HATCH energy

Hamilton Conservation Authority

Ancaster, Ontario

Dam Stability and Condition Assessment

Crooks' Hollow Dam

Final Report

July 2007

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(d) cost estimates are based on several factors over which Hatch Ltd. has no control, including without limitation site conditions, cost and availability of inputs, etc.; and Hatch Ltd. takes no responsibility for the impact that changes to these factors may have on the accuracy or validity of this estimate;

(e) the report is based on information made available to Hatch Ltd. by HCA; and Hatch Ltd. has not verified the accuracy, completeness or validity of such information, makes no representation regarding its accuracy and hereby disclaims any liability in connection therewith.

Executive Summary

The Crooks' Hollow Dam is owned and operated by the Hamilton Conservation Authority (HCA) and is located on Spencer Creek in the community of Greensville within the City of Hamilton. The dam was originally constructed in 1913 to supply potable water to the community of Dundas. Years later, this use ceased after a municipal supply of water was established for the village. Between 1959 and 2001, the Dundas Valley Golf and Curling Club used the reservoir as a source for irrigation water. The reservoir and surrounding lands currently provide recreational opportunities that include hiking, fishing and limited boating. The reservoir provides no meaningful flood attenuation.

The dam is a concrete structure approximately 6.1 m high and 36.6 m long with four stop-log spillways. The condition of the dam is considered to be fair. Noted deficiencies include poor condition of the concrete surface on the below-water upstream side and on portions of the downstream spillway end wall, fill settlement associated with the north abutment, dislodgement of the downstream spillway wall and seepage. A previous 1993 study concluded that the dam would not be able to withstand a major storm event if the dam was operated at its normal operating water level (Peto MacCallum Limited, 1993). To confirm the current (2005) condition and stability of the dam, the HCA has initiated a dam safety review and stability assessment study of the dam.

Given the dam's age and deficiencies, it is apparent that the Crooks' Hollow Dam requires significant corrective rehabilitation to ensure its safe operation under major storm events or it should be decommissioned and either removed or modified into an overflow weir. For these reasons, the HCA is investigating various options for the final disposition of the Crooks' Hollow Dam.

A key issue associated with dam removal is the disposition of sediment deposits in the bottom of the reservoir. These have been naturally deposited over many years. A separate study reports on the nature of these sediments, the extent of sediment contamination, and the issues related to dam removal and proper sediment handling/removal/management.

The issues identified in the dam safety assessment of the Crooks' Hollow dam are summarized in Figure ES-1.

Figure ES-1

Crooks' Hollow Dam

Upstream Face of Dam **Downstream Face of Dam**

Primary Water-Retaining Structure

Main Spillway Dam:	4 sluiceway bays (3 overflow weirs, one stop-log bay) Non-overflow bulkhead at either side of spillway
Drainage Area:	157.9 km^2
Reservoir Area:	less than 1 km^2
Storage:	estimated 67,900 m^3
Original Construction:	1913

Hydrotechnical Issues

Dam Classification (ODSG):	SMALL dam (height) with SMALL storage (reservoir volume) => SMALL DAM
Overall IHP Classification:	LOW
IHP Sunny Day Failure:	LOW
IHP Flood:	LOW
IDF:	1:100-yr to Regulatory Flood (334 m^3/s)
Spillway Capacity:	Adequate

Civil/Structural Issues

General Condition:	Fair to good
Design Basis Earthquake:	1:100 years
Stability:	Inadequate for original design water level Inadequate for flood condition Adequate for reduced operating level

Geotechnical Issues

General Condition: Dam founded on sound bedrock, nonerodible
Some dam seepage observed at right bank

Safety and Operating Issues

- Seasonal placement and removal of timber stop logs is difficult and exposes operators to some hazards.

Condition Recommendations

- Concrete deterioration of many of the structure components is evident, including previous repair concrete. Previous concrete coring reportedly revealed weak concrete within the structure and various leakage horizons.
- To restore the design water level, extensive stabilization works would be necessary, including post-tensioned anchoring. Further investigations would be required to determine if the existing concrete structure could tolerate these additional concentrated loads.
- Various alternatives need to be considered.

Costs of Recommendations: $660,000 to $875,000 (preliminary estimate, See Section 9)

1 Introduction

1.1 Background

The province of Ontario has not yet implemented dam safety regulations. However, as part of their mandate under the *Lakes and Rivers Improvement Act*, the Ontario Ministry of Natural Resources (MNR) has introduced dam safety and flood emergency contingency planning requirements that are based, in part, on the Canadian Dam Association (CDA) guidelines. These have been formalized in the form of a draft document entitled "Ontario Dam Safety Guidelines" (ODSG) (draft dated 1999).

There are approximately 2200 dams in Ontario. Nearly half of these are privately owned, with the remainder owned by Ontario Power Generation (OPG), MNR, and conservation authorities (CAs). The Hamilton Conservation Authority (HCA) owns and operates the Crooks' Hollow. The spillway dam creates a modest reservoir for which once served a small mill operation and subsequently impounded water for golf course irrigation purposes. At present, the reservoir is used only for limited recreation uses. It offers no meaningful flood attenuation, especially since the upstream Christie Dam provides significant reservoir storage.

Hatch Energy was retained by the HCA to undertake an independent dam safety review of the Crooks' Hollow dam. This report presents the results of civil, geotechnical, mechanical, and hydrologic and hydraulic assessments for the dam located on Spencer Creek in the community of Greensville within the City of Hamilton (Figure 1.1).

1.2 Dam Safety Review Objectives

According to the draft ODSG, a dam safety review

> *". . . involves a phased process beginning with the collection and review of existing information, proceeding to detailed inspections and analyses, and culminating with formal documentation."*

With this as a basis, the objectives of a dam safety review include

- assessment of the conditions of the dam and its components
- performance of detailed site inspections
- identification of any necessary repairs and/or continuing maintenance needs
- establishment or the review of an emergency action plan to help minimize adverse impacts
- documentation of the results of the safety assessment so that the information is available in times of need and can be readily updated
- assessment of operational methods and equipment.

Specifically, the safety assessment of a dam comprises a procedural evaluation of the ability of a water-retaining structure to safely withstand all forces that could be expected to act on such a structure during its lifetime. Figure 1.2 displays a comprehensive dam safety assessment process, which is a graphical representation of the Ontario dam safety process. A number of criteria have been developed to allow a systematic evaluation and classification of structures with respect to the potential failure risk it imposes. These criteria incorporate a classification system that addresses the following aspects:

- hazard classification
- flood handling capability evaluation
- geological/geotechnical assessments
- dam break flood evaluation [to evaluate incremental hazard potential (IHP) classification] if required
- structural integrity and stability assessment.

The first step in the process involves a comprehensive site inspection and an evaluation of the incremental hazards that failure of the dam could pose. This evaluation includes an assessment of the potential incremental economic damages, environmental losses and the potential for incremental loss of life in the event of a dam failure.

Based on this assessment, a hazard classification index is determined on the basis of guidelines provided in the draft ODSG as detailed in Table 1.1. Once the IHP is determined, an appropriate inflow design flood (IDF) is selected, using the criteria detailed in Table 1.2, and the design basis earthquake (DBE) is selected

Figure 1.1
Hamilton Conservation Authority
Crooks' Hollow Dam
Location of Crooks' Hollow Dam

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Back of figure 1.1

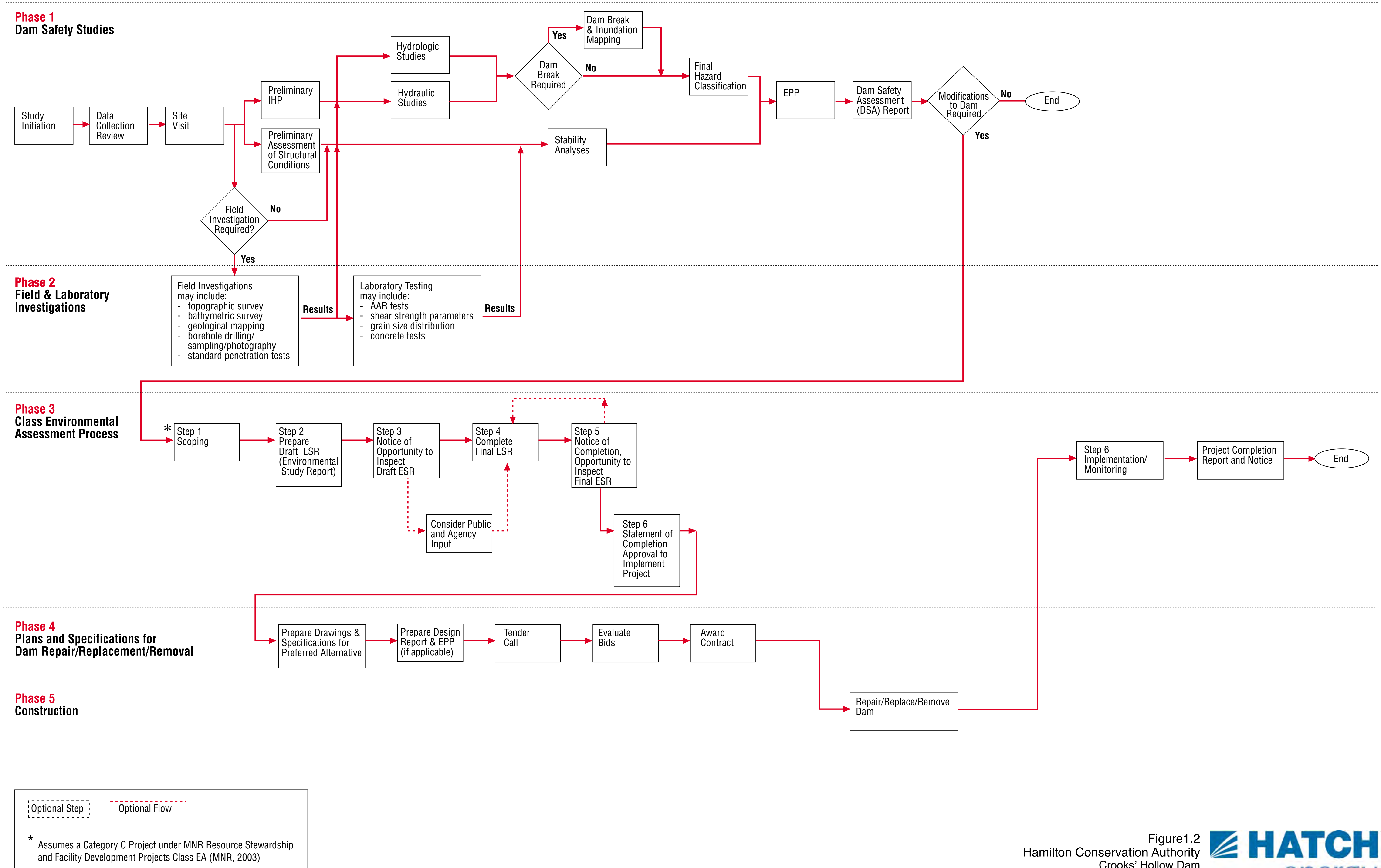


Figure1.2
Hamilton Conservation Authority
Crooks' Hollow Dam
Dam Safety Assessment Evaluation Activities

HATCH™ energy

Back of figure 1.2

using the criteria detailed in Table 1.3. The discharge facilities are then rated on the basis of their capacity to pass the IDF as well as the capability of the structure to be operated reliably during emergency conditions. Water levels are then established for normal and flood (IDF) conditions and an assessment of available freeboard is made for water-retaining structures.

Once loading conditions have been established on the basis of the hydrotechnical analyses and the IHP rating for the dam, the structural integrity of the dam to resist the loads imposed on it during normal conditions, during passage of the IDF and during an earthquake is determined. The results of these assessments, together with an assessment of the overall condition of the structure and issues such as public and workplace safety, are then reviewed and detailed recommendations/ costs for measures to upgrade the structure to satisfy current dam safety requirements are established.

