1: 2 (vertical vs. horizontal) rip-rap slope is assumed at this stage. Thus,

$$K_1 = 0.73 \tag{3}$$

Based on a Flow-3D modeling results in Table 2,

$$\begin{aligned} V &= 5.5 \text{ m/s} \\ d_{avg} &= 3.3 \text{ m} \end{aligned} \tag{4}$$

Thus, the median size of rip-rap is

$$\begin{aligned} D_{50} &= 0.007\, V^3/({d_{avg}}^{0.5} {K_1}^{1.5}) \\ &= 0.007 \times 5.5^3/(3.3^{0.5} \times 0.73^{1.5}) \\ &= 1.03m \approx 1.0\text{m} \end{aligned} \tag{5}$$

The riprap gradation in Table 3 meets the requirement of HEC (USACE, 1989).

Table 3: Gradation limits of riprap

Diameter (m)	Volume (m^3)	Weight (tons)	Percent of Smaller Than
1.70	2.57	6.81	100
1.50	1.77	4.68	
1.40	1.44	3.81	85
1.20	0.90	2.40	
1.00	0.52	1.39	50
0.60	0.11	0.30	
0.40	0.03	0.09	15

2.3.2 Zoning of Riprap Berm

A riprap berm was placed along the south bank toe to an elevation of 265 m as a minimum, the slope varies:

- 1V:2H, Ch 0 to 50
- 1V:2.5H, Ch 60 to 110
- 1V:3H, Ch 120 to 160.

In addition to hydraulic conditions, the berm design aims to minimize disturbance of existing slope and minimize occupation of the current channel width, fill materials from outside to inside of the berm are as follows:

- Zone 1: armour-stone, 400 mm to 1700 mm
- Zone 2: Rip-rap, 25 mm to 300 mm
- Zone 3: Granular B

The Zone 1 (armour-stone) thickness is suggested to be 3 time of medium diameter at least; thus the top width of the rip-rap is set as 3 meter. An excavator with a hydraulic thumb was used to strategically place the armourstone to achieve the maximum interlock to provide better resistance to turbulent undercutting and erosion.

Zone 2 is a transition zone with a thickness of 4 m with both side slope of 1V:1H. Zone 3 fills the gap between the existing ground and Zone 2. Zone 2 prevents sand in Zone 3 migrating into Zone 1 and eroding into the river channel. However, there is no Zone 2 placed between Ch 00+00 to 50+00 where the Granular B was placed over existing 1938 riprap to provide access road for site investigation. This design aims to minimize the encroachment into the river channel.

3 CONCLUSIONS

An understanding of the geologic setting and completing in-situ and laboratory soils investigations and testing resulted in the selection of realistic soil strength parameters that when used in slope stability assessments have good agreement with observed behaviour. The use of Flow 3D simulations were effective in determining the expected flow velocity and water elevation such that an efficient design of erosion protection and slope stabilization could be undertaken to resist the design flow of 400 cms (summer) and 200 cms (winter). The as-constructed slope meets OPG's slope stability requirements.

Construction of this project was completed by OPG internal construction crew in fall of 2014. The berm experienced high flows up to 240 cms on October 20, 2014 and performed satisfactorily. Monitoring and inspection of this slope and the armourstone/riprap berm are included into the Frederickhouse Dam operation and maintenance procedure.

4 ACKNOWLEDGEMENT

Authors would like to appreciate OPG for approval to publish this paper, and grateful to Allan Zheng and Jim Lamarche who were the project engineer and the project manager of this erosion control project, respectively.

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