Upper Thames River
Conservation
Authority

UPPER THAMES RIVER
CONSERVATION AUTHORITY

DAM SAFETY ASSESSMENT REPORT FOR
Embro Dam
Final
July, 2007

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prepared by
ACRES
International

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Executive Summary

The Embro Dam is located approximately 2 km south of the town of Embro on Spring Creek, a tributary of the North Branch Creek. The dam and reservoir were built for recreational purposes and is adjacent to a cultivated farm plot and County Road 16. The conservation reservoir surface area is very small and is impounded by a low earth-fill embankment dam about 100 m long located at the southern end of the pond. The dam has a height of approximately 4.5 m and a freeboard of about 1.1 m. A concrete pipe conduit passes through the dam and an emergency spillway is located on the left abutment.

The dam controls a small drainage area of 7 km^2 comprising mostly agricultural land. Flow releases from the dam outlet enters a narrow channel of the creek as it meanders in a southeasterly direction for approximately 1.6 km before entering the main stem of the North Branch Creek. The North Branch Creek continues to flow in a southerly direction some 4 km before reaching the confluence with the Middle Thames River.

The discharge facilities at the dam consist of a concrete bottom draw inlet structure and an inverted V-shaped trashrack anchored to the top of the inlet. There is an emergency spillway located on the left or east abutment. The upstream end of the emergency spillway has a grassed channel with a clear width of approximately 4 m. The invert of the spillway is about 0.6 m below the adjacent dam crest. The downstream spillway channel runs parallel to the creek before joining it and was overgrown with grass and weeds.

The neighboring area is rolling with a relief of about 15 m or less. Overburden forms both banks upstream and downstream of the dam. No bedrock was seen.

Embro pond has a surface area of 0.008 km^2 (0.08 ha) and controls a total drainage area of 7 km^2. The embankment dam is approximately 3.8 m high and impounds a total estimated storage volume of 0.03 x 10^6 m^3. This classifies the structure as a SMALL dam on the basis of height and a SMALL dam on the basis of storage impounded.

On the basis of the results of the year 2002/2003 dam safety assessment,

- the dam is founded on overburden over its entire length

- the dam can be classified as a SMALL dam on the basis of reservoir size and height

- the dam is classified as a VERY LOW incremental hazard potential (IHP) structure for a dam failure during a flood event

- the inflow design flood (IDF) for this dam is the flood resulting from the 50-yr, 8-day spring snowmelt event

- the dam, with three stop logs removed in the fall, is overtopped during passage of the IDF and has inadequate freeboard. The dam is deemed to not have adequate spillway capacity to pass the IDF.

- both upstream and downstream embankment slopes do not meet slope stability acceptance criteria

- the emergency spillway requires excavation in order to properly convey flood flows away from the left downstream toe of the dam.

The costs associated with the maintenance repairs recommended to ensure the ongoing safe operation of this dam are in the order of $80,820.

Figure ES-1

Embro Dam

Description: Earth Embankment

Original Construction: Unknown
Last Upgrade: 1958
Last Repairs: Unknown
Height: approx. 4.5 m
Length: approx. 100 m
Reservoir Area: 0.005 km^2

Hydrotechnical Issues

Overall IHP Classification: VERY LOW
- Flood VERY LOW (economic loss or loss of life)
- Earthquake VERY LOW (economic loss or loss of life)

IDF: 50-yr, 8-day spring snowmelt event
Spillway Capacity: Inadequate

Issues

General Condition: Generally in good condition
Stability: Both upstream and downstream slopes do not meet criteria

Safety and Operating Issues

Operations: Spring and fall
Signage: Inadequate
Debris Boom: Not applicable
Fall Arrest Systems: Not applicable

Recommendations

- Excavate emergency spillway to divert flows away from toe of dam.
- Install additional signs to satisfy Ministry of Natural Resources' draft standards.
- Flatten upstream and downstream slopes.
- Test the emergency preparedness plan.
- Check pipe outlet alignment.

Costs $80,820

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 Guidelines. These have been formalized in the form of a draft document entitled “Ontario Dam Safety Guidelines” (ODSG).

There are approximately 2200 dams in Ontario. Nearly half of these are privately owned, with the remainder owned by Ontario Power Generation (OPG) and conservation authorities (CAs). The Upper Thames River Conservation Authority (UTRCA), one of 36 CAs in the province of Ontario, operates as do most CAs, under the direction of a Board of Directors comprised of local municipal representatives. Various committees give direction to the CA’s programs and projects involving numerous partnerships. UTRCA owns, operates and maintains dams and other control structures on the Upper Thames River and its various tributaries.

In October 2002, Acres International (Acres) was retained by UTRCA and Ausable Bayfield Conservation Authority (ABCA) to undertake an independent dam safety review of 15 dams and control structures located in the Upper Thames and Ausable/Parkhill basins. Thirteen structures were examined for UTRCA under this review.

This report presents the results of civil, geotechnical, mechanical and hydrologic and hydraulic assessments for the Embro Dam located on Spring Creek, a tributary of the North Branch Creek which flows into the Middle Thames River (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 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 the general 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]
- 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

Figure 1.1
Upper Thames River Conservation Authority
Dam Safety Assessment Report
Aerial Photograph of Dam and Surrounding Area

ACRES International

BACK OF FIGURE 1.1

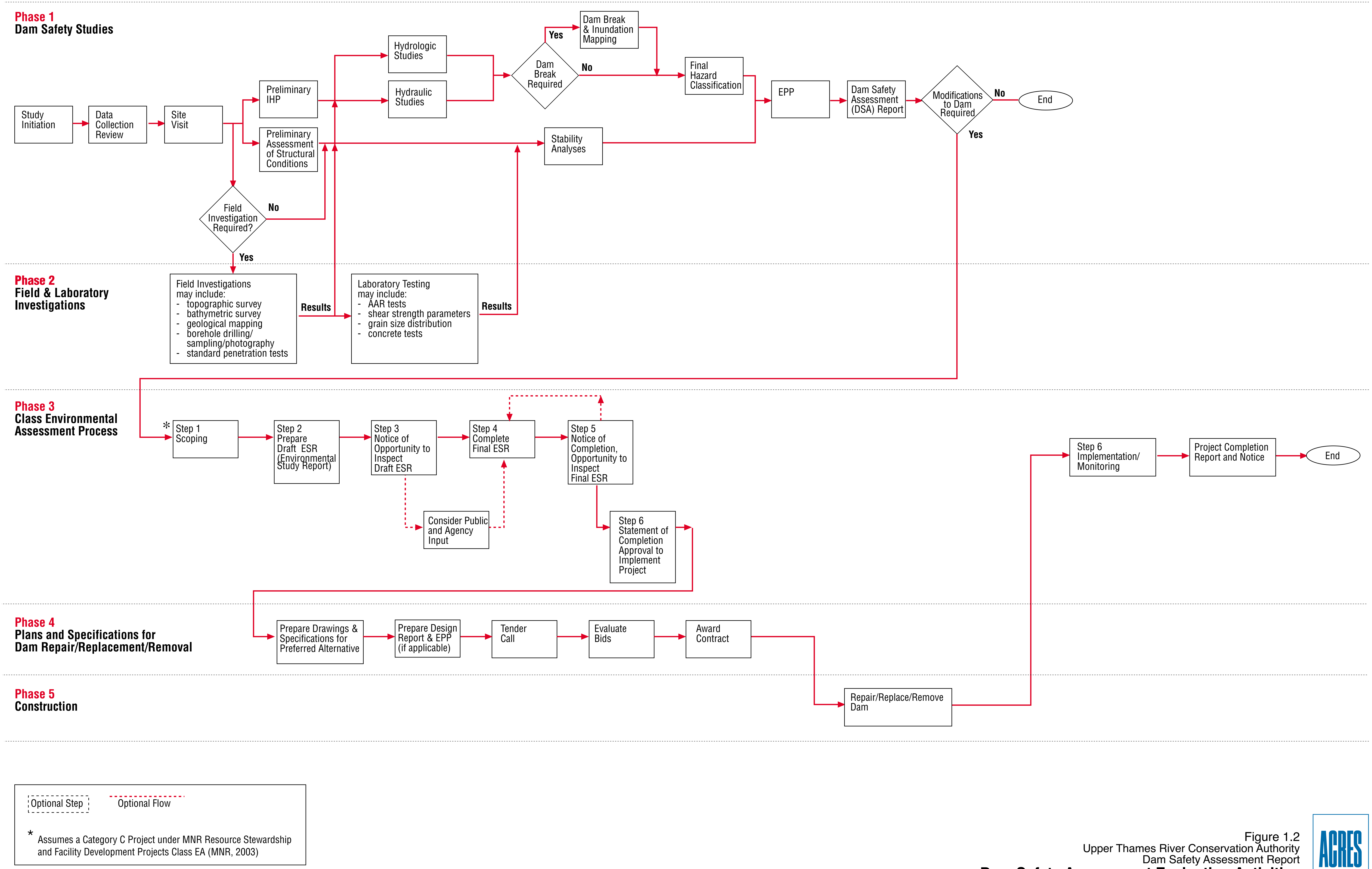

Figure 1.2
Upper Thames River Conservation Authority
Dam Safety Assessment Report
Dam Safety Assessment Evaluation Activities

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FIGURE 1.2 – BACK OF PAGE

damages, environmental losses and the potential for incremental loss of life in the event of a dam failure.

Based on this assessment, an IHP 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 maximum design earthquake (MDE) is selected 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 fill 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) report and a review of the emergency preparedness plan (EPP).

1.3 The Embro Dam Safety Assessment

The Embro Dam is located on Spring Creek, a tributary of the North Branch Creek which flows into the Middle Thames River, as shown in Figure 1.3.