The deliverables for the dam safety evaluation include a comprehensive dam safety assessment (DSA) and reporting on any additional investigations or testing that may be done during the course of the study.

1.3 The Crook's Hollow Dam DSA

The Crooks' Hollow dam has not been recently subjected to a formal dam safety review. Therefore, HCA engaged Hatch Energy to carry out a DSA of the water-retaining structures at the dam in accordance with the draft ODSG (MNR, 1999). This project is approached as an initial evaluation as opposed to a periodic review.

Characteristics of the dam are shown in Table 1.4.

Table 1.1
Hazard Potential Classification for Dams
Selection Criteria
(Source: MNR, Draft ODSG)

Hazard Potential	Loss of Life	Economic and Social Losses	Environmental Losses
Very Low	Potential for LOL: None.	Damage to dam only. Little damage to other property. Estimated losses do not exceed $100,000.	Environmental Consequences: Short-term: Minimal Long-term: None
Low	Potential for LOL: None. The inundation area (the area that could be flooded if the dam fails) is typically undeveloped.	Minimal damage to agriculture, other dams or structures not for human habitation. No damage to residential, commercial, industrial or land to be developed within 20 years. Estimated losses do not exceed $1 million.	No significant loss or deterioration of fish and/or wildlife habitat. Loss of marginal habitat only. Feasibility and/or practicality of restoration or compensating in kind is high, and/or good capability of channel to maintain or restore itself.
Significant	Potential for LOL: None expected. Development within inundation area is predominantly rural or agricultural, or is managed so that the land usage is for transient activities such as with day-use facilities. There must be a reliable element of warning if larger development exists.	Appreciable damage to agricultural operations, other dams or residential, commercial, industrial development, or land to be developed within 20 years. Estimated losses do not exceed $10 million.	Loss or significant deterioration of important fish and/or wildlife habitat. Feasibility and/or practicality of restoration and/or compensating in kind is high, and/or good capability of channel to maintain or restore itself.
High	Potential for LOL: One or more. Development within inundation area typically includes communities, extensive commercial and industrial areas, main highways, public utilities and other infrastructure.	Extensive damage to communities, agricultural operations, other dams and infrastructure. Typically includes destruction of or extensive damage to large residential areas, concentrated commercial and industrial land uses, highways, railways, power lines, pipelines and other utilities. Estimated losses exceed $10 million.	Loss or significant deterioration of critical fish and/or wildlife habitat. Feasibility and/or practicality of restoration and/or compensating in kind is low, and/or poor capability of channel to maintain or restore itself.

* Supporting References: MNR Guidelines for Approval Under the Lakes and River Improvement Act, 1977
MNR Fisheries Section, 1999
US Army Corps of Engineers, Dam Safety Assurance Program, 1995
Dam Structure Assessment Program, Ontario Hydro, 1990

Notes:

1. Consideration should be given to the cascade effect of dam failures in situations where several dams are situated along the same watercourse. If failure of an upstream dam could contribute to failure of a downstream dam(s), the minimum hazard potential classification of the upstream dam should be the same as or greater than the highest downstream hazard potential classification of the downstream dam(s).
2. Economic losses refer to all direct and indirect losses to third parties; they do not include losses to owner, such as loss of the dam, associated facilities and appurtenances, loss of revenue, etc.
3. Estimated losses refer to incremental losses resulting from failure of the dam or misoperation of the dam and appurtenant facilities.
4. For Hazard Potential Classification and Safety Criteria for tailings dams, refer to "Guidelines for Proponents, Rehabilitation of Mines", issued by Ontario Ministry of Northern Development and Mines, 1995.

<table>
<tr><th colspan="7">Table 1.2
Minimum Inflow Design Floods for Dams
(Source: MNR, Draft ODSG)</th></tr>
<tr><th></th><th colspan="6">Size of Dam and Inflow Design Floods</th></tr>
<tr><th>Hazard</th><th colspan="2">Small</th><th colspan="2">Medium</th><th colspan="2">Large</th></tr>
<tr><th>Potential</th><th>Height
< 7.5 m</th><th>Storage
< 100 x 10^3 m^3</th><th>Height
7.5 to 15 m</th><th>Storage
100 x 10^3 to 1000 x 10^3 m^3</th><th>Height
> 15 m</th><th>Storage
> 1000 x 10^3 m^3</th></tr>
<tr><td>Very Low</td><td colspan="2">25-yr flood
to
50-yr flood</td><td colspan="2">50-yr flood
to
100-yr flood</td><td colspan="2">100-yr flood
to
RF</td></tr>
<tr><td>Low</td><td colspan="2">25-yr flood
to
100-yr flood</td><td colspan="2">100-yr flood
to
RF</td><td colspan="2">RF
to
PMF</td></tr>
<tr><td rowspan="2">Significant</td><td colspan="2" rowspan="2">100-yr flood
to
RF</td><td colspan="2" rowspan="2">RF
to
PMF</td><td colspan="2">PMF</td></tr>
<tr><td colspan="2">Policy for existing dams is under consideration</td></tr>
<tr><td rowspan="2">High</td><td colspan="2">RF
to
PMF</td><td colspan="2">PMF</td><td colspan="2">PMF</td></tr>
<tr><td colspan="6">Policy for existing dams is under consideration</td></tr>
</table>

Legend: RF – regulatory flood
PMF – probable maximum flood

Notes:

1. For Minimum Inflow Design Floods for Mine Tailings dams, refer to "Guidelines for Proponents, Rehabilitation of Mines", issued by Ontario Ministry of Northern Development and Mines, 1995.

2. Existing dams refer to those structures built prior to 1978.

Table 1.3
Criteria for Design Earthquakes

Hazard Potential Classification [a]	MDE	
	Deterministically Derived	Probabilistically Derived (Annual Exceedance Probability)
High	50% to 100% MCE [b] [c] [d]	1:1000 to 1:10 000 [d]
Significant	– [e]	1:100 to 1:1000 [e]

(a) Hazard potential classification established separately for each dam.

(b) For a recognized fault or geographically defined tectonic province, the maximum credible earthquake (MCE) is the largest reasonably conceivable earthquake that appears possible. For a dam site, MCE ground motions are the most severe ground motions capable of being produced at the site under the presently known or interpreted tectonic framework. Use upper values in the range, where loss of life and property damage due to failure would be unacceptably high.

(c) An appropriate level of conservatism shall be applied to the factor of safety calculated from these loads, to reduce the risks of dam failure to tolerable values. Thus, the probability of dam failure could be much lower than the probability of extreme event loading.

(d) In the high hazard potential category, the MDE is based on the consequences of failure. Design earthquake approaching MCE would be required where loss of life and property damage due to failure would be unacceptably high.

(e) If a structure in the significant hazard potential category cannot withstand the minimum criteria, the level of upgrading may be determined by economic risk analysis, with consideration of environmental and social impacts.

Table 1.4
Description of the Dam

Name of Dam	Access	Description			
		Reservoir Area (km^2)	Dam Height (m)	Length (m)	Spillway/ Discharge Facility
Crooks Hollow	Public Road – Crooks Hollow Road	Less than 1 km^2	6.1	36.6	3 free overflow bays and 1 stop log sluiceway bay

Note: See also Figure ES-1.

Details of the analyses and assessments performed for this dam are described in the following main sections:

- Executive Summary

- Section 1 – introduction and explanation of approach

- Section 2 – description and history of the Crooks' Hollow dam

- Section 3 – details of the initial data review including the types of documents reviewed

- Section 4 – details of the comprehensive site inspections including civil, structural, geotechnical and hydrotechnical observations

- Section 5 – comments on the need for any additional Phase 2 site investigations which might have arisen or of the need to fill data gaps identified during the initial site inspections and evaluations

- Section 6 – details of the hydrological/hydraulic assessments. The section includes the following main topics:

  – IHP and IDF selection
  – flood routing and freeboard analysis.

- Section 7 – details of the civil/structural stability assessments are provided. These include a description of the load cases evaluated, the rationale for the selection of shear strength parameters and details of any measures that might be needed to upgrade the dam to satisfy current dam safety requirements.

- Section 8 – provides a summary of a review of the existing dam documentation (not included in the current scope of work).

- Section 9 – concludes with a review of alternative remedial measures for the dam.

Photographs of the damsite and the dam itself are contained in Appendix A of this report.

2 Crooks' Hollow Dam

The Crooks' Hollow Dam is owned and operated by the HCA and is located on Spencer Creek in the community of Greensville within the City of Hamilton. The dam was originally constructed in 1913 to supply potable water to the community of Dundas. Years later, this use ceased after a municipal supply of water was established for the village. Between 1959 and 2001, the DVGCC used the reservoir as a source for irrigation water. The reservoir and surrounding lands currently provide recreational opportunities that include hiking, fishing and limited boating.

The dam is a concrete structure approximately 6.1 m high and 36.6 m long with four stop-log spillways. The condition of the dam is considered to be fair. Noted deficiencies include poor condition of the concrete surface on the below-water upstream side and on portions of the downstream spillway end wall, fill settlement associated with the north abutment, dislodgement of the downstream spillway wall and seepage (Acres, 2005). A previous 1993 study concluded that the dam was not stable when operated at its normal operating water level and during flood conditions (Peto MacCallum Limited, 1993). The reservoir's summer operating level was intentionally lowered by almost 2 m to help ensure the safety of the dam. This remains the normal operating regime for the reservoir. To confirm the current (2005) condition and stability of the dam, the HCA has initiated a dam safety review and stability assessment study of the dam.

Given the dam's age and deficiencies, it is apparent that the Crooks' Hollow Dam requires corrective rehabilitation to ensure its safe operation under major storm events or it should be decommissioned and either removed or modified into an overflow weir. For these reasons, the HCA is investigating various options for the final disposition of the Crooks' Hollow Dam.

HCA retained Hatch Energy to review safety of the dam against the Ontario Ministry of Natural Resources Dam Safety Guidelines (ODSG, 1999 Draft) and to complete the environmental planning and documentation, including public and agency consultation to meet the requirements of Conservation Ontario's *Class Environmental Assessment* (Class EA) *for Remedial Flood and Erosion.*

3 Initial Data Collection and Review

As a first step in the assessment process, a kick-off meeting was held and available information was collected at HCA offices on May 19, 2005 prior to the site inspections. This information was later reviewed in detail at Hatch Energy Niagara Falls office. As part of this process, the following information was collected from various sources for examination:

- drawings of the dam (limited)
- reports – see Bibliography at the end of this report
- watershed maps showing damsite and drainage areas
- Ontario Geological Survey maps and documents
- hydrological data from selected stations
- data from selected streamflow gauging stations from Water Survey of Canada (WSC)
- selected topographic maps (1:50 000-scale)
- water level data.

The results of this review provided a general understanding of the characteristics of the site and the operational issues and the types of structural problems that might be expected on the basis of the prevailing topographic, climatic and geological conditions.

The following are some problems which may be expected to occur at dams of this type:

- leakage at defects in the concrete, at the concrete/foundation contact or through open bedrock discontinuities
- typical concrete deterioration problems
- sliding stability problems associated with winter ice loadings
- inadequate spill capacity
- public and operational safety issues (signage, fall arrest systems, handrail condition, etc).

During the site inspection, the potential for these types of problems was specifically addressed in addition to other issues that became apparent during the course of the site visit.

4 Comprehensive Site Inspections

4.1 Introduction

A site evaluation of the Crooks' Hollow dam was made by Hatch Energy's civil, hydrotechnical and geotechnical engineers. The results of these inspections are presented in the following sections, on digital photographs (Appendix A) and on Forms B1 and B2 (Appendix B). The work was generally carried out in accordance with MNR, ODSG (Draft), August 1999. A dam operator's questionnaire is presented in Appendix C.