Characteristics of this 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 1.2

Minimum Inflow Design Floods for Dams

(Source: MNR, Draft ODSG)

Hazard Potential	Size of Dam and Inflow Design Floods					
	Small		Medium		Large	
	Height < 7.5 m	**Storage** < 100 x 10^3 m^3	**Height** 7.5 to 15 m	**Storage** 100 x 10^3 to 1000 x 10^3 m^3	**Height** > 15 m	**Storage** > 1000 x 10^3 m^3
Very Low	25-year flood to 50-year flood		50-year flood to 100-year flood		100-year flood to RF	
Low	25-year flood to 100-year flood		100-year flood to RF		RF to PMF	
Significant	100-year flood to RF		RF to PMF		PMF	
					Policy for existing dams is under consideration	
High	RF to PMF		PMF		PMF	
	Policy for existing dams is under consideration					

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)

Notes:

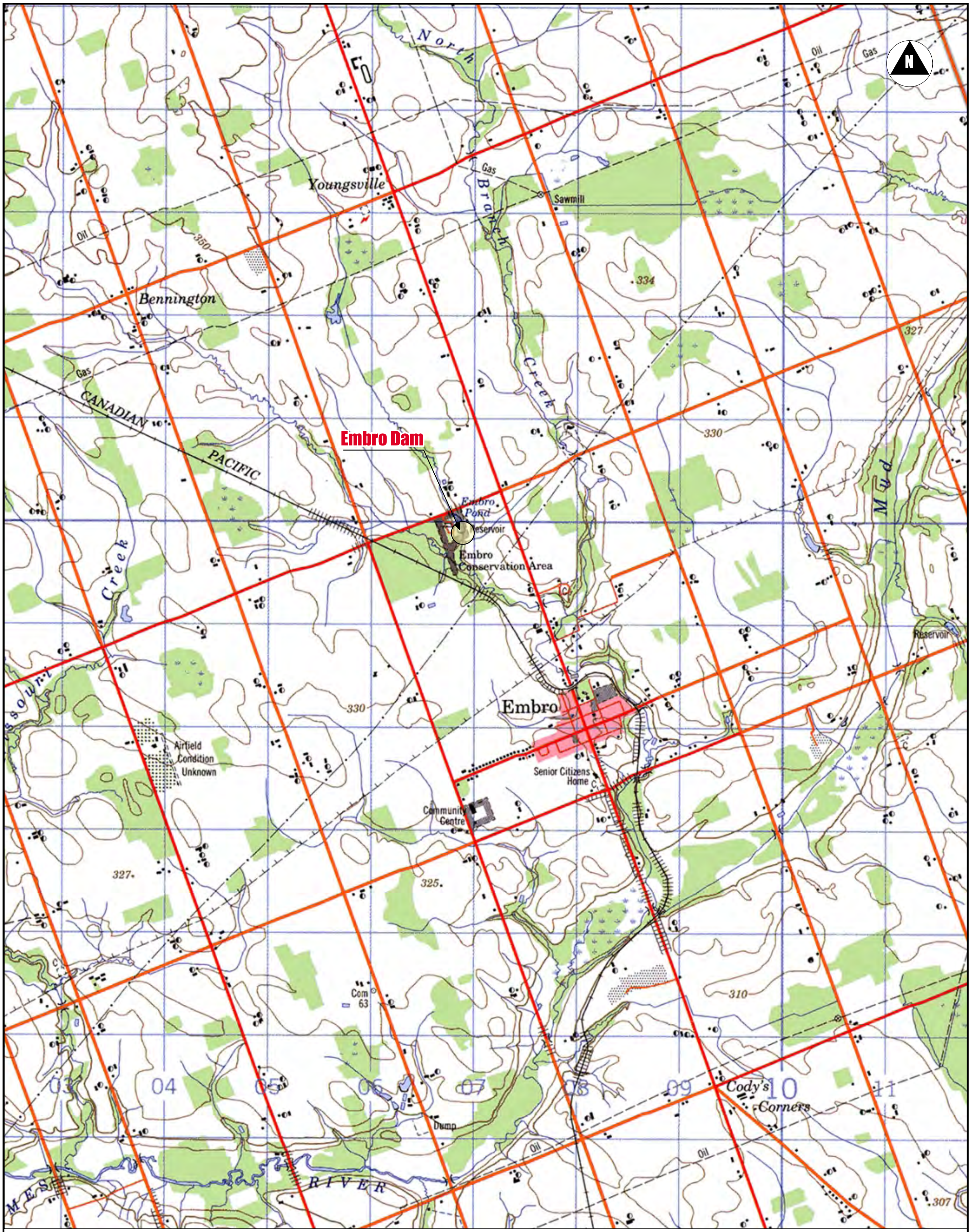
(a) Hazard potential classification established separately for each dam.

(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 damsite, 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.



0 1000 2000m

Figure 1.3
Upper Thames River Conservation Authority
Dam Safety Assessment Report
Location of Embro Dam

FIGURE 1.3 BACK OF PAGE

Table 1.4

Description of the Dam

Name of Dam	Access	Description				
		Drainage Area (km^2)	Reservoir Area (km^2)	Height (m)	Length (m)	No. of Sluices
Embro Dam	Off County Road 16	7	0.005	4.0	100	1 bottom draw inlet with a concrete pipe outlet

Photographs of the damsite and the dam itself are contained in Appendix A of this report. 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 – history of the Embro 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 – details of the results of any site investigations performed to fill data gaps identified during the initial site inspections
- Section 6 – details of the hydrological/hydraulic assessments. The section includes the following main topics:
    - descriptions of river basin characteristics
    - development of flood and storm events
    - development of rainfall/runoff and flood routing models
    - flood flow estimates

  - preliminary IHP and IDF classifications
  - determination of the IDF.

- 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 – details the geotechnical assessments performed including the stability of any earth embankments, seepage, erosion and liquefaction problems and instrumentation found or needed at the dam

- Section 9 – details the results of the evaluation of workplace and public safety at the dam. It includes issues such as the need for fall restraint, signage, operational methods used, the requirement for log booms and other related issues.

- Section 10 – provides a summary of details of the EPP

- Section 11 – provides a summary of the recommended remedial measures needed at the dam and estimated costs.

2 The Embro Dam

2.1 History*

Purchase of the damsite on Spring Creek, northwest of that Oxford County village, was recommended by Dr. Richardson in November 1947, as one of five projects that should be undertaken without delay. But money was scarce, and it was not until 1958 that development began. The then existing dam was in a state of disrepair and part of the spillway had broken down. The old dam was replaced by a 300-ft structure and a lake, 600 ft long by 300 ft wide, was created. To provide a suitable recreation area, 14 acres of the Oxford County Forest and 7 acres of the Charles Harris property were purchased. The area embraces about 29 acres. The official opening took place on October 26, 1959.

In 1964, part of the wooded area was leased to the Thames Valley Scout Council for camping and instruction in sound conservation practices. A new well was installed in 1966. The existing recreation area was expanded in 1968 to better accommodate the general public.

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* Reproduced, with permission, from “Twenty Five Years of Conservation on the Upper Thames Watershed”, 1947 to 1973. Published by the Upper Thames River Conservation Authority.

3 Initial Data Collection/ Review

3 Initial Data Collection/Review

As a first step in the assessment process, a detailed review of the information contained in the UTRCA files was made on November 8, 2002 and on February 5 to 7, 2003. As part of this process, the following documents were examined:

- watershed maps showing damsites and drainage areas
- correspondence files
- previous internal inspection reports
- Ontario Geological Survey maps and documents
- historical records
- meteorological data from selected stations
- records of water levels
- data from selected streamflow gauging stations from Water Survey of Canada (WSC)
- selected topographic maps (1:50 000-scale)
- rating curve calculations.

Provided in Table 3.1 is a list of all documents obtained from UTRCA records. 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. Generally, the dams located in this region are small- to medium-sized concrete or embankment dams constructed on competent bedrock or till foundations with either glacial till or bedrock abutments. The following are some problems which may be expected to occur at dams of this type:

- leakage at overburden contacts, at defects in the concrete, at the concrete/ foundation contact or through open 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 were specifically addressed in addition to other issues that became apparent during the course of the site visit.

TABLE 3.1

UPPER THAMES RIVER CONSERVATION AUTHORITY
Reference Information

Dam	Year	Type of Document	Author	Title	UTRCA Document No.	Filed Under (Dam/Multi)	Acres has Original Document	Acres has Copy of Document	Acres has Summary Notes of Document	Date Received	Date Returned	Comments
Embro	2001	Drawing	R.P. of UTRCA	Dam Hazard Identification, Embro Dam (WECS Program)	-	Dam	No	Yes	-	8-Nov-02	-	
Embro	2000	Inspection Report	Chris Tasker and Al Merry (UTRCA)	-		Dam	No	Yes	No	8-Nov-02	-	1 page report
Embro	1985	Inspection Report	B. Bevan and J.C. Campbell (UTRCA)	Embro Inspection Report		Multi	No	No	Yes	-	-	
Embro	1985	Response Summary	B. Bevan (UTRCA)	Summary of UTRCA Responses RE: 1985 MNR Inspections	WM.2.1	Multi	No	Yes	No	8-Nov-02	-	
Embro	1982	Inspection Report	J. Jilek and P. Bane (UTRCA)	-		Multi	No	No	Yes	-	-	some erosion in spillway

4 Comprehensive Site Inspections and Condition Assessments

4 Comprehensive Site Inspections and Condition Assessments

4.1 Introduction

A site evaluation of the Embro Dam was made on November 12, 2002, by Acres civil and geotechnical engineers, and on November 20, 2002, by hydrotechnical personnel as part of the Dam Safety Program: Review of Dams Owned/Operated by UTRCA and ABCA. The results of these inspections are presented in the following sections, on digital photographs and on Forms B1 and B2 (see Appendix B), all in accordance with MNR, Ontario Dam Safety Guidelines (Draft), August 1999 and the requirements of the request for proposal (RFP), July 2002.