4.2 Antecedent Weather Conditions

Seepage observations noted during site inspections at water-retaining structures may be influenced by weather conditions which occur at the time of the inspection and during the preceding period. During the May 19, 2005 site inspection, the weather was sunny and 16ºC. There had been no precipitation within the preceding 24 hours.

4.3 Record of Observations

4.3.1 General Description

The Crooks' Hollow Dam is owned and operated by the HCA. The dam was originally constructed in 1913 to supply potable water to the community of Dundas. Years later, this use ceased after a municipal supply of water was established for the village. Between 1959 and 2001, the DVGCC used the reservoir as a source for irrigation water. The reservoir and surrounding lands currently provide recreational opportunities that include hiking, fishing and limited boating. The reservoir provides no meaningful flood attenuation.

4.3.2 Hydrotechnical Aspects

The Crooks' Hollow Dam is a concrete gravity structure with four openings as spillway. Spillway number 2 is controlled by stop logs, but the other openings are overflow weirs. The widths of the spillways vary from 3.7 m to 4.3 m. The dam is 36.6 m long and 6.1 m high. The storage behind the dam is estimated as 67 900 m^3. Crooks' Hollow dam has a drainage area of

157.9 km^2. During regional storm conditions the dam will discharge a peak flow of 453 m^3/s and will be overtopped. Therefore the dam is classified as SMALL dam (<7.5 m) with respect to dam height, with SMALL storage (<1,000,000 m^3) with respect to storage.

Downstream of the dam, the channel bed is rough with irregular channel shapes. The channel is narrow at the tailrace and expanded significantly farther downstream with a varied valley width. The slope of the downstream channel is flat and well vegetated and hence the roughness is high. The flow velocity downstream of the dam is slow with low erosion potential under normal conditions.

There are permanent residents on both banks downstream of the dam. But the houses immediately downstream of the dam are all located on high ground. Approximately 2 km downstream, there are two houses, one on the north bank and one on the south bank have lower elevations, the failure of the Crooks' Hollow dam might lead to some flooding for these houses (by visual observations and should be verified by a dam break analysis). Farther downstream, the creek runs down a very steep reach. At the end of the steep reach, the channel has a sharp bend. The flow velocity at the end of the reach is very high. A school is located right at the bend. The flood wave induced by the failure of the Crooks' Hollow dam might lead to overflow in this area and lead to flooding of the school. However, due to the small storage of the reservoir, the flooding in the school area may not be significant (this must be confirmed by a dam break assessment). In conclusion, there might be flooding problem at the downstream reach of the Crooks' Hollow dam but loss of life is not expected due to the small storage.

The Crooks' Hollow dam is located downstream of the Christie Dam, which has much larger storage. If the dam fails due to the failure of the Christie Dam, the consequence of the dam failure would be much more significant. The cascade dam failure event must be investigated in detail.

Given the above conditions, the dam is preliminarily assigned a SIGNIFICANT IHP rating for both the sunny day and flood conditions.

4.3.3 Civil/Structural Aspects

History

Condition assessments of the dam carried out in 1968 and 1976 by William L. Sears identified enough concern about the integrity/stability of the dam that the normal operating level was lowered to reduce the loads on the structure during major storm events. Subsequent assessments in 1993 (Peto MacCallum Ltd.) identified the poor condition of the concrete and notably, the spillway piers which exhibited severe concrete delamination and cracking. The dam was considered to be stable under current operating conditions (see above) and for short-term increases in water levels up to 1.5 m above spillway Nos. 1, 3 and 4 (elevation +/-218.82 m) in the event of a major storm event. However, the dam was not considered to be able to withstand the force of a major storm event if the normal operating water level was maintained at its original design operating level of 1.8 m above the top of spillways Nos. 1, 3 and 4. As a result, to ensure the integrity of the dam, the HCA modified the operating procedure by reducing the normal (summer) operating level to the sill elevation of spillway Nos. 1, 3, and 4 (elevation +/-217.32 m).

Various repairs to the dam have been completed since the 1970's including concrete repairs in 1977, shotcrete resurfacing in 1987-88, installation of an upstream membrane in 1994 and repairs to the catwalk decking in 1995, no major rehabilitations to the structure have been made to address the stability deficiency. HCA has continued to operate the dam at the lowered normal (summer) level since 1993.

Current Observations

The dam was inspected when the reservoir was drawn down to its winter water level. The concrete structures were generally observed to be in fair to poor condition. The repairs made to the dam over the years are showing signs of deterioration. Photos 1 and 2 show the upstream and downstream faces respectively. In particular

- the shotcrete resurfacing has fine cracking over much of the area (Photo 3). Some of these exhibit efflorescence which is not particularly worrisome but is indicative of moisture transmission. Some cracks have been repaired (Photo 4). In various locations, the shotcrete appears to have become delaminated from the underlying concrete.

- the upstream elastomeric coating has performed well and is believed to have been instrumental in extending the life of the structure. It however, is failing (Photos 5 and 6) and upon removal reveals weak and deteriorating concrete (Photo 7). Previous reports of concrete coring investigations had indicated weak internal concrete and these observations support those conclusions.

- there has been significant settlement of the soil element of the left abutment wingwall as shown on Photo 8. Wooden and concrete steps appear to have settled approximately 150 mm.

- the downstream left training wall, although founded on rock is undercut (Photos 9 and 10) and misaligned (Photo 11).

- the valve chamber located to the left of the spillway (Photo 12) is full of water, making any inspection or maintenance of the low level control valve impossible.

- the soil at the downstream face of the right abutment has eroded away, exposing original concrete that was not protected with remedial shotcrete (Photo 13).

- the toe of the spillway is undercut (Photos 14 and 15).

- the stop-log handling system is a manual operation and somewhat awkward. Operators indicated that the hand winch capacity is not rated and that this could be an OH&S concern to workers (Photos 16 and 17).

Previous reports indicated that the interior concrete of the dam is weak and the current on-site observations support this. It is considered that any remedial efforts to extend the life of the dam and restore its capability to safely withstand its original design water level would have to be very comprehensive. Such an effort would involve the use of post-tensioned steel bars or strands to anchor the dam to the foundation bedrock and to stabilize the dam. There is significant doubt as to whether the dam could withstand these concentrated loads.

4.3.4 Geotechnical Aspects

General Geology

The dam is located on Spencer Creek near Greensville (Dundas) just above the Niagara Escarpment. The creek discharges into the Hamilton Harbour embayment. The reservoir and dam are located in rolling, hilly and forested terrain with a local relief of the order of 40 m. Physiography in the area is the result of the Wisconsin glaciation. Thin deposits of overburden overlie flat-lying sedimentary rocks of Paleozoic age, such as limestone, shale and sandstone. The overburden comprises fine-grained silts and sands of glacio-lacustrine origin, overlying Halton silt and clay glacial till.

Results of Inspection

The site is in a small narrow valley. Abutments slopes are overburden-covered and are gently sloped on the left (north) and moderate to steep on the right. Bedrock is exposed in the riverbanks.

The inspection indicated that the structure comprises a four-bay, concrete gravity spillway flanked on both sides by apparently open-ended concrete 'boxes' that are earth filled. The spillway is founded on horizontally and thin-bedded limestone bedrock, which was observed in the reservoir (low water level at the time of inspection) and in the tailrace. The concrete box units are assumed to be founded on rock. The head across the structure at the time of inspection was approximately 3 m.

Just upstream of the dam on the left bank, groundwater was observed issuing from a spring in the bedrock. On the same bank, a tributary stream flowed into the tailrace just downstream from the dam. The net result was that the groundwater level in the left abutment area was high, as was evident next to the left training wall extending downstream from the dam. At the downstream extremity, this wall was observed to have settled about 0.15 m and tilted toward the tailrace. While most of this wall was constructed on bedrock and was upright, the downstream section may have been founded on overburden or soft rock.

A pipe leaking water into the tailrace was seen in the left training wall at the foot of the first spillway bay. In addition, a small amount of groundwater was observed spilling over the left training wall and down into the tailrace. This water appears to be leaking from an adjacent valve access chamber located next to the first spillway bay. The valve is on a gated waterline passing through the dam. The valve chamber was found to be flooded. The leakage noted above on the downstream side, including that through the pipe, may have been seeping through leaky construction joints in the concrete dam itself.

On the right bank, minor seepage was noted in the bedrock face just downstream from the dam. This leakage is dam related, i.e., it is reservoir leakage passing through the bedrock. No signs of general groundwater levels were noted in the right abutment.

On the left bank, in the concrete box referred to above, up to 0.15 m of settlement had occurred in the earth fill, as suggested by the offset concrete stairs next to the valve access chamber. Settlement of fill was also noted along the upstream wall. This settlement may be the result of poor fill compaction during construction, particularly along the upstream wall, which is inclined outward. Some of the fill may also be piping into the valve access gallery through leaky construction joints, causing settlement. No offsets in the upstream concrete wall were observed.

In several areas, such as the piers and downstream face, the old concrete was apparently refaced with shotcrete (smoothened) some years ago. No significant, open cracks were observed in the shotcrete, indicating a general absence of structural movement.

The shotcrete on the upstream face of the upstream concrete wall on the left bank was observed to be 'drummy' below the reservoir water line, indicating no bond (Photos 6 and 7). Further, the shotcrete was locally slumped and cracked. On the right bank downstream side, shotcrete on the spillway end wall was observed to be separated from the bedrock.

Dam investigations were done by Peto MacCallum in 1993. This included concrete cores and one borehole drilled through the dam to bedrock. A piezometer was apparently installed. This borehole could not be located, and piezometer data is presently unavailable. This information would be useful for stability analyses.

Stability of the structure against sliding depends on the angle of friction for the foundation bedrock. In the absence of test data or further bedrock information, an angle of 35° is estimated, using the Barton equation. This assumes a roughness factor (JRC) of 2, a foundation loading of 0.1 MPa and a bedrock strength of 40 MPa. Details of the Barton approach for estimating foundation friction properties are included in Appendix E.

4.3.5 Mechanical Aspects

Equipment at Sluices

The main sluice is equipped with manual chain winches at each pier for placing and removing timber stop logs seasonally. The logs are not used to regulate the reservoir water level and are either all in or all out.

Placing and removing the stop logs is a labor intensive undertaking and this is generally done only in low flow times. Six personnel are required for these log movements and it is reported to take up to 1 day to complete the operation. In the off-season, the stop logs are stored off site. Moving the logs on and off the dam is also labour intensive.

The existing winched appear to be serviceable and in good condition. The load rating of the equipment is unknown and this is a potential issue for workers.

The stop-log handling system is not routinely tested, although it is in effect exercised twice per year. Experience staff is necessary to undertake this task safely.

There is no electrical power at the dam.

Potential for Debris Blockage of Discharge Facilities

The accumulation of debris at the sluices has reportedly been an occasional problem in the spring, usually associated with a flood event.

Availability of Adequate Lighting for Night-Time Operations

Portable lighting can be brought to the site for emergencies.

5 Phase 2 Site Investigations

No Phase 2 investigations were carried out as part of this assessment. However, a number of additional site visits were made to assess the sediment in the bottom of the reservoir and various material samples were retrieved and depth soundings were taken in support of the environmental assessment work.

If extensive dam repairs are proposed, updated concrete coring will be recommended. This will not be required if the dam removal alternative is selected.

6 Hydrotechnical Assessment

6.1 Preliminary Incremental Hazard Potential (IHP) and Inflow Design Flood (IDF)

6.1.1 Background

The hazard classification index (HCI) of a dam is determined by the degree of exposure of development located on the river downstream from the dam, in terms of potential flood damage and LOL in case of a dam failure. Based on this assessment, an HCI is determined and a corresponding IDF is assigned. Table 1.1, taken from the ODSG (MNR, 1999), lists the hazard classifications for dams. The HCI for a dam, also referred to as the IHP or incremental consequence category (ICC), reflects the hazard potential associated with the failure of the dam, and it does not reflect the severity/magnitude of a particular flood event itself. The IDF is selected based on the HCI and forms the basis for the dam safety assessment. Table 1.2, taken from the ODSG (MNR, 1999), stipulates a range on IDF magnitudes, depending on the hazard potential the dam poses.