4.2 Antecedent Weather Conditions

Seepage observations noted during site inspections at water-retaining structures may be influenced by weather conditions which occurred at the time of the inspection and during the preceding period. Table 4.1 is a summary of recorded daily precipitation for the month of November 2002 at Embro Innes close to the Embro damsite. From the table, it can be seen that no rain fell on November 12, the day of the civil/geotechnical inspection, but precipitation occurred on November 5, 9, 10 and 11 (total of 51.0 mm during the week prior to the civil/geotechnical inspection).

4.3 Record of Observations

4.3.1 General Description

The Embro Dam is located approximately 2 km south of the town of Embro on Spring Creek, a tributary of the North Branch Creek. The dam and reservoir were built for recreational purposes and is adjacent to a cultivated farm plot and County Road 16, where it crosses over a roadway culvert to Embro pond. The entrance to the dam and reservoir, as well as the park/picnic area, is from Road 16. The conservation reservoir surface area is very small and is impounded by a low earth-fill embankment dam about 100 m long located at the southern end of the pond (Photo 1). The dam has a height of approximately 4.5 m and a freeboard of about 1.1 m. A concrete pipe conduit

Table 4.1

Summary of Daily Precipitation Records from Environment Canada

Environment Canada Environnement Canada

Daily Data Report for November 2002

Notes on Data Quality.

EMBRO INNES
ONTARIO

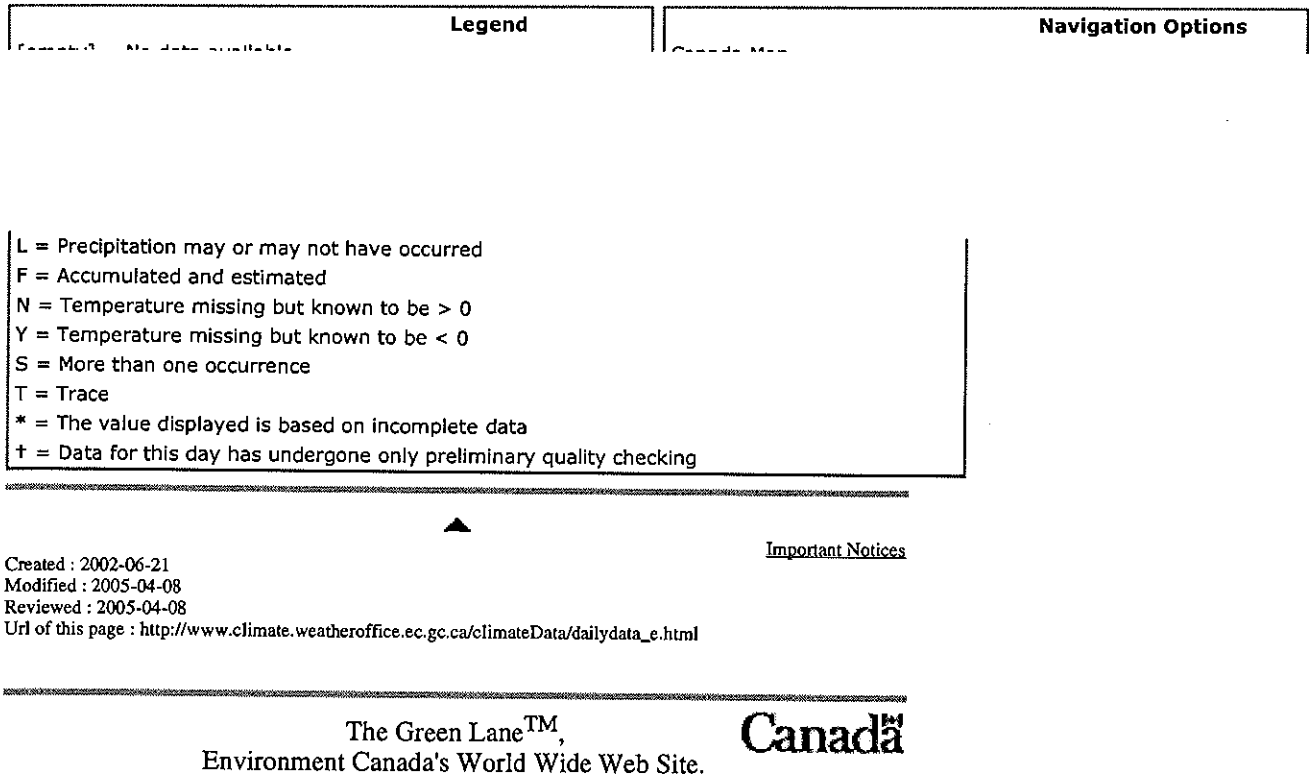
Latitude: 43° 15.000' N **Longitude:** 80° 55.800' W **Elevation:** 358.10 m

Climate ID: 6142295 **WMO ID:** **TC ID:**

Daily Data Report for November 2002

D a y	Max Ter °C	Min Ten °C	Mean Te °C	Heat Deg D °C	Cool Deg D °C	Total Ra mm	Total Sn cm	Total Pre mm	Snow on G cm	Dir of Max (10's Deg	Spd of Max (km/h
01						0.0	6.0	6.0	M		
02						0.0	0.0	0.0	8		
03						0.0	0.0	0.0	3		
04						0.0	0.0	0.0	M		
05						1.6	1.2	2.8	0		
06						T	0.0	T	M		
07						0.0	0.0	0.0	M		
08						0.0	0.0	0.0	M		
09						3.6	0.0	3.6	M		
10						41.8	0.0	41.8	M		
11						4.0E	0.0	4.0E	M		
12						0.0	0.0	0.0	M		
13						0.0	0.0	0.0	M		
14						3.0E	T	3.0E	M		
15						0.0	0.0	0.0	M		
16						0.0	T	T	M		
17						0.0	1.0	1.0	M		
18						T	0.0	T	M		
19						1.0	0.0	1.0	M		
20						0.0	0.0	0.0	M		
21						6.0E	0.0	6.0E	M		
22						T	0.0	T	M		
23						0.0	0.0	0.0	M		
24						0.0	2.0E	2.0E	M		
25						0.0	0.0	0.0	M		
26						0.0	0.0	0.0	M		
27						0.0	0.0	0.0	M		
28						0.0	0.0	0.0	M		
29						0.0	T	T	M		
30						0.0	8.0	8.0	M		
Sum						**61.0E**	**18.2E**	**79.2E**			
Avg											
Xtrm											

Table 4.1
Summary of Daily Precipitation Records from Environment Canada - 2



Legend	Navigation Options

L = Precipitation may or may not have occurred
F = Accumulated and estimated
N = Temperature missing but known to be > 0
Y = Temperature missing but known to be < 0
S = More than one occurrence
T = Trace
* = The value displayed is based on incomplete data
† = Data for this day has undergone only preliminary quality checking

▲

Important Notices

Created : 2002-06-21
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Reviewed : 2005-04-08
Url of this page : http://www.climate.weatheroffice.ec.gc.ca/climateData/dailydata_e.html

The Green Lane^{TM},
Environment Canada's World Wide Web Site.

Canada

passes through the dam (Photo 2), and an emergency spillway is located on the left abutment (Photo 3).

4.3.2 Hydrotechnical Aspects

The dam controls a small drainage area of 7 km^2 comprising mostly agricultural land. Flow releases from the dam outlet enters a narrow channel of creek (Photo 4), as it meanders in a southeasterly direction for approximately 1.6 km before entering the main stem of the North Branch Creek. The North Branch Creek continues to flow in a southerly direction some 4 km before reaching the confluence with the Middle Thames River.

The Embro pond has a limited fetch (Photo 5) and, therefore, negligible wind-generated wave heights at the dam are expected. Sediment deposits were noticed in the pond around the intake structure and along the upstream face of the dam. The water quality of the reservoir does not appear to be very good based on the odor from the pond during the visit. The left or east upstream shoreline is well-vegetated with grasses, brushes and trees (Photo 5); whereas the right west shoreline is a grassed park area with trees (Photo 5).

The discharge facilities at the dam consist of a concrete bottom draw inlet structure and an inverted V-shaped trashrack anchored to the top of the inlet (Photo 6). There is an emergency spillway located on the left or east abutment (Photo 3). The upstream end of the emergency spillway has a grassed channel with a clear width of approximately 4 m. The invert of the spillway is about 0.6 m below the adjacent dam crest. The downstream spillway channel runs parallel to the creek before joining it and was overgrown with grass and weeds.

The entire damsite is founded on overburden, and the discharge from the bottom draw inlet and circular pipe has formed a small pool at the pipe outlet (Photo 7). The channel downstream slopes gradually away from the outlet pipe and the banks are overgrown with grasses, brushes and trees (Photo 4). The creek channel downstream of the reservoir has a reasonable bed slope that winds through wooded land until it reaches open area (farms). There are no permanent dwellings or development in the downstream reach of the discharge channel for a distance of nearly 1 km. The dam was previously overtopped in the summer of 2000 with minor damage.

4.3.3 Geotechnical Aspects

The neighboring area is rolling with a relief of about 15 m or less. Overburden forms both banks upstream and downstream of the dam. No bedrock was seen.

4.3.3.1 Upstream Slope

No unusual conditions such as sloughing, sinkholes, cracking, settlement or displacement were observed on the upstream slope (Photo 8). The slope was overgrown with reeds and grass. No slope protection was apparent. No erosion or benching was apparent.

4.3.3.2 Crest

No unusual conditions such as cracking, displacement, settlement, or sinkholes were observed (Photo 9).

4.3.3.3 Downstream Slope

The downstream slope is densely overgrown with grass and shrub. At the downstream toe on the left side, there was an eroded gully as a result of emergency spillway overflow (Photo 10). The gully is very heavily overgrown with grass and shrub. Overflow water apparently diverts through a topographic low point from the intended emergency spillway channel and follows the downstream toe of the slope, discharging into the tailrace near the concrete pipe conduit. At the discharge point, the gully is up to 0.75 m deep and is about 1.5 m wide. Erosion is working upstream along the toe. One sinkhole was evident in this area. It is not clear why this would develop here. Erosion was also observed on the right side of the concrete pipe conduit (Photo 11). Erosion of the gully on the left side and on the right side may have been exacerbated by heavy discharge through the concrete pipe conduit.