A spillway is rated on the basis of its capacity as well as its capability to respond to emergency flood conditions on the river. These ratings are compared to the IHP and IDF requirement.

This section presents the results of a preliminary evaluation of the IHP for the Crooks' Hollow dam and the corresponding IDF that was selected, and complements other information dealing with the specifics of the damsite's facilities, based on data gathered during the site visit, plus other available information obtained from the HCA.

Confirmation of the IHP designation and the IDF for the dam needs information about incremental flooding due to dam break. However, dam break study simulation results for the Crooks' Hollow dam are not available presently and hence the IHP rating for this dam is not confirmed at this time. Except for calculations of spillway capacity and safety factors for stability of a structure, the dam safety assessment process allows for some qualitative interpretation where a strictly scientific approach is not possible. Generally, however, the guidelines are sufficiently well-defined to permit determination of dam safety conditions within practical and accurate limits.

6.1.2 Preliminary IHP and IDF

The consequences of a dam failure are assessed in terms of the incremental hazard posed by a dam structure, based on guidelines and procedures given in the draft ODSG (MNR, 1999). The IHP can be defined as the potential for increase in LOL, property damage and disruption of social and economic activities, and environmental impacts, caused by failure of the dam structure above that which would have occurred without failure of the dam. The hazard classification is generally determined by simulating dam break floods and assessing the effects of the resultant downstream flood inundation.

Initially, for this study, preliminary hazard potential at the damsite was selected on the basis of available information (e.g., characteristics of the dam, reservoir, watershed, discharge facilities, downstream development, recreational activities, historical flooding, etc). This preliminary hazard potential was then used to determine the IDF for the site.

The Crooks' Hollow dam is approximately 6.1 m high and impounds 67,900 m^3 of water in the head pond. This places the dam in the category of SMALL dam with SMALL storage.

There are permanent residents on both banks downstream of the dam. But the houses immediately downstream of the dam are all located on high ground. Approximately 2 km downstream, there are two houses, one on the north bank and one on the south bank. These appear to have lower elevations and the failure of the Crooks' Hollow dam might lead to some flooding for these houses (by visual observations). This should be verified by a dam break analysis. Farther downstream, the creek run downs a very steep reach. At the end of the steep reach, the channel has a sharp bend to the right (looking downstream). The flow velocity at the end of the reach is very high. A school is located right at the bend. The flood wave induced by the failure of the Crooks' Hollow dam might lead to overflow in this area and lead to flooding of the school. However, due to the small storage of the reservoir, flooding in the school area may not be significant (this must be confirmed by a dam break assessment). In conclusion, there might be a flooding problem at the downstream reach of the Crooks' Hollow dam but loss of life is not expected due to the small amount of reservoir storage.

The Crooks' Hollow dam is located downstream of the Christie Dam, which has much larger storage. If the dam fails due to the failure of the Christie Dam, the consequence of the dam failure would be much more significant. The cascade dam failure event should be investigated in detail (but this task would be associated with the Christie Dam and this issue may already have been assessed).

Given the above conditions, the dam is preliminarily assigned a SIGNIFICANT IHP rating for both the sunny day and flood conditions.

According to the draft dam safety guidelines (MNR, 1999), the IDF for this dam should be the 1:100 yr to the regulatory flood (RF).

6.2 Estimates of Design Floods

6.2.1 Hydrologic Analysis

The purpose of the hydrologic analyses was to estimate peak flood flows and hydrographs for the 2 yr, 5 yr, 10 yr, 25 yr, 50 yr, 100 yr and regulatory for the study area. The regulatory flood for the study basin is the historical Hurricane Hazel event. The design hydrographs were used in the flood routing studies and subsequent dam safety assessment analysis that are described in the following sections.

6.2.2 Hydrologic Model Background

The peak flows at the damsite were estimated through deterministic modeling of watershed runoff on an event basis. A hydrologic model of the Spencer Creek/Coots Paradise basin, developed for HCA using the hydrologic simulation program QUALHYMO was made available for this study by HCA.

In the original hydrologic model the Crooks' Hollow dam was not modeled as a separate subwatershed. For the purpose of obtaining the water levels corresponding to the various design events under consideration, the model was modified to include the Crooks' Hollow Dam as separate catchment and a reservoir routing component was added into the model. The local drainage area between the Christie Dam and Crooks' Hollow Dam is estimated to be approximately 0.594 km^2.

6.2.3 General Description of QUALHYMO Model

The Spencer Creek watershed model is designed to simulate the surface runoff response of the basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process over the entire watershed, or within a portion of the basin, commonly referred to as a sub-basin. A component may represent a surface runoff entity, a stream channel, or a reservoir. Representation of a component requires a set of parameters that specify the particular characteristics of the component and mathematical relations, which describe the physical process. The modeling process computes streamflow hydrographs at desired locations in the river basin. QUALHYMO is a simple continuous water quantity (and quality) simulation model which was developed in 1983 at the University of Ottawa for the analysis of stormwater detention ponds. The model can be used as a general tool for simulating rainfall runoff processes.

Meteorological input to the QUALHYMO model consists of hourly precipitation and temperature records. During the winter period, precipitation is categorized as liquid or snow depending on air temperature relative to a specified threshold value at or near 0^oC. Snowpack accumulation and ablation is estimated by a temperature index equation.

The QUALHYMO model for the Spencer Creek Basin was developed from readily available source of watershed information which provided the most recent documentation of land use, soils and topographic features. The details of the model can be found in the Technical Report of the Spencer Creek Watershed Hydrology Study (MacLaren Plansearch, 1990).

6.2.4 Model Calibration

Extensive hydrological modeling has been undertaken for this area, i.e., the Spencer Creek watershed hydrology study (MacLaren Plansearch, 1990). Model calibrations were performed in the study for the 1973 to 1979 period (which contained the high flow events of interest). For this reason, the hydrological parameters were extracted from the previous models that cover the current study area without modifications.

The modeling of the Crooks' Hollow Dam requires data sets representing the characteristics of the dam including the stage-discharge and stage-storage curves. The stage-discharge curve for the spillway was developed using standard weir equations.

6.2.5 Storm Event Data

The design storms of 1:2-yr, 1:5-yr, 1:10-yr, 1:25-yr, 1:50-yr and 1:100-yr events have been developed for the Spencer Creek watershed as part of the hydrological study and were used in the OUALHYMO model. The QUALHYMO models for the design events were provided by HCA. The regulatory storm (Hazel) is a well documented historical event and the published data for this event were used.

6.2.6 Storm Temporal Distributions

Three important parameters – storm volume or depth, duration, and temporal distribution – affect the shape and peak value of the resulting runoff hydrograph from the QUALHYMO model. The storm temporal distributions used in the original QUALHYMO model remain unchanged.

6.2.7 Regional Storm

The regional storm for the study area is the Hurricane Hazel storm based on the Floodplain Management Guidelines (MNR, 1986). This 48-hr design storm was recorded from a rainfall gauge located at Snelgrove just north of Brampton, Ontario.

During a 48-hr period on October 15 and 16, 1954, the remnants of Hurricane Hazel dumped over 285 mm of rain in the Toronto area. The total rainfall volume in the first 36 hours is 73 mm. The heaviest rains fell on the watershed during the final 12 hours of the storm when 212 mm of rain was recorded on saturated ground surface. Toward the end of the storm, 53 mm of rain fell in 1 hour while 91 mm was recorded during a 2-hr period.

6.2.8 Event Modeling – Crooks' Hollow Dam

The QUALHYMO model was used to evaluate the Crooks' Hollow Dam basin discharge behavior under a wide range of precipitation events, with return periods of 2, 5, 10, 25, 50 and 100 years. The Hurricane Hazel storm was also modeled. The results are summarized in Table 6.1.

6.3 Hydraulic Analyses

6.3.1 Rating Curve of the Dam

In order to evaluate the flood handling capacity of the dam, flow rating curves were developed for the Crooks' Hollow Dam based on available drawing. The dam has 4 sluice bays and 1 sluiceway (bay 2) is controlled by stop logs. The sill elevation of the sluice number 2 is 214.15 m which has seven 0.30-m (1-ft) high stop logs. The top elevation of the stop logs is 216.28 m (when seven stop logs are installed). The deck elevation is 219.33 m. The full supply level is 216.28 m. For sluiceways 1, 3 and 4, the sill elevation is 217.32 m. Figure 6.1 presents the flow rating curves for bay number 2. Figure 6.2 shows the flow rating curve of bays 1, 3 and 4. Figure 6.3 presents the stage-storage relationship of the reservoir.