4.3.3.4 Abutments

No unusual conditions such as cracking, movement or leakage were observed in either abutment.

4.3.3.5 Emergency Spillway

At the upstream end, the emergency spillway was clear for a short distance (Photo 3). The downstream flow path was overgrown with grass and was not a defined channel. Given the results described in Section 4.3.3.3, the emergency spillway channel topography is not what it should be.

4.3.3.6 Instrumentation

No instrumentation for dam performance was seen to exist.

4.3.4 Civil/Structural Aspects

At the time of inspection, the concrete intake was overflowing and clogged with weeds and other debris (Photo 6). The trashrack is constructed of galvanized steel and appears to be in good condition and well-anchored to the concrete intake.

The outlet conduit consists of precast concrete pipe with an inside diameter of 762 mm (30 in.) (Photo 2). No joints were exposed at the outlet, and the concrete appeared to be in good condition.

No signs were posted around the dam. The dam is fully accessible to the public with a picnic area located on the upstream right bank (Photo 5).

5 Site

Investigations

5 Site Investigations

Embro Dam was investigated with one borehole which was drilled on November 20, 2003. The borehole was located on the dam centerline a few metres to the right of the culvert which is connected to the bottom draw intake structure in the reservoir as shown on Drawing 14504-EM-001. A CME 75 hollow-stem auger drill was used for the drilling. Close-spaced sampling was done.

The borehole penetrated the fill embankment and the glacial till dam foundation to a total depth of 7.46 m. A standpipe type piezometer was placed to monitor water levels in the embankment.

The borehole log attached presents the detailed findings during the drilling and sampling. A summary is given below.

Laboratory testing was done on some of the samples. This included testing for moisture content, Atterberg limits, grain-size distribution and wet density. Results are found in Table 5.1. Attached are the grain-size plot and plasticity chart.

Table 5.1

Laboratory Test Results for Embro Dam

Bore-hole	Sample	Depth (m)	Wet Density (kg/m^3)	% Moist	LL (%)	PL (%)	PI (%)	Gravel (%)	Sand (%)	Fines (%)
BH-1	AQ3	2.29-2.89		43.8						
BH-1	AQ4	3.05-3.66		35.6						
BH-1	AQ5	3.81-4.42	2080.99	17.4	32	16	16			
BH-1	AQ6	4.57-5.18	2436.96	10.2				8.5	42.8	48.7

Notes:
1 – Samples AQ3, AQ4 and AQ5 are in the embankment fill.
2 – Sample AQ6 is in the glacial till foundation.

Nine split-spoon samples were taken along with the standard penetration test (SPT); five in the embankment fill and four in the till foundation.

Sampling in the fill indicates that the fill is 4.49 m deep and comprises dark brown clay with sandy fine gravel. The material is classified as CL. SPT ‘N’ values in the fill are relatively low and generally range from 3 to 4 (soft consistency), with the exception of one value of 8 at the base of the fill. Liquid limits, plasticity limits and plasticity index of the fines in the embankment are 32%, 16% and 16%, respectively, indicating medium plasticity.

Sampling in the foundation till indicates that the till comprises sandy silt with fine gravel. The material is classified as ML (low plasticity). SPT ‘N’ values in the till increase with depth from 33 (dense) to 69 (very dense). Piezometric levels in the fill were approximately 2.7 m below the crest at the time of the work.

Moisture content of the embankment fill is high, i.e., approximately 44%, in the upper part and decreases with depth to approximately 17%. In the till, it is 10.2%.

Shear strength parameters have been interpreted from the above information for the purpose of stability analysis. This is discussed in Section 8.

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List of Abbreviations and Terms

(Sheet 1)

General

Elevations
Refer to datum indicated on drilling report.

Depth
All depths are given in metres measured from the ground surface unless otherwise noted.

Sample Type
The first letter describes the sampling method and the second, the shipping container.

Sampling Method
A - Split Tube
B - Thin Wall Tube
C - Piston Sampler
D - Core Barrel
E - Auger
F - Wash
G - Shovel Grab Sample
K - Slotted Sampler

Shipping Container
N - Insert
O - Tube
P - Water Content Tin
Q - Jar
R - Cloth Bag
S - Plastic Bag
U - Wooden Box
Y - Core Box
Z - Discharged

Sample No.
Samples are numbered consecutively in the order in which they were obtained in the borehole.

Sample Size
Dimension in millimetres and refers to the nominal diameter of the sampler.

Sample Recovery
Indicates the length in millimetres of sample retained in the sampler.

Sample Retained
Indicates length of sample retained for storage or testing purposes.

Abbreviations
N/A - No applicable
N/E - Not encountered
N/O - Not observed

Permeability

Degree of Permeability	*k(cm/s)*
Very high	$>10^{-1}$
High	10^{-1} to 10^{-3}
Medium	10^{-3} to 10^{-5}
Low	10^{-5} to 10^{-7}
Practically impermeable	$<10^{-7}$

Soil

Standard Penetration Test (SPT)
The test is carried out in accordance with ASTM D-1586 and the >N= value corresponds to the sum of the number of blows required by a 63.5-kg hammer, dropped 760 mm, to drive a 50-mm diameter split tube sampler the second and third 150 mm of penetration.

Grain Size

Clay	<0.002 mm
Silt	0.002 - 0.075 mm
Sand	0.075 - 4.75 mm
Gravel	4.75 - 75 mm
Cobbles	75 - 300 mm
Boulder	>300 mm

Soil Classification and Description
Precise soil classification and description follows USCS, ASTM D 2487. Soil identification that is unsupported by laboratory testing is based on visual examination and manual tests defined in ASTM D 2488.

Relative Density (Granular Soils)

	N(SPT)
Very loose	0 - 4
Loose	4 - 10
Compact	10 - 30
Dense	30 - 50
Very dense	>50

Consistency (Cohesive Soils)

		Undrained Shear Strength	
	N(SPT)	*kPa*	*psf*
Very soft	<2	0 - 12	0 - 250
Soft	2 - 4	12 - 25	250 - 500
Firm	4 - 8	25 - 50	500 - 1000
Stiff	8 - 15	50 - 100	1000 - 2000
Very stiff	15 - 30	100 - 200	2000 - 4000
Hard	>30	>200	>4000

Plasticity/Compressibility

		Liquid Limit (%)
Low plasticity clays	Low compressibility silts	<30
Med. plasticity clays	Med. compressibility silts	30 - 50
High plasticity clays	High compressibility silts	>50

Dilatancy
None - No visible change

Slow - Water appears slowly on surface of specimen during shaking and does not disappear or disappears slowly upon squeezing.

Rapid - Water appears quickly on the surface of specimen during shaking and disappears quickly upon squeezing.

Sensitivity

Insensitive	<2
Low	2 - 4
Medium	4 - 8
High	8 - 16
Quick	>16

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List of Abbreviations and Terms
(Sheet 2)

Rock

Core Recovery
Sums of lengths of rock core recovered from a core run, divided by the length of the core and expressed as a percentage.

RQD (Rock Quality Designation)
Sum of lengths of hard, sound pieces of rock core equal to or greater than 100 mm from a core run, divided by the length of the core run and expressed as a percentage. Measured along centerline of core. Core fractured by drilling is considered intact. RQD normally quoted for N-size core.

RQD (%) Rock Quality

90 - 100	Excellent
75 - 90	Good
50 - 75	Fair
25 - 50	Poor
0 - 25	Very Poor

Grain Size

Term	*Grain Size*
Very coarse-grained	>60 mm
Coarse-grained	2 mm - 60 mm
Medium-grained	60 µm - 2 mm
Fine-grained	2 µm - 60 µm
Very fine-grained	<2 µm

Bedding

Term	*Bed Thickness*	
Very thickly bedded	>2 m	>6.50 ft
Thickly bedded	600 mm - 2 m	2.00 - 6.50 ft
Medium bedded	200 mm - 600 mm	0.65 - 2.00 ft
Thinly bedded	60 mm - 200 mm	0.20 - 0.65 ft
Very thinly bedded	20 mm - 60 mm	0.06 - 0.20 ft
Laminated	6 mm - 20 mm	0.02 - 0.06 ft
Thinly laminated	<6 mm	<0.02 ft

Discontinuity Frequency
Expressed as the number of discontinuities per metre or discontinuities per foot. Excludes drill-induced fractures and fragmented zone.

Discontinuity Spacing

Term	*Average Spacing*	
Extremely widely spaced	>6 m	>20.00 ft
Very widely spaced	2 mm - 6 m	6.50 - 20.00 ft
Widely spaced	600 mm - 2 mm	2.00 - 6.50 ft
Moderately spaced	200 mm - 600 mm	0.65 - 2.00 ft
Closely spaced	60 mm - 200 mm	0.20 - 0.65 ft
Very closely spaced	6 mm - 60 mm	0.06 - 0.20 ft
Extremely closely spaced	<20 mm	<0.06 ft

Note: Excludes drill-induced fractures and fragmented rock.

Broken Zone
Zone of full diameter core of very low RQD which may include some drill-induced fractures.

Fragmented Zone
Zone where core is less than full diameter and RQD = 0.