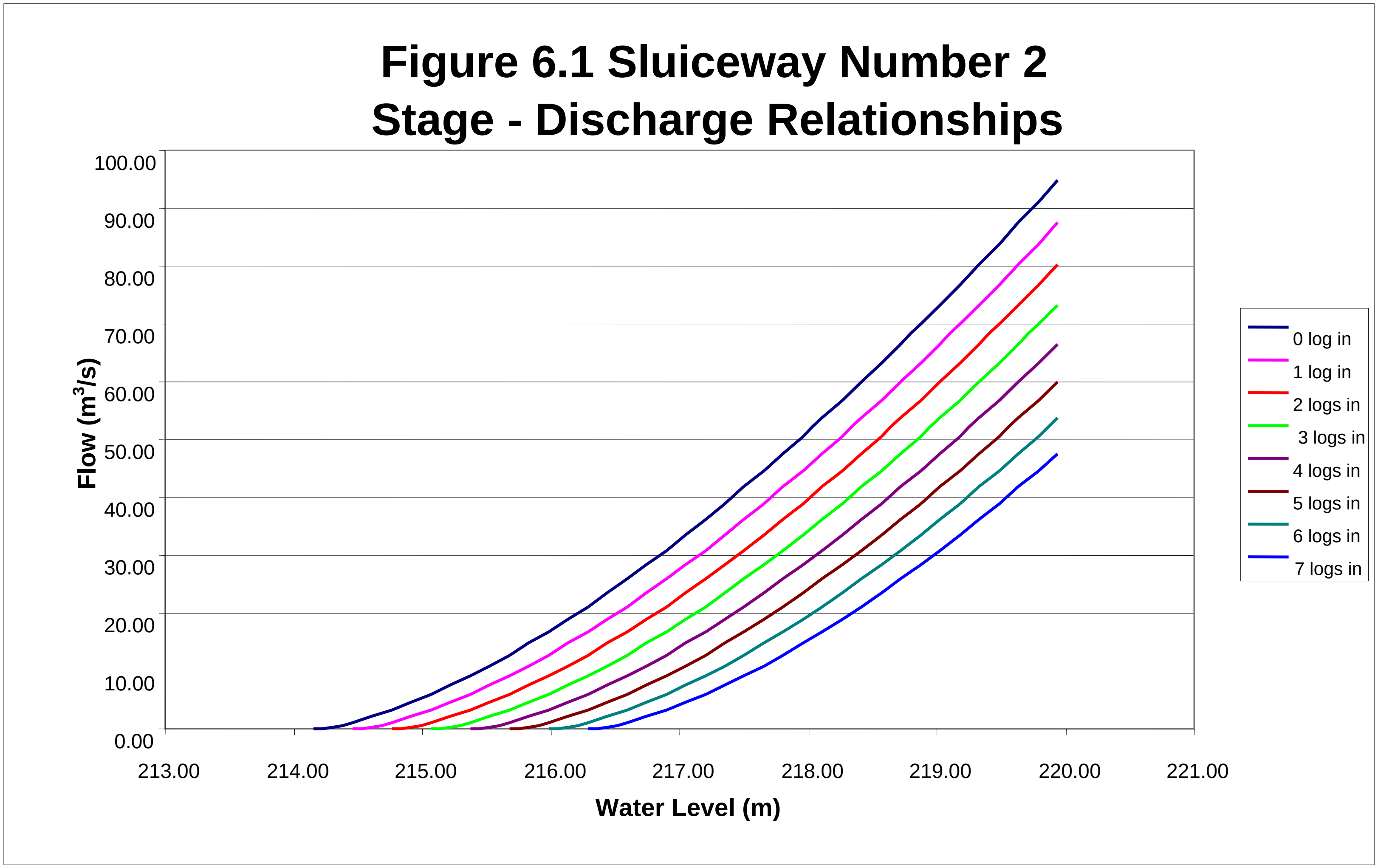
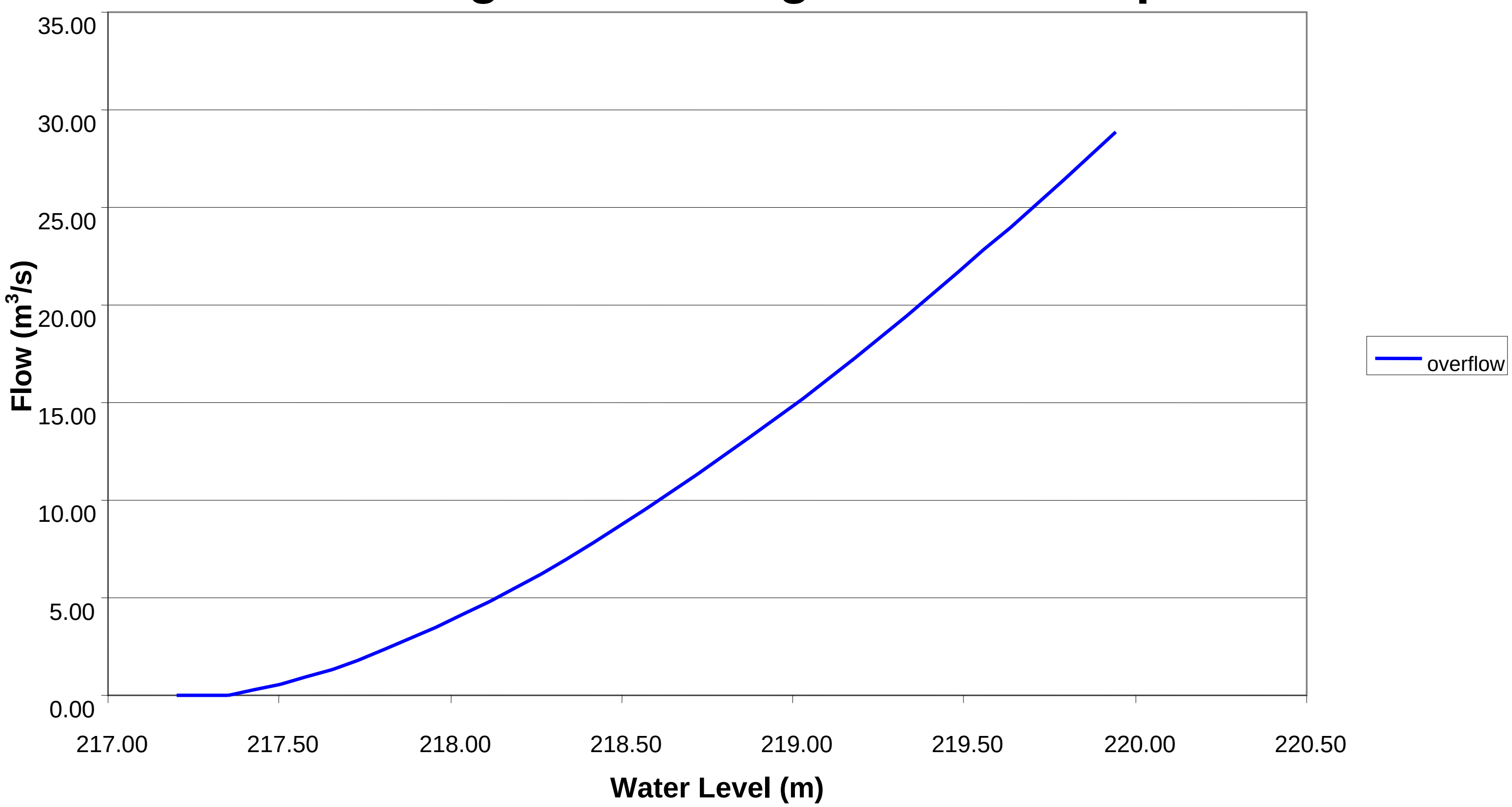


Figure 6.1 Sluiceway Number 2
Stage - Discharge Relationships
100.00
90.00
80.00
70.00
60.00
50.00
40.00
30.00
20.00
10.00
0.00
Flow (m^3/s)
213.00
214.00
215.00
216.00
217.00
218.00
219.00
220.00
221.00
Water Level (m)
0 log in
1 log in
2 logs in
3 logs in
4 logs in
5 logs in
6 logs in
7 logs in

Figure 6.2 Sluiceway Number 1, 3 and 4 Stage – Discharge Relationship



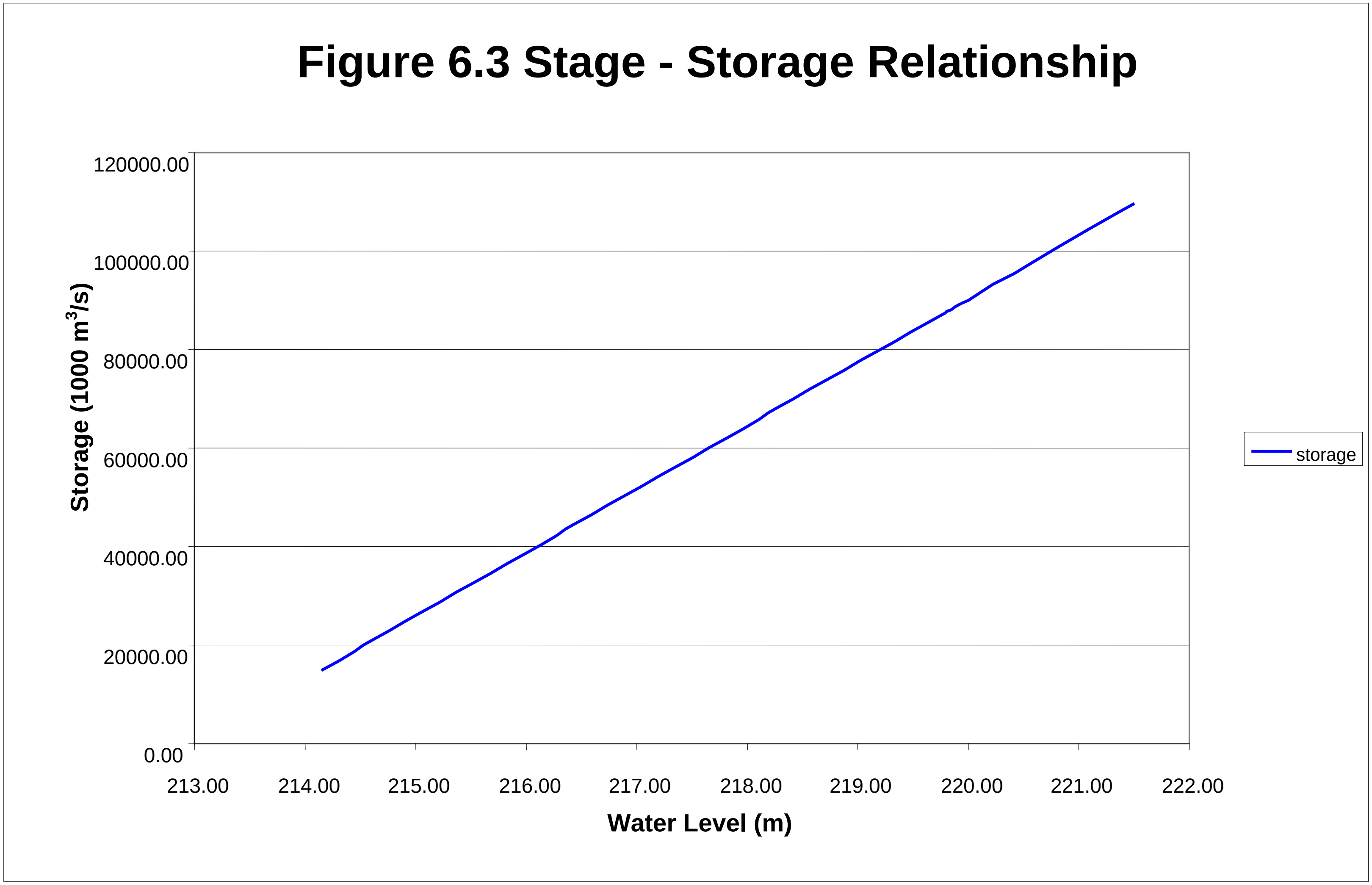


Figure 6.3 Stage - Storage Relationship
120000.00
100000.00
80000.00
60000.00
40000.00
20000.00
0.00
Storage (1000 m3/s)
213.00
214.00
215.00
216.00
217.00
218.00
219.00
220.00
221.00
222.00
Water Level (m)
storage

6.3.2 Water Surface Analysis

The QUALHYMO model was used to analyze water surface elevation for the design flood conditions.

The stage-storage and stage-discharge rating curves of the dam were used in the simulations. It was assumed that the starting water level in summer is 216.28 m and the starting water level in winter is 215.14 m.

6.3.3 Results of Water Level Simulation

The water levels corresponding to the peak flood flows were estimated by using the QUALHYMO model.

Table 6.1 summarizes the results of peak water level simulations.

Table 6.1
Summary of Design Peak Flows and Water Levels

Starting Water Level (m)	Return Period (yr)	Peak Inflow (m3/s)	Peak Outflow (m3/s)	Peak Water Level (m)
215.06 (Winter)	2	15.89	15.88	216.61
	5	21.08	21.07	217.13
	10	24.80	24.79	217.41
	20	28.00	27.99	217.66
	50	31.60	31.58	217.89
	100	34.27	34.26	217.99
216.28 (Summer)	2	15.89	15.89	217.74
	5	21.08	21.07	217.85
	10	24.80	24.79	217.93
	20	28.00	28.00	218.04
	50	31.60	31.52	218.13
	100	34.27	34.27	218.20
	Hazel	333.37	333.37	221.50

Note: The deck elevation is 219.33 m

Two cases were examined in the simulations, representing the two stop-log settings with regard to the operations. One simulation assumed that four stop logs are removed from the dam and the starting water level is 215.14 m. This is the winter stop-log setting. In this case, the water levels will be low as shown in Table 6.1.

The other simulation assumed that seven stop logs are installed in bay number 2, which is the summer stop-log setting condition. In this case, the starting water level would be higher and the hazel event will overtop the deck and left wing wall. However, for the IDF of 1:100-yr (see Section 6.5), the spillway capacity is adequate and the spillway deck will not be overtopped. This means that the dam has sufficient capacity to pass the regulatory flood which is the inflow design flood for this dam.

6.4 Freeboard Analysis

Freeboard was examined by calculating wind setup, wave height and wave run-up for FSL and IDF conditions. Wind setup was computed using the procedure outlined in the US Department of the Interior Freeboard Criteria (USBR, 1981). Design wave heights were determined using the US Army Corps of Engineers Shore Protection Manual (SPM) (US Army, 1984). To obtain conservative estimates of freeboard requirements, the effective fetch in the reservoir was calculated with the primary wind direction aligned with the longest fetch length or radial in the vicinity of the dam structure. Since the reservoir is relatively small, no corrections were made from overland to over water wind speeds.

The wind data at Hamilton Airport was used to estimate both the 100-yr and 1000-yr wind speeds. Because of the limited fetch, the wave height will not be restricted by wind duration. The wind durations at either 35 m/s (100-yr) or 43 m/s (1000 yr) will both be long enough to establish steady-state wind/wave conditions in the head pond.

As shown in Table 6.2, the resulting calculated wind setups were negligible in both cases. The significant wave height was calculated as a function of effective fetch and wind speed. The design wave was taken as the average of the highest 10% of waves (H_{10}), and was determined from the significant wave height from the SPM ($H_{10} \approx 1.27$ Hs). Nonbreaking wave forces against vertical wall structures were also computed using the method described in the SPM. The resulting wave heights and wave run-ups are summarized in Table 6.2 for the 100-yr and 1000-yr wind speeds.

Minimum freeboard requirements were assessed in accordance with MNR guidelines (MNR, 1999).

- Under maximum normal head-pond water levels and 1000-yr wind condition, normal freeboard requirements at the damsite are given in Table 6.2.

- Under peak IDF water level conditions, minimum freeboard requirements at the damsite have been conservatively established for specified 100-yr wind conditions. Minimum freeboard requirements are given in Table 6.2.

These results show that, during passage of the IDF, the dams would have adequate freeboard. During normal operation, the dam has 2.69 m of freeboard, which is sufficient.

6.5 Confirmation of IHP and IDF

Confirmation of the IHP is fundamentally determined by dam break modeling. These assessments provide an estimate of potential downstream inundation consequences of a dam breach failure. Once the potential consequences are known, the IHP classification is determined and an appropriate IDF is selected from published tables (MNR 1999).

Given the very small size of the Crooks' Hollow reservoir, dam breach modeling was not included in the scope of work for this project. However, under a separate engineering assignment, Klohn Crippen undertook dam breach modeling of the Christie Dam which is located a short distance upstream. From this, the potential inundation limits for a failure of the Christie dam would generally characterize the potential impacts within the floodplain. In particular, it was considered that the impacts of a failure of the Crooks' Hollow dam would be less than that for the Christie dam.

Preliminary results from the Christie Dam failure modeling were reviewed. From this it was concluded that a failure of the Crooks' Hollow dam is not expected result in loss of life. However, the release of silt accumulations behind the dam would potentially cause significant environmental damage to the sensitive Cootes Paradise area. On this basis, the IHP rating is confirmed to be SIGNIFICANT for both the sunny day and flood conditions. According to the MNR draft dam safety guidelines (Table 1.2, MNR, 1999), the IDF is within the range of the 1:100 yr to the regulatory flood. For this case, it is considered that the Crooks' Hollow dam would fall into the lower end of the range and accordingly, the 1:100-yr event (34 m^3/s) was chosen for the IDF.

Table 6.2

Freeboard Assessment for Crooks' Hollow Dam

Structure		Deck/ Crest Elevation m. (ft)	Normal Condition					Unusual Condition (IDF)					Available Freeboard		Remarks
				1:1000 Wind			Total		1:100 Wind			Total			
			Normal Water Level m. (ft)	Design Wave Height m. (ft)	Wave Run-Up m. (ft)	Wind Setup m. (ft)	Wind and Wave Effects m. (ft)	IDF Water Level m. (ft)	Design Wave Height m. (ft)	Wave Run-Up m. (ft)	Wind Setup m. (ft)	Wind and Wave Effects m. (ft)	Deck/ Crest [(1)] Normal m. (ft)	Deck/ Crest [(2)] IDF m. (ft)	
Concrete Gravity Dam	m. ft.	219.33 719.59	216.28 709.58	0.39 1.28	0.36 1.18	0.00 0.00	0.36 1.18	218.20 715.88	0.30 0.98	0.30 0.98	0.00 0.00	0.30 0.98	2.69 8.83	0.83 2.72	Freeboard is adequate for Normal and IDF conditions

Notes:
Normal freeboard is calculated using the normal water level of the reservoir.
Unusual freeboard is calculated using the inflow design flood reservoir water level.
All elevations referred to Canadian Geodetic Datum (CGD).
(1) Normal available freeboard = crest/deck elevation - (NWL + 1:1000-yr wind setup + 1:1000-yr wave run-up).
(2) Unusual available freeboard = crest/deck elevation - (IDF + 1:100-yr wind setup + 1:100-yr wave run-up). A negative value indicates overtopping.

7 Civil/Structural Assessment

Stability analyses were performed using the parameters and the general methods described herein. In performing these analyses, notes and photographs produced during the site inspection phase of the work, as well as site-specific geologic data, were used to assist in the assessment of the structure. These site-specific data obtained during the site visit are described in Section 4 of this report. The results of the stability analyses were used to determine if the Crooks' Hollow Dam satisfies the criteria provided in Sections 6.0 and 7.0 of the draft ODSG. The results from these analyses, together with the results obtained from the various other assessments prepared as part of this study, form the basis of the recommendations for remedial work as detailed in Section 9 of this report.

7.1 Method of Analysis

A dam safety analyses involves the assessment of the ability of the structure to resist

- sliding at the dam-foundation interface, within the dam and at any plane in the foundation under all loading conditions
- overturning
- overstressing of the concrete dam or foundation.

This analysis was performed using the 'rigid body' limit equilibrium method with various load combinations treated as static because of the relatively sustained nature of loads involved.

For critical, representative sections of the structures, sliding and strength factors, normal stresses at the heel and the toe, and the position of the resultant were determined. Where the location, magnitude, direction and duration of computed tensile stresses were such that the stresses would be likely to produce tensile cracking, the extent of cracking was evaluated.

Seismic analyses are typically performed at different levels of sophistication depending on the hazard potential rating of the dam and the probability of unacceptable performance. Because of the relatively low earthquake potential in the Hamilton region, pseudostatic methods of analysis were used.

7.2 Selection of Loads

The following loads were considered in the stability assessment of the Crooks' Hollow Dam:

- dead loads of permanent structures and equipment (D)
- maximum normal headwater level combined with the most critical concurrent tailwater level (H)
- maximum flood headwater level based on the IDF (Hazel Flood in 1954) with corresponding tailwater levels (H_F)
- internal water pressure and foundation uplift (U)
- static thrust created by an ice sheet (I)
- loading due to rock or soil backfill and loads from silt deposited against the structure (S)
- maximum design earthquake (MDE) (Q).

7.2.1 Ice Loads

The Crooks' Hollow reservoir is subject to thermally driven, static, ice loads associated with the formation of a solid ice sheet in front of the dam. Values used in the design review were assessed by taking into consideration site-specific characteristics and dam operator information.

For ice loadings, it is assumed that horizontal thrust created by thermal expansion of ice sheets would occur 0.3 m below the head-pond level. Research by OPG, Manitoba Hydro, Fleet Technology and others has shown that the magnitude of this ice thrust depends on factors such as the thickness of the sheet of ice, the average ambient temperature, the rate of temperature change in the ice, fluctuations in the water surface, reservoir characteristics and wind drag.

Temperature data required as part of the ice load assessment was established by considering January 1% temperatures from the Ontario Building Code. For the Crooks' Hollow Dam, the January 1% temperature for Hamilton was determined to be -19°C.

Reservoir shoreline characteristics were measured from the topographic details established during the site inspections. The shoreline characteristics may be considered as fitting within the category "steeper shore", having a slope of 20° to 45° on average.

Using procedures for estimating ice loads presented by OPG at a workshop on ice held at the annual Canadian Dam Association conference in 2000 and detailed in Table 7.1, the resulting ice thrust values used for analysis can be estimated. The results of this assessment showed that the following ice loads should be considered at the Crooks' Hollow Dam:

- ice load on concrete 73.0 kN/m
- ice load on timber logs 29.2 kN/m

Base on the configuration of the Crooks' Hollow Dam, additional ice loads were considered when the winter water level was below the sill elevation in bays 1, 3, and 4. As a result, ice loads were considered to act against concrete for the entire width of bays 1, 3, and 4.

Table 7.1
Thermal Ice Loads on Concrete Dams

Reservoir Shoreline Characteristics	Winter Air Temperature (January 1% Temperature* from OBC)		
	Mild 0° to -20°C	**Average -21° to -29°C**	**Severe -30°C & Lower**
Flat Shore (<20° slope)	58.4 kN/m (4 kips/ft)	80.2 kN/m (5.5 kips/ft)	102.1 kN/m (7 kips/ft)
Steeper Shore (20° to 45° slope)	73.0 kN/m (5 kips/ft)	87.5 kN/m (6 kips/ft)	116.7 kN/m (8 kips/ft)
Steep Rocky Shore (>45° slope)	87.5 kN/m (6 kips/ft**)	116.7 kN/m (8 kips/ft**)	145.9 kN/m (10 kips/ft**)

Notes:
1. *The January 1% temperature is defined as the lowest temperature at or below which only 1% of the hourly exterior air temperatures in January occur. The January 1% temperature for selected locations in Ontario are tabulated in the Ontario Building Code (OBC).
2. ** For steep rocky shoreline, careful study of the site-specific condition with regard to the shape of the head pond, snow cover data and temperature records is required to determine the design ice load magnitude, as the ice load can be larger than the values shown in the table.
3. Ice load for steel gates = 50% of the values shown in the table.
4. Ice load for timber logs = 29.2 kN/m (2.0 kips/ft).
5. Ice load reduction where timber crib remains exist at or above the waterline shall be based on the location, top elevation, and flexibility of the subject timber crib structure.
6. Minimum ice load where ice sheet existed against the structure = 29.2 kN/m (2.0 kips/ft).
7. Maximum water level in January from past records (from 30 to 80 years) shall be considered for the 'winter operating condition' in the design review. However, this water level may not be much different from the maximum headwater level given for the summer condition.
8. Site-specific conditions based on the design review inspection shall be used in selecting the appropriate design ice load.

7.2.2 Hydrostatic Uplift

Hydrostatic pressures within the dam and foundation are considered as follows.

- **Case 1:** For dams with no foundation drains or pressure relief systems, full uplift, varying linearly from 100% headwater pressure at the upstream face to 100% tailwater pressure at the downstream face, is assumed to act on the entire base area of the dam.

- **Case 2:** For dams equipped with an effective drainage and/or pressure relief system (based on field investigations and/or monitoring data), reduced uplift is used. The reduced uplift varies from 67% of upstream headwater pressure to 100% tailwater pressure, only if the actual recorded uplift is less.

At the Crooks' Hollow Dam, Case 1 applies to the sections considered for the stability analysis.

The uplift assumption corresponds to the design water levels and does not consider any 'locked in' pressures. If base tensions exceed allowable limits (typically assumed to be zero), it is assumed that tension cracking of the base occurs at that level, which allows full uplift pressures to be transmitted along the crack for cases not involving earthquake loadings. In the case of earthquakes, it is assumed that the motions are of such a short duration that uplift pressures will not be increased within any crack that may be theoretically induced from the earthquake loadings.

7.2.3 Seismic Loads

Probabilistic earthquake parameters for the damsite were established based on data obtained from the Geological Survey of Canada (GSC) in Ottawa, for the stability assessment on the near by Christie Dam, as summarized in Table 7.2 (Acres, August 2002).