Strength

Term	*Description*	*Unconfined Compressive Strength*	
		(MPa)	*(psi)*
Extremely weak rock	Indented by thumbnail.	0.25-1.0	36-145
Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife.	1.0-5.0	145-725
Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.	5.0-25	725-3625
Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer to fracture it.	25-50	3625-7250
Strong rock	Specimen requires more than one blow of geological hammer to fracture it.	50-100	7250-14500
Very strong rock	Specimen requires blows of geological hammer to fracture it.	100-250	14500-36250
Extremely strong rock	Specimen can only be chipped with geological hammer	>250	>36250

Weathering

Term	*Description*
Fresh	No visible sign of rock material weathering.
Faintly weathered	Discoloration on major discontinuity surfaces.
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than in its fresh condition.
Moderately weathered	Less than half of the rock material is decomposed and/or disintegration to a soil. Fresh of discolored rock is present either as a continuous framework or as corestones.
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely	All rock material is decomposed and/or disinteweathered to a soil. The original mass structure is still largely intact.
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment

HOLE: EM BH1
PAGE: 1 **OF:** 4

SITE: Embro Dam

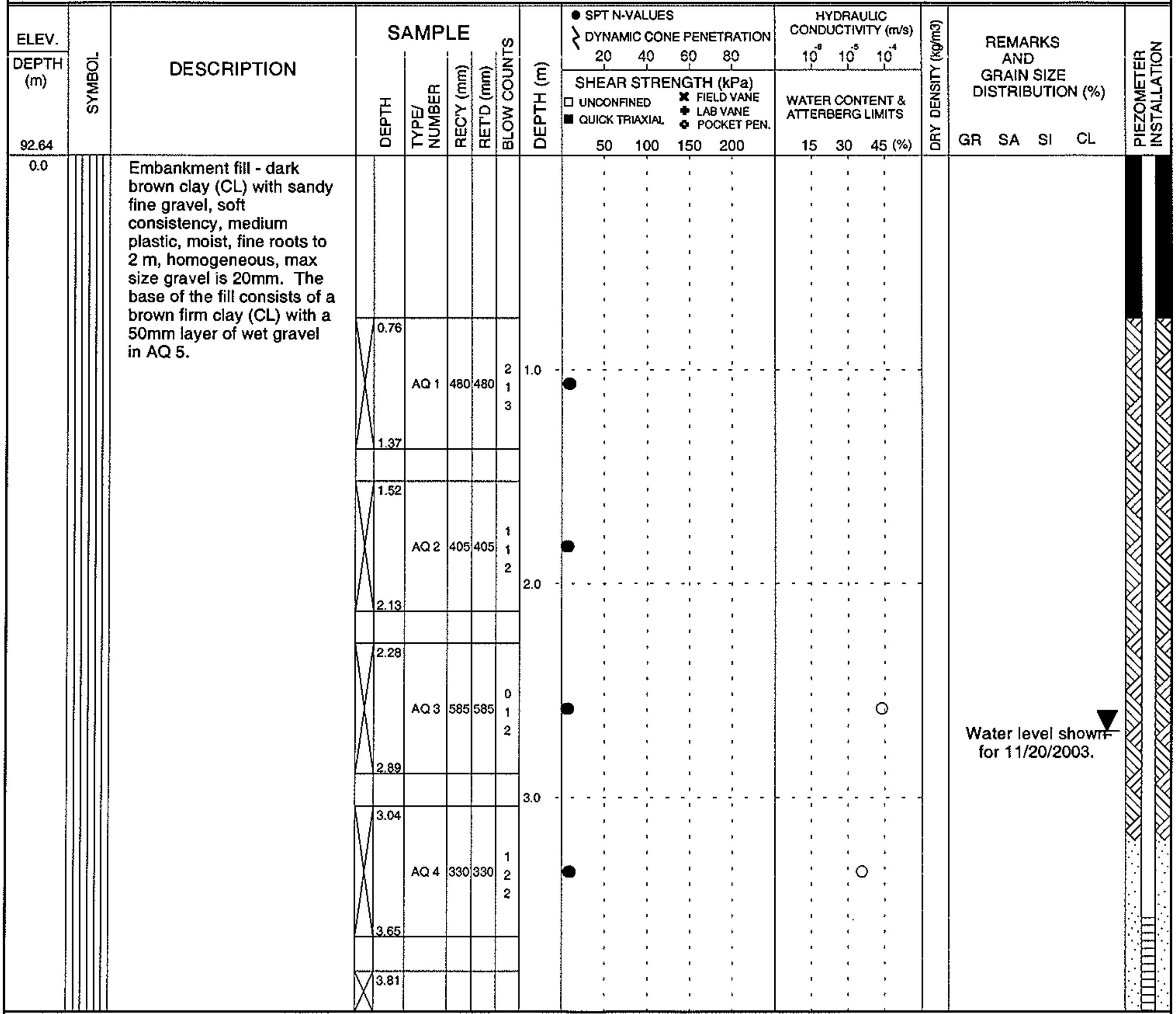
COORDINATES: On dam centerline, 3.5m right of centreline culvert
DIP DIRECTION: 0
DIP: 90

ELEVATIONS
DATUM: Crest assumed elev 100m
PLATFORM:
GROUND: 92.64
END OF HOLE: 85.18

CONTRACTOR: Atcost Soil Drilling Inc.
DRILL TYPE: CME 75
METHOD SOIL: Hollow stem auger
ROCK:
CASING: Auger 200mm OD
CORE:

STARTED: 20 Nov 2003
FINISHED: 20 Nov 2003
INSPECTOR: D. Besaw
LOGGED BY: D. Besaw
REVIEWED: B Sinclair
DATE: 10/3/04

See end of log for detailed groundwater measurements



Embankment fill - dark brown clay (CL) with sandy fine gravel, soft consistency, medium plastic, moist, fine roots to 2 m, homogeneous, max size gravel is 20mm. The base of the fill consists of a brown firm clay (CL) with a 50mm layer of wet gravel in AQ 5.

DEPTH	TYPE/ NUMBER	REC'Y (mm)	RET'D (mm)	BLOW COUNTS
0.76 1.37	AQ 1	480	480	2 1 3
1.52 2.13	AQ 2	405	405	1 1 2
2.28 2.89	AQ 3	585	585	0 1 2
3.04 3.65	AQ 4	330	330	1 2 2
3.81				

Water level shown for 11/20/2003.

SAMPLING METHOD

A - Split Tube
B - Thin Wall Tube
C - Piston Sample
D - Core Barrel
E - Auger
F - Wash
G - Shovel Grab
K - Slotted

SHIPPING CONTAINER

N - Insert
O - Tube
P - Water Content Tin
Q - Jar
R - Cloth Bag
S - Plastic Bag
U - Wooden Box
Y - Core Box
Z - Discarded

PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT
W_P W_N W_L

☐ Constant Head Test
▨ Falling Head Test
▩ Lab. Permeability

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment

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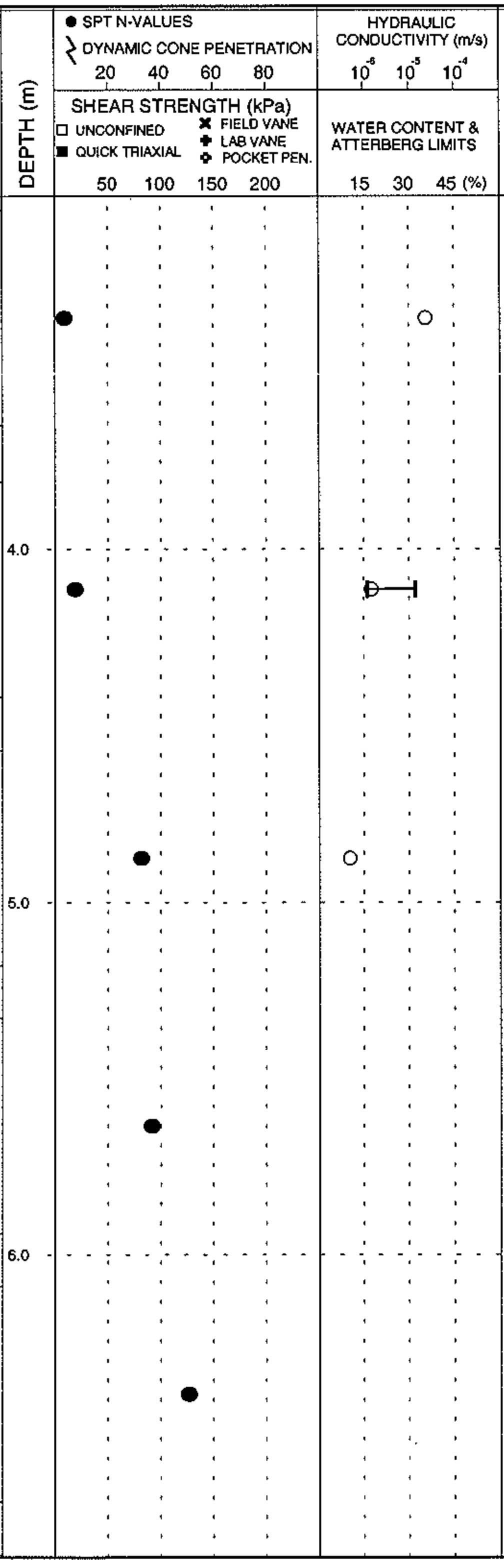
PAGE: 2 **OF:** 4

ELEV. DEPTH (m)	DESCRIPTION	SAMPLE DEPTH	TYPE/ NUMBER	REC'Y (mm)	RET'D (mm)	BLOW COUNTS	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		3.04 3.65	AQ 4	330	330	1 2 2	
		3.81 4.42	AQ 5	405	405	4 4 4	
88.15 4.49	Glacial till (ML) - foundation material, tan colored sandy silt with subrounded fine gravel, maximum size of 10mm, dense to very dense, low plasticity, homogeneous, moist.	4.57 5.18	AQ 6	510	510	7 13 20	See gradation for AQ6.
		5.33 5.94	AQ 7	530	530	11 17 20	
		6.09 6.7	AQ 8	530	530	14 29 32	



DRY DENSITY (kg/m3)

PIEZOMETER INSTALLATION

SAMPLING METHOD

A - Split Tube
B - Thin Wall Tube
C - Piston Sample
D - Core Barrel
E - Auger
F - Wash
G - Shovel Grab
K - Slotted

SHIPPING CONTAINER

N - Insert
O - Tube
P - Water Content Tin
Q - Jar
R - Cloth Bag
S - Plastic Bag
U - Wooden Box
Y - Core Box
Z - Discarded

PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT
W_P W_N W_L

☐ Constant Head Test
▨ Falling Head Test
▩ Lab. Permeability

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BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment

HOLE: EM BH1
PAGE: 3 **OF:** 4

ELEV. DEPTH (m)	SYMBOL	DESCRIPTION	SAMPLE DEPTH	SAMPLE TYPE/ NUMBER	SAMPLE REC'Y (mm)	SAMPLE RET'D (mm)	BLOW COUNTS	DEPTH (m)	● SPT N-VALUES DYNAMIC CONE PENETRATION 20 40 60 80 SHEAR STRENGTH (kPa) □ UNCONFINED ■ QUICK TRIAXIAL ✖ FIELD VANE ✚ LAB VANE ✜ POCKET PEN. 50 100 150 200	HYDRAULIC CONDUCTIVITY (m/s) 10^{-6} 10^{-5} 10^{-4} WATER CONTENT & ATTERBERG LIMITS 15 30 45 (%)	DRY DENSITY (kg/m3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	PIEZOMETER INSTALLATION
			6.85										
								7.0					
				AQ 9	530	530	15 29 40		●				
85.18 7.46			7.46										

END OF BOREHOLE

SAMPLING METHOD

A - Split Tube
B - Thin Wall Tube
C - Piston Sample
D - Core Barrel
E - Auger
F - Wash
G - Shovel Grab
K - Slotted

SHIPPING CONTAINER

N - Insert
O - Tube
P - Water Content Tin
Q - Jar
R - Cloth Bag
S - Plastic Bag
U - Wooden Box
Y - Core Box
Z - Discarded

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W_N — LIQUID LIMIT W_L

☐ Constant Head Test
⬚ Falling Head Test
▨ Lab. Permeability

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment

HOLE: EM BH1
PAGE: 4 **OF:** 4

WATERLEVEL READINGS

11/20/2003 1:00:00 PM	2.71
12/22/2003 3:30:00 PM	2.75

NOTES/COMMENTS

1 **Water level Measurements**

Water level measurements are recorded from ground level.

Reservoir level is +-1.3 m below crest for 2.71 reading and 1.49 m for the 2.75.

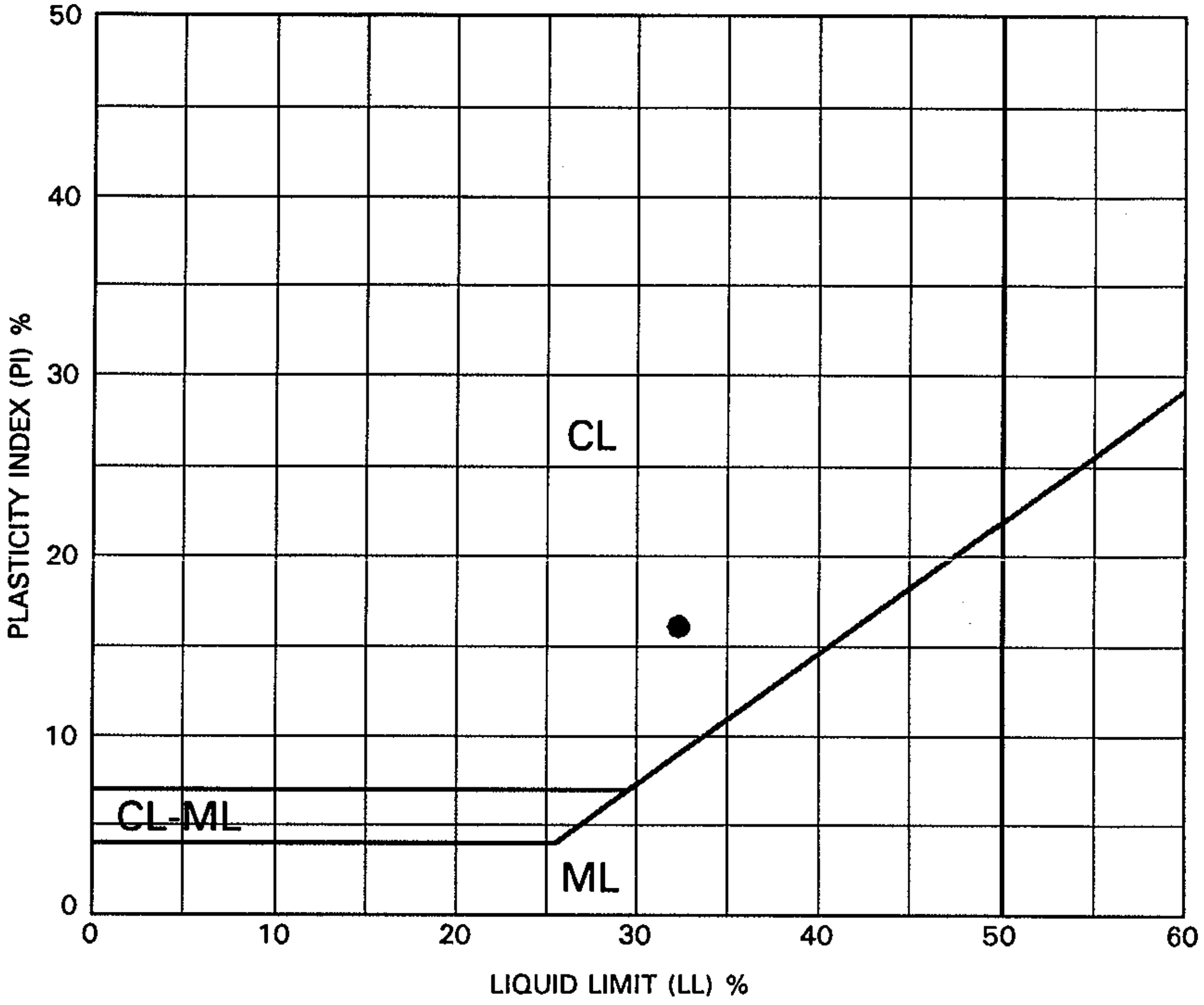
.

2 **Piezometer Installation**

Surface - Flush mount cap embedded in Sakcrete

0-0.76 Bentonite chips
0.76-3.20 Bentonite slurry
3.20-3.56 Coarse sand pack
3.56-4.48 Slottted screen in coarse sand pack
4.48-7.46 Coarse sand pack

Note: - riser and slotted screen consist of 50mm ID rigid, flush-coupled PVC



LEGEND	BH	DEPTH (m)	LL%	PL%	PI%
●	EMBH1	3.81	32	16	16

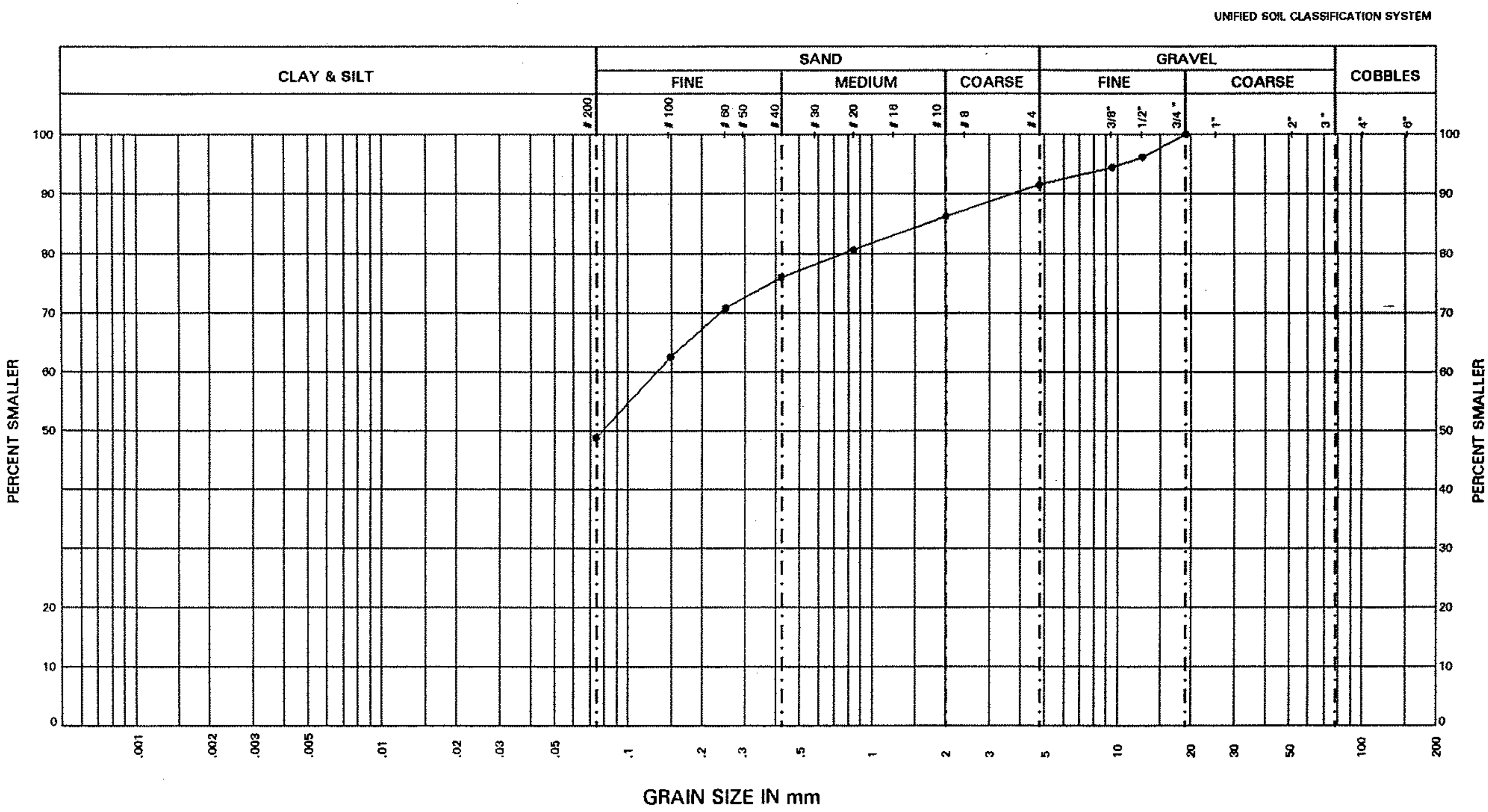
NP = Non-Plastic

Geotechnical Investigations
Upper Thames Conservation Authority

PLASTICITY CHART
Embro Dam

PROJECT NO. P1450404
FIGURE NO.