Table 7.2 **Probabilistic Earthquake Parameters**				
Peak Horizontal Ground Accelerations for Christie Dam				
Probability of Exceedance per Year	0.010	0.005	0.0021	0.001
Return Period	1:100 yrs	1:200 yrs	1:476 yrs	1:1000 yrs
Peak Horizontal Ground Acceleration	0.026 g	0.034 g	0.048 g	0.065 g
Pseudo-static Design Value (horizontal)	0.017 g	0.023 g	0.032 g	0.043 g
Pseudo-static Design Value (vertical)	0.012 g	0.015 g	0.021 g	0.029 g

The draft ODSG require that dams

> "*. . . be designed and evaluated to withstand ground motions associated with a Maximum Design Earthquake (MDE), without release of the reservoir*"

with the selection of the MDE for a dam being based on the hazard potential classification and consequences of dam failure. As shown in Table 1.3, for any given site, the MDE increases with increasing hazard potential due to dam failure.

For the case of the Crooks' Hollow Dam, an IHP classification of SIGNIFICANT was established. On this basis, a 1:100-yr earthquake event was selected as the MDE for stability assessment. In order to examine the case where an earthquake coincides with maximum ice loads, an earthquake of 1:100-yr event was also selected.

For the seismic loading condition in conjunction with summer water levels (MDE), a horizontal pseudo-static design force of 0.017 g was applied (increasing the driving force) concurrent with a vertical pseudostatic design force of 0.012 g (decreasing the resisting force).

Similarly for the seismic loading used with winter water levels in conjunction with horizontal ice loading, a horizontal pseudostatic design force of 0.017 g was applied concurrent with a vertical pseudostatic design force of 0.012 g.

7.2.4 Hydrostatic Loads

Water levels used in the stability assessment for the various load combinations were based on the original maximum design water levels and the reduced water level that was recommended from a previous stability report by Peto MacCallum in 1993. The maximum flood water levels were based on the IHP classification of the dam with the IDF equivalent to the Regional Flood. Maximum levels were determined to be as follows:

Original Designed Water Levels

• Summer Normal	headwater level	=	218.24 m
	tailwater level	=	212.83 m
• Winter Normal	headwater level	=	217.27 m
	tailwater level	=	212.83 m

Reduced Water Level, 1993

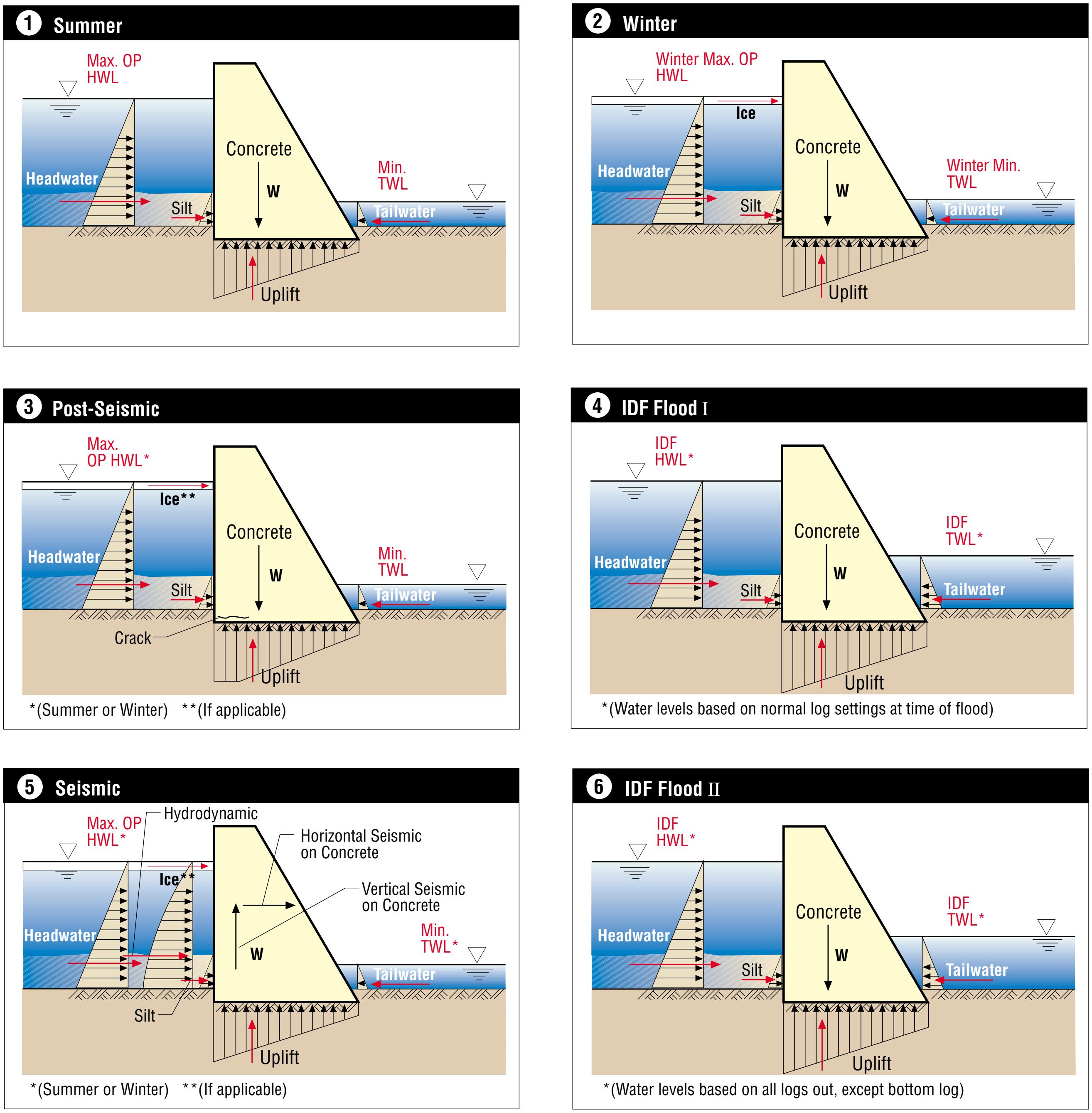
• Summer Normal	headwater level	=	216.28 m
	tailwater level	=	212.83 m
• Winter Normal	headwater level	=	215.06 m
	tailwater level	=	212.83 m

Flood Water Levels

• 1:100yr. Flood	headwater level	=	218.20 m
	tailwater level	=	213.48 m
• Hazel Flood (IDF)	headwater level	=	221.50 m
	tailwater level	=	216.15 m

7.2.5 Load Combinations

The various loading combinations used in the stability assessment of the Crooks' Hollow Dam are shown schematically in Figure 7.1 and are described as follows. Numbers in parentheses refer to the numbers in Figure 7.1.



Legend
OP Operating
HWL Head Water Level
TWL Tail Water Level
IDF Inflow Design Flood

Figure 7.1
Hamilton Conservation Authority
Crooks' Hollow Dam
Schematic of Load Cases

H/01668D0_rm

Back of figure

Usual Loading (1) and (2)

Permanent and operating loads were considered for both summer and winter conditions including self-weight, ice, silt, earth pressure, and the normal maximum operating water level with appropriate uplift pressures and tailwater level.

Unusual Loading (3)

Where earthquake-induced cracking at the rock concrete interface or any weak section was identified, a stability analysis was carried out to determine the stability of the structure, in its post-earthquake condition, under the effects of the usual loading conditions that could include concurrent ice loadings in areas where appropriate. Full reservoir pressure within the earthquake-induced cracks is assumed for the post-earthquake case.

Flood Loading I (4)

Permanent and operating loads of the usual loading case, except for ice loading, were considered in conjunction with reservoir and tailwater levels and uplift resulting during the passage of the 1:100-yr flood with all stop logs removed. The effect of ice loads was not considered simultaneously with design flood conditions in accordance with the requirements of the draft ODSG.

Flood Loading II (5)

Permanent and operating loads of the usual loading case, except for ice loading, were considered in conjunction with reservoir and tailwater levels and uplift resulting during the passage of the IDF (Hazel flood) with all stop logs removed out of all bays. The effect of ice loads was not considered simultaneously with design flood conditions in accordance with the requirements of the draft ODSG.

Seismic Loading (6)

Permanent and operating loads from the usual loading were considered in conjunction with the seismic loads that would be generated during the MDE. During this extreme load case, ice loads are also considered. Uplift pressures were assumed to be those corresponding to the normal loadings, and were not modified during the seismic event.

7.3 Stability Assessment Criteria

7.3.1 Selected Shear Strength Parameter

As described in Section 4.3.4 of this report, the Crooks' Hollow Dam is founded on horizontally and thinly bedded limestone bedrock

Stability of the structure against sliding depends on the angle of friction for the foundation bedrock. In the absence of test data or further bedrock information, an angle of 35° was estimated, using the Barton equation. As noted previously, this was based on an assumed a roughness factor (JRC) of 2, a foundation loading of 0.1 MPa and a bedrock strength of 40 MPa.

The allowable bearing capacity of the bedrock under the Crooks' Hollow Dam was estimated to be 4 MPa.

7.3.2 Performance Indicators

The assessment of the stability of the Crooks' Hollow Dam was measured against the following performance indicators:

- position of resultant force
- normal stresses at the heel and the toe
- calculated sliding factors and strength factors.

Position of Resultant Force

The draft ODSG guidelines indicate that the position of the resultant should be within the middle third of the base for usual loading cases. For other load cases, the resultant may be outside the middle third, provided the other performance indicators are found to be satisfactory.

Normal Stresses at the Heel and the Toe

The draft ODSG allow tensile stresses as long as the limits of 0.1 fc' and 0.05 fc' (where fc' is the compressive strength of concrete, assumed to be 20 MPa), within the mass concrete and at lift joints, respectively, are not exceeded. Compressive stresses at the toe of a dam are limited to 0.45fc'. Tensile strength along the concrete rock interface was taken as zero.

Compressive strength along this same interface was conservatively taken as 4 MPa, the value for bedrock.

Sliding Factors and Strength Factors

The draft ODSG require resistance of a gravity dam against sliding on any surface to be assessed by comparing the Net Driving Force with its Available Shear Strength. The ratio of the Available Shear Strength and the Net Driving Force is referred to as the Sliding Factor (SF).

$$SF = \frac{\text{Available Shear Strength}}{\text{Net Driving Force}}$$

Acceptable sliding and strength factors, used in the stability assessment of the Crooks' Hollow Dam are listed in Table 7.3. These are based on the residual or post-peak shear strength values assumed.

Table 7.3
Acceptable Sliding and Strength Factors for Gravity Dams

Type of Analysis	Load Case			
	Usual	Unusual (Post-Earthquake)	Earthquake (MDE)	Flood (IDF)
Residual Sliding Factor (RSF)	1.5	1.1	1.0	1.3
Concrete Strength Factor	3.0	1.5	1.1	2.0

7.4 Summary of Stability Analysis

7.4.1 Input Parameters

Loads were combined as described in Section 7.2 to a 3-dimensional 'rigid-body' model of the Crooks' Hollow Dam. The model represents three critical stability components of the dam, comprising of two typical pier sections with adjacent spillways on either side and a non-overflow bulkhead section. The model, assumed material properties, and other input are summarized in Appendix D.

7.4.2 Results

Detailed results of the stability analysis are found in Appendix D. They have been compared using assessment criteria as outlined in Section 7.3. Results of the stability assessment for the most unstable section, Pier 2 (Spillways 2 and 3) are summarized in Table 7.4.