ACRES



LAB SAMPLE NO.
LAB TEST NO.
DATE December 2003
TESTED BY RS
CHECKED BY AT
APPROVED BY

REMARKS: Embro Dam
UTRCA

GRAIN SIZE DISTRIBUTION

DEPTH	SAMPLE NO.	HOLE NO.	JOB NO.
4.57m	AQ6	BH1	P14504.04

ACRES

6 Hydrotechnical Assessment

6 Hydrotechnical Assessment

6.1 Approach and Methodology

6.1.1 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 250-yr return period floods (regulatory flood), and regional storm (Hurricane Hazel) for the study area shown in Figure 1.3. The design hydrographs were used in the flood routing studies and subsequent dam safety assessment analysis that are described in Sections 6.2 and 6.3.

Two methods were used for estimating peak flows:

- deterministic modeling of watershed runoff on an event basis
- statistical frequency analysis using local historical streamflow data (where streamflow data were available and prorated to the damsite) or regional flood frequency analysis.

The Embro Dam in this study is not located at or near appropriate WSC streamflow gauging stations. Application of transposed or regional runoff flood characteristics for dam safety use requires verification, which can be only accomplished by deterministic modeling means. The regulatory flood adopted by UTRCA for the study basin is frequency-based and has been selected as the 1:250-yr flood. This is approximately equivalent to the historical 1937 flood in the basin.

The deterministic peak flow estimates of the watershed hydrographs were derived using the HEC-HMS rainfall-runoff model (US Army, 2002) for the damsite. The statistical approach made use of the index flood method (MNR, 1986).

6.1.2 Rainfall-Runoff Modeling

6.1.2.1 HEC Hydrologic Modeling System (HEC-HMS)

(a) Rainfall-Runoff Model Selection

The Hydrologic Modeling System (HEC-HMS) is a computer model for precipitation-runoff analysis, developed by the Hydrologic Engineering Center of the US Army Corps of Engineers (US Army, 2002). HEC-HMS supersedes the HEC-1 Flood Hydrograph Package and was selected for application to the individual basins of the study Conservation Area because of its ability to develop discharge hydrographs for hypothetical rainfall events at one or more locations in a basin and its general versatility as an event model. The HEC-HMS model is capable of representing a single runoff event occurring over a period of time, utilizing an appropriate calculation time-step, to accurately compute runoff from the chosen event storm rainfall. The model has a wide variety of options for specifying precipitation, losses, base flow, runoff transformation and the method of routing.

(b) General Description of the Model

The HEC-HMS model is designed to simulate the surface runoff response of a river 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 subbasin. 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. One model may include different versions of a component such as basin models that may be combined with different meteorological data or precipitation events. The result of the modeling process is the computation of streamflow hydrographs at desired locations in the river basin.

(c) Setup of the HEC-HMS Model

The first step in the setup of this model, for application to the individual dam basin, consisted of configuring or schematizing the basin into watersheds/subbasins, channel and reservoir/lake elements (i.e., the hydrologic and hydraulic components). Figure 6.1 shows the discretized drainage area of the Embro site plus the 13 other area study subbasins. The additional basin (Waubuno Creek) used for model calibration is also denoted in this figure. Setup of the HEC-HMS model for application to the dam is described in Section 6.2.

(d) Input Data

Physical parameters for the river basin, including drainage area, stream-course length and average slope, were developed by measurements taken from 1:50 000-scale topographic maps (Department of Energy, Mines and Resources Canada) and 1:10 000-scale Ontario Base Maps (OBMs) from the MNR. The lag time for the river basin is a function of the basin and main stream-course characteristics and was initially estimated by the US Soil Conservation Service (SCS) method (SCS, 1985). More accurate calculations were derived based on a comparison of observed and calculated values for the calibration basin using a formula after Watt/Chow.

The curve number (CN) of the watershed was estimated based on the land-use conditions and soil mapping units prepared by UTRCA for Perth, Oxford and Middlesex Counties, together with their physical soil characteristics (texture and infiltration rates). Sensitivity in the selections of the basin CN and the computed time-to-peak values were evaluated in the calibration runs of the HEC-HMS model. Weighted basin CN values for the antecedent moisture conditions (AMC) I, II and III were then computed for the calibration basin. These computed values and the similarity of the physiographic characteristics between the two basins were then used to establish CN values for the study basin.

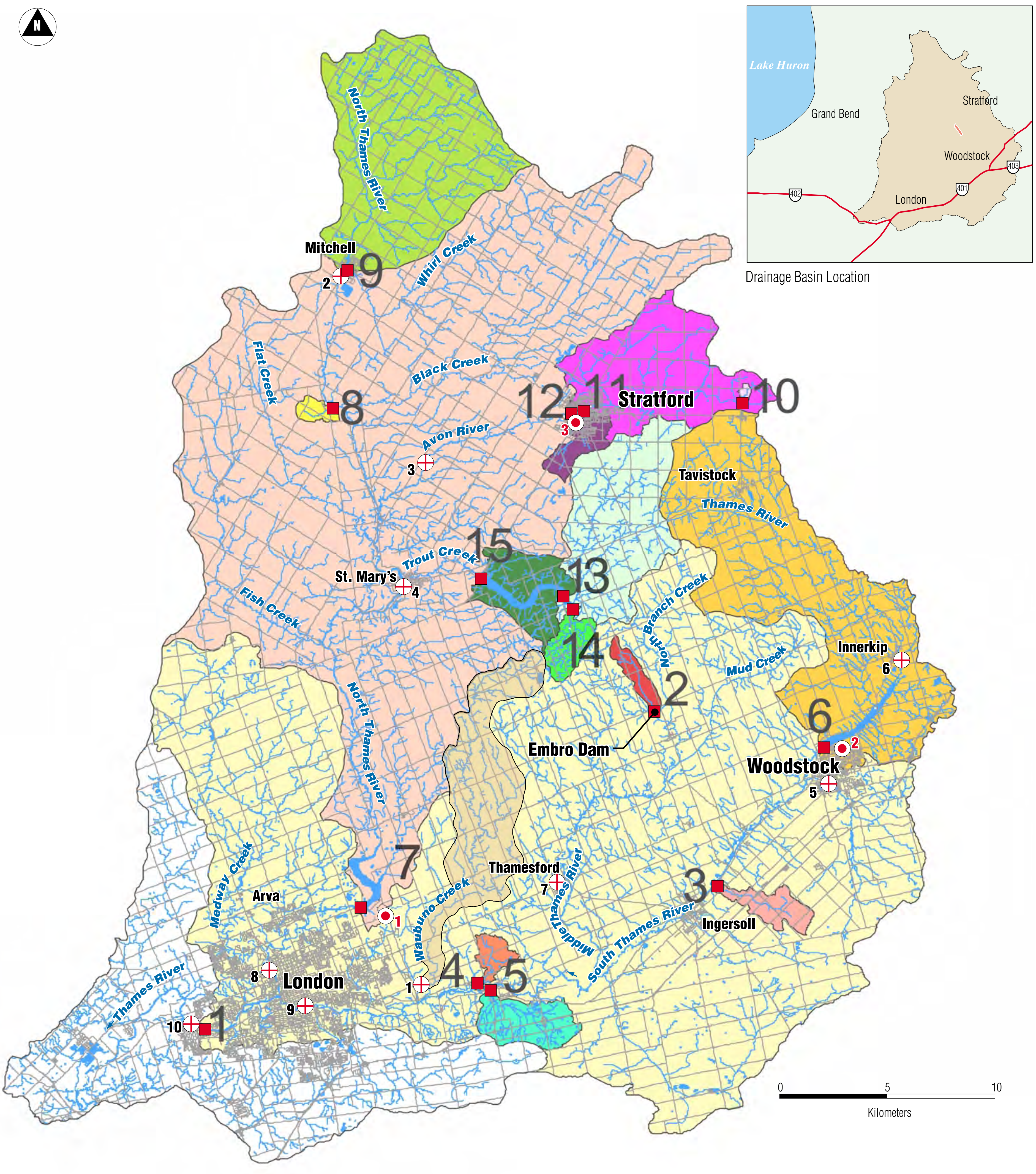
- **Precipitation:** Where more than one precipitation station data were used, the average precipitation over the basin was determined externally by applying Thiessen weighting coefficients and then input to the program. After the storm depth and duration have been established, a representative hyetograph must be selected for input to the model. The temporal distribution of the weighted rainfall in the selected storms represented southern Ontario conditions and was applied to calculate the input storm rainfall distribution or hyetograph.

- **Losses:** The runoff volume for the subbasin was computed by the US SCS CN method with an optional initial loss. This method took into account the hydrologic soil characteristics and AMC. AMC I reflects drier than average soil conditions which can develop if no significant rainfall has fallen for an extended period of time prior to a storm event. AMC II represents the case where soil saturation conditions are average prior to the rainfall event. AMC III represents the situation where significant rainfall occurring prior to the rainstorm has saturated the soil or the ground is partially or completely frozen.

- **Stream Channel Routing:** Routing of hydrographs through channels, where necessary in the HEC-HMS model, was accomplished by the Muskingum-Cunge method. Some of the watersheds in the study have limited reaches where channel routing has marginal attenuation effects, or are dominated by lakes or reservoirs, located immediately upstream of the dams. In these cases, the weighted CN value already accounts for minor channel storage effects of small river reaches and no channel routing was required.