Table 7.4
Results of Stability Assessment for Pier 2

Load Combination	SF		Position of Resultant (m)		Normal Stress (kPa) (compression = -ve)			
					Toe		Heel	
	Calc.	Allow.	Calc.	Allow.	Calc.	Allow.	Calc.	Allow.
Usual (Summer) Original WL	0.07	1.50	-5.488	1.88 to 3.77	Unstable	-2667	Unstable	-2667
Usual (Winter) Original WL	0.27	1.50	0.186	1.88 to 3.77	Unstable	-2667	Unstable	-2667
Usual (Summer) Reduced WL, 1993	1.50	1.50	2.986	1.88 to 3.77	-46.58	-2667	-32.99	-2667
Usual (Winter) Reduced WL, 1993	1.60	1.50	2.801	1.88 to 3.77	-44.58	-2667	-46.96	-2667
Unusual (Post EQ)	1.50	1.10	2.986	-	-46.58	-3636	-32.99	-3636
Flood I (1:100yr.)	1.02	1.30	2.315	-	-19.27	-3077	-64.75	-3077
Flood II (Hazel)	0.20	1.30	-1.680	-	Unstable	-3077	Unstable	-3077
EQ (summer)	1.36	1.00	2.871	-	-40.93	-4000	-37.10	-4000
EQ (winter)	1.48	1.00	2.724	-	-40.15	-4000	-49.85	-4000

These results indicate that the Crooks' Hollow Dam control structure does not meet the stability performance indicators as outlined in the draft ODSG for the original water level. Preliminary rock anchor calculations were preformed to determine the number and size of rock anchor required to stabilize the Crooks' Hollow Dam. These calculations are summarized in Appendix D.

The acceptance criteria are met for the normal loading conditions at the reduced reservoir level but not for the 1:100-yr flood loading case.

7.4.3 Preliminary Rock Anchor Stabilization

Preliminary calculations were preformed to determine the number and size of remedial post-tensioned rock anchor required to stabilize the Crooks' Hollow dam for the original water level since this is one option being considered for the disposition of this structure. These calculations are summarized in Appendix D. Three 32-mm diameter post-tensioned rock anchor bars per pier are required to stabilize the dam at the original design water levels. However, this number of anchors is considered to be an impractical installation in the existing pier sections, given their condition.

The concrete coring preformed in 1993 by Peto MacCallum indicated that the concrete in the pier section is in poor condition. As a result, further concrete coring investigations will be required to determine the structural integrity of the pier section before determining the most economical and practical solution to stabilize the Crooks' Hollow Dam.

For the purposes of this report, it is assumed that the poor concrete generally reported within the structure would preclude the use of rock anchors since their highly concentrated load would cause damage to or potential cracking of, the spillway piers. Accordingly, cost estimates for remedial solutions involving anchoring generally consider that the spillway piers would be replaced prior to anchoring.

8 Review of Current Dam Documentation

This was not included as part of the scope of work.

9 Recommendations and Budget Cost Estimates

9.1 Conclusions and Recommendations

A summary of the results of the DSA of the Crooks' dam is presented below. Life extension or decommissioning options are addressed in detail in a separate Class Environmental Assessment Report which explores various disposition alternatives for the dam.

- In accordance with the draft ODSG, based on dam height and reservoir storage volume, the dam is considered to be SMALL height with a SMALL storage reservoir.

- Based on a review of dam break modeling carried out for the Christie dam by others it appears that the incremental effects of a sunny day dam breach would be minimal and no loss of life is expected.

- The incremental effects of a dam breach during the IDF (i.e., regulatory flood) would be minimal and no incremental loss of life is expected.

- On the basis of the consequences of dam failure (as estimated by dam break modeling), the dam is classified as having a SIGNIFICANT IHP.

- The draft ODSG indicates that for a SMALL dam with a SIGNIFICANT hazard potential, the IDF will be between the 1:100-yr flood and the regulatory flood. For the Crooks' Hollow dam, which would be considered to be at the low end of the range, the 1:100-yr flood is specified with an inflow of 34 m^3/s. This dam benefits from the flow regulation offered by the Christie dam immediately upstream. The dam has adequate spillway capacity to safely pass the IDF.

- The freeboard criteria are satisfied for both the normal and flood conditions.

- The concrete structures are generally in fair to poor condition. Cracking and general deterioration of the remedial shotcrete layer was observed on various surfaces.

- Based on stability calculations prepared for this study, the concrete structures do not meet current stability criteria for the load cases checked when the original design water level is applied. The structure is however, considered to meet current stability criteria for the reduced water levels as currently operated for normal loading conditions. Stability criteria are not met for the IDF flood case.

- Public safety signage is sparse and inadequate. New signage at the spillway should be provided based on current MNR guidelines.

9.2 Budget Cost Estimates

Four alternatives are being considered for the disposition of the Crooks' Hollow dam:

1 - Do-Nothing Alternative
2 - Repair the dam to accept the original design water levels
3 - Convert the dam to a low overflow weir (two crest heights considered and pedestrian access is maintained through the provision of a steel footbridge)
4 - Complete removal of the dam.

These are discussed and evaluated in a parallel report entitled "Crooks' Hollow Dam – Class EA" Hatch Acres, 2006. A summary table from the report is produced below including life cycle costs. Tables 9.1 through 9.4 provide more detail on these alternatives.

These costs have been extended to consider full life cycle costs as shown in Figure 9.1 on the following page.

Figure 9.1
Life Cycle Costs for Various Alternatives

All costs = $ x 1000

Alt	Description	Construction Cost ($2007)	Engineering & Construction Management	Sediment Management	Subtotal	Cont. 25%	Total Estimated Construction Cost ($ 2007)	Annual Operations, Maintanance/ Repair Cost	NPV of Annual Costs [1]	Total Life Cycle Cost
1	Do Nothing[2]	--	--	--	--	--	--	$30	$413	$443
2	Repair Dam (Table 9.1)	$572	$200	--	$772	$193	$965	$15	$206	$1186
3	Convert to Overflow Weir (EL 216.28m) (Table 9.2)	$455	$150	--	$605	$151	$756	$5	$69	$830
4	Convert to Overflow Weir (EL 215.06m) (Table 9.3)	$429	$150	--	$579	$145	$724	$5	$69	$798
5	Complete Removal[3, 4] (Table 9.4)	$320	$210	$200	$730	$183	$913	$2	$28	$943

Notes:

1. i=6%, 30 yrs
2. Not an acceptable approach since dam safety criteria for sliding stability are not met
3. Maintenance includes for foot bridge
4. Engineering includes for additional special studies (fluvial geomorphology, fish habitat restoration/compensation, sediment transport analysis

$$\boxed{A = P\left[\frac{i(1+i)^N}{(1+i)^N - 1}\right]}$$

HATCH energy

Table 9.1
Repeair of Existing Dam Cost Estimate

July 2007

Item	Description	Estimated Quantity	Unit	Unit price ($)	Total ($)
1	**Mobilization and Demobilization**				
1.1	Mobilization	1	L.S.	$6,000	$6,000
1.2	Demobilization	1	L.S.	$3,000	$3,000
1.3	Water Control	1	L.S.	$10,000	$10,000
Subtotal					**$19,000**
2	**Removal**				
2.1	Removal of Steel Deck	1	L.S.	$6,000	$6,000
2.2	Concrete Demolition	133	m^3	$1,200	$159,600
Subtotal					**$165,600**
3	**Concrete Replacement**				
3.1	Concrete	150	m^3	$900	$135,000
3.2	Install 36 mm Dia Rock Anchors (9~10m Deep)	13	ea	$10,000	$130,000
Subtotal					**$265,000**
Subtotal Construction Cost					**$449,600**
Contingency (20% of Total Construction Cost)					**$89,920**
Total Cost ($2006)					**$540,000**

Escalated to $2007 (@ 6%) 6 **$572,000**

Exclusions:

- Escalation Beyond August 2007
- Project Insurance
- GST
- Engineering

Table 9.2
Converting Existing Dam to Overflow Weir-Cost Estimate (EL.216.28m)

HATCH energy

July 2007

Item	Description	Estimated Quantity	Unit	Unit price ($)	Total ($)
1	**Mobilization and Demobilization**				
1.1	Mobilization	1	L.S.	$6,000	$6,000
1.2	Demobilization	1	L.S.	$3,000	$3,000
1.3	Water Control	1	L.S.	$10,000	$10,000
Subtotal					**$19,000**
2	**Removal**				
2.1	Removal of Steel Deck	1	L.S.	$6,000	$6,000
2.2	Concrete Demolition	106	m^3	$1,500	$159,000
Subtotal					**$165,000**
3	**Concrete Replacement**				
3.1	Concrete	100	m^3	$900	$90,000
3.2	Install 36 mm Dia Rock Anchors	8	ea	$10,000	$80,000
Subtotal					**$170,000**
4	**Shore Protection**				
4.1	Rip rap	96	m^3	$40	$3,840
Subtotal					**$3,840**
5	**Footbridge**				
5.1	Bridge Span 19 m	1	L.S.	$50	$50
Subtotal					**$50**
Subtotal Construction Cost					**$357,890**
Contingency (20% of Total Construction Cost)					**$71,578**
Total Cost ($2006)					**$429,000**

Escalated to $2007 (@ 6%) 6 **$455,000**

Exclusions:

- Escalation Beyond August 2007
- Project Insurance
- GST
- Engineering

HATCH energy

Table 9.3
Converting Existing Dam to Overflow Weir-Cost Estimate (EL.215.06m)

July 2007

Item	Description	Estimated Quantity	Unit	Unit price ($)	Total ($)
1	**Mobilization and Demobilization**				
1.1	Mobilization	1	L.S.	$6,000	$6,000
1.2	Demobilization	1	L.S.	$3,000	$3,000
1.3	Water Control	1	L.S.	$10,000	$10,000
Subtotal					**$19,000**
2	**Removal**				
2.1	Removal of Steel Deck	1	L.S.	$6,000	$6,000
2.2	Concrete Demolition	103	m^3	$1,500	$154,500
Subtotal					**$160,500**
3	**Concrete Replacement**				
3.1	Concrete	82	m^3	$900	$73,800
3.2	Install 36 mm Dia Rock Anchors	8	ea	$10,000	$80,000
Subtotal					**$153,800**
4	**Shore Protection**				
4.1	Rip rap	96	m^3	$40	$3,840
Subtotal					**$3,840**
5	**Footbridge**				
5.1	Bridge Span 19 m	1	L.S.	$50	$50
Subtotal					**$50**
Subtotal Construction Cost					**$337,190**
Contingency (20% of Total Construction Cost)					**$67,438**
Total Cost ($2006)					**$405,000**

Escalated to $2007 (@ 6%) 6 **$429,000**

Exclusions:

- Escalation Beyond August 2007
- Project Insurance
- GST
- Engineering

HATCH energy

Table 9.4
Removal of Existing Dam Cost Estimate

July 2007

Item	Description	Estimated Quantity	Unit	Unit price ($)	Total ($)
1	**Mobilization and Demobilization**				
1.1	Mobilization	1	L.S.	$6,000	$6,000
1.2	Demobilization	1	L.S.	$3,000	$3,000
1.3	Water Control and Silt Barriers	1	L.S.	$14,000	$14,000
Subtotal					**$23,000**
2	**Demolition**				
2.1	Demolition of Steel Deck	1	L.S.	$6,000	$6,000
2.2	Concrete Demolition	975	m^3	$215	$209,625
Subtotal					**$215,625**
3	**Protection Work**				
3.1	Back Fill Removal	183	m^3	$10	$1,830
3.2	Fill Placement and Landscping	312	m^3	$10	$3,120
3.3	Shore Protection (Rip Rap)	192	m^3	$40	$7,680
Subtotal					**$12,630**
4	**Footbridge**				
4.1	Bridge Span 27 m	1	L.S.	$75	$75
Subtotal					**$75**
Subtotal Construction Cost					**$251,330**
Contingency (20% of Total Construction Cost)					**$50,266**
Total Cost ($2006)					**$301,596**

Escalated to $2007 (@ 6%) 6 **$320,000**

Exclusions:

- Escalation Beyond August 2007
- Project Insurance
- GST
- Engineering

10 Bibliography

Acres International Limited, "Christie Dam, Evaluation of New Winter Operating Level". August 2002 (P14260.00).

Hatch Acres, 2006, Sediment Study

MacLaren Plansearch 1990. "Technical Report of the Spencer Creek Watershed Hydrology Study".

Peto MacCallum, 1993.

Sears, William L. 1968

Sears, William L. 1976.