- **Reservoir Routing:** A reservoir storage routing technique was used by the model to route flows through lakes or reservoirs upstream of damsites. Applicable reservoir elevation/outflow relationships were derived from available site information. For those sites with no information, the lake area was determined from topographical map information and a storage volume/elevation relationship developed. Discharge rating curves for the lakes or reservoirs were developed by taking into account the type and physical characteristics of the outlet structure and any bank overflow areas. The discharge rating curves are the input to HEC-HMS, used to compute the reservoir outflows.

- **Base Flow:** Base flow was specified on an individual basis by the following input variables: (a) an initial discharge at the beginning



Legend

🟥 Dam

― Watercourse

― Roads

◉ Meteorological Stations in Basin
1 - London Airport
2 - Woodstock
3 - Stratford

⊕ Gauging Stations
1 - Waubuno Creek near Dorchester (02GD020)
2 - North Thames near Mitchell (02GD014)
3 - Avon River below Stratford (02GD018)
4 - North Thames at St. Mary's (02GD005)
5 - Cedar Creek at Woodstock (02GD011)
6 - Thames River at Innerkip (02GD021)
7 - Middle Thames at Thamesford (02GD004)
8 - Medway River at London (02GD008)
9 - Thames River near Earling (02GD001)
10 - Thames River at Byron (02GE002)

Basins

Springbank (3110 km²)
1 - Springbank
2 - Embro (7 km²)
3 - Centreville (13 km²)
4 - Dorchester CA (7 km²)
5 - Dorchester Mill Pond (23 km²)
6 - Pittock (250 km²)

Fanshawe (1435 km²)
7 - Fanshawe
8 - Fullarton (4 km²)
9 - Mitchell (176 km²)

Avon River Above John St. Weir (107 km²)
10 - Shakespeare (1 km²)
11 - RT Orr (93 km²)
12 - John Street Weir

Wildwood (137 km²)
13 - Ducks Unlimited (92 km²)
14 - Harrington (12 km²)
15 - Wildwood

Waubuno Creek Basin - Calibration Basin

Source: UTRCA GIS Database
Produced by Acres International Limited under licence with the UTRCA, 2003.

Figure 6.1
Upper Thames River Conservation Authority
Dam Safety Assessment Report
UTRCA Dams Watershed Areas-
Location of Dam Drainage Basins

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FIGURE 6.1 – BACK OF PAGE

of the simulation, (b) an exponential recession rate term and (c) a recession threshold discharge for the recession limb of the hydrograph. The base flow component of the storm hydrographs is usually not a significant parameter in relation to the magnitude of the ensuing storm runoff. However, recession discharge is part of the total storm runoff, and characteristics described in Items (b) and (c) above were calibrated in the HEC-HMS analysis.

- **Rainfall Excess To Runoff Transformation:** Precipitation excess was transformed to direct runoff using the unit hydrograph technique. The unit hydrograph adopted was expressed in terms of the SCS unit hydrograph parameters.

(e) Input Rainfall Data

Precipitation data, which is required for input to the event model, is described in Section 6.1.3.

6.1.3 Assessment of Precipitation

Precipitation data are required as the driving input to the HEC-HMS model. These data are required on an event basis (covering at least one day, depending on the size of the watershed) and to provide an appropriate calculation resolution between runoff volume, peak discharge and response time of the various drainage basins.

Floods vary greatly in intensity and duration depending on storm patterns, drainage basin characteristics, and other factors. A summer storm on a small drainage basin may generate a flood with a very high peak flow but of short duration. On a large basin, the peak flow from a similar storm may be significantly attenuated by storage and resistance in the catchment before it reaches the basin outlet. Spring rain-on-snowmelt events, on the other hand, are likely to be of lesser intensity but of much greater areal coverage and longer duration (days). The runoff volume is the dominant factor resulting in flood flows for this type of flood event. Unused storage capacity in a catchment that may be sufficient to attenuate peak runoff and prevent significant flooding from a summer storm of short duration may be ineffective for a severe event of this type. Both types of flood events need to be analyzed to determine the design flood.

Based on the above, two types of design precipitation events were analyzed and used in the study. The first is the summer/fall design storm event (May to November). The other design event is the rain-on-snowmelt conditions. Data from three meteorological stations, Woodstock (Station 6149625), for the period 1871 to 2002, Stratford (Station 6148105), for the period 1959 to 2002 and London A (Station 6144475), covering the period 1940 to 2002 were available for the analysis. To determine the applicable weighting factors, the Thiessen polygon technique was applied based on the location of the study basin relative to the meteorological stations. The results of the weighting analysis indicated that the single station at Woodstock was most representative of the storm events expected for the Embro basin. The data from the Woodstock station was analyzed and applied in the simulation model.

6.1.4 Design Storms and Temporal Distributions

A design storm consists of three important factors: storm volume or depth, duration and temporal distribution. The choice of these parameters would significantly affect the shape and peak value of the resulting runoff hydrograph.

Rainfall Depth-Duration-Frequency Relationship

Rainfall depth-duration-frequency (DDF) or intensity-duration-frequency data are available in the form of tables and graphs from the Atmospheric Environment Service (AES), Environment Canada[*]. AES provides both short duration DDF (from 5 minutes to 24 hours) and long duration DDF (from 1-day to 30-day) design storm depths. The DDF data are based on statistical analysis of long-term rain gauge records in the selected region. Maximum cumulative rainfall amounts for 1-day to 30-day events have been fitted to a modified Gumbel extreme value distribution by AES in their supplied data. Total precipitation for any return period could then be obtained from the fitted distribution.

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[*] The organization Atmospheric Environment Service (AES) is now Meteorological Service Canada (MSC).

Before a design storm can be developed from AES data, two storm parameters must be determined: the duration of the storm and the distribution of the time interval for each rainfall increment in the storm. The duration needed is directly related to the time of concentration of the basin, as determined from an analysis of recorded data or by computation. The duration should be at least as long as, but preferably longer than the time of concentration. A duration less than the time of concentration would not allow all parts of the basin to contribute runoff simultaneously at the outlet during the course of the storm. Runoff from the lower parts of the basin would have left the basin before runoff from the upper parts of the basin had reached the outlet and the estimated peak discharge would be too low. A long duration storm is required to capture the attenuation effects of large natural storage areas.

The 6-hr, 12-hr and 24-hr rainfall durations were, therefore, used in the simulations. The longer durations of 1-day, 2-day and 3-day storms were also analyzed and their results compared to those above.

The time interval of storm increments should be small enough to accurately define the flood hydrograph. The selected time interval of storm increments used in the study varied between 15 minutes and 2 hours, and depended on the storm duration.

The results for rainfall DDF curves for Woodstock are presented in Tables 6.1 and 6.2, respectively.

Table 6.1

AES Rainfall Events for Woodstock Station 6149625 (1962 to 1971)

Return Period (yrs)	Total Precipitation (mm)		
	6-Hr	12-Hr	24-Hr
2	39.5	44.0	50.3
5	52.8	57.9	65.5
10	61.6	67.1	75.6
25	72.7	78.8	88.3
50	81.0	87.4	97.7
100	89.2	96.0	107.1
250	100.1	107.1	119.4

Table 6.2

AES Rainfall Events for Woodstock Station 6149625 (1871 to 2002) for Fall (May to November)

Return Period (yrs)	Total Precipitation (mm)		
	1-Day	2-Day	3-Day
2	50.9	56.4	60.8
5	69.3	75.6	80.8
10	81.5	88.4	94.0
25	96.9	104.4	110.8
50	108.3	116.3	123.2
100	119.7	128.2	135.6
250	134.5	143.6	152.0

Time Distribution

Various types of rainfall distribution curves have been developed for use in hydrograph calculations. The two main categories of rainfall curves comprise statistically derived distributions and the center-peaking distribution or balanced storm. A design storm developed from AES data is sometimes referred to as a ‘balanced’ storm (Chow et al., 1988) because its rainfall curve is symmetrical in appearance and has the most intense portion of the storm located near the center of the storm. This is preceded and followed by periods of much less intense rainfall. This type of rainfall curve is created from the DDF data.

UTRCA has also used a similar type of rainfall curve in their Visual Otthymo, Version 2 (VO2) modeling for the Thames River basin (MMM, 1983; UTRCA, 1995; M. Wood personal communication, 2003). However, the peak intensity occurs at around 30% of the storm duration instead of at the center followed with less intense rainfall over a longer period. This was felt to be more appropriate for the Embro basin and was adopted in the model runs. The comparison shows that for a given identical rainfall depth the computed runoff hydrograph peaks did not vary significantly for these two rainfall curves. These storm distributions are applicable to rainfall and not rainfall plus snowmelt conditions. The VO2-UTRCA storm distribution is plotted in Figure 6.2. Figure 6.3 illustrates the 1:50-yr rainfall hyetographs over a 24-hr duration. Appendix C summarizes the VO2-UTRCA storm distributions in Table C1.

AES DDF curves describe the variation of point rainfall with time for a given frequency. The curves do not include an adjustment for the variation with space and area. When simulations are undertaken for watersheds larger than 25 km^2, an areal reduction to point rainfall is required in accordance with the Technical Guidelines for Floodplain Management in Ontario (MNR, 1986). Since the size of Embro basin drainage area is 7 km^2, it was not necessary to apply an areal reduction factor for watershed rainfall[*].

Rain-On-Snowmelt Event

The DDF data of the rain-plus-snowmelt event were obtained from AES, Environment Canada. These data are derived using AES snowmelt models and the amounts are given in equivalent rainfall (water) amounts. The rain-on-snowmelt design events were derived using daily mean

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[*] Though no areal reduction was necessary, it should be noted that the >25-km^2 threshold is based on MNR guidelines and The World Meteorological Organization (WMO) curves (MNR, 1986; WMO, 1974) parameters not the US National Weather Service curves that are also presented in the MNR guidelines.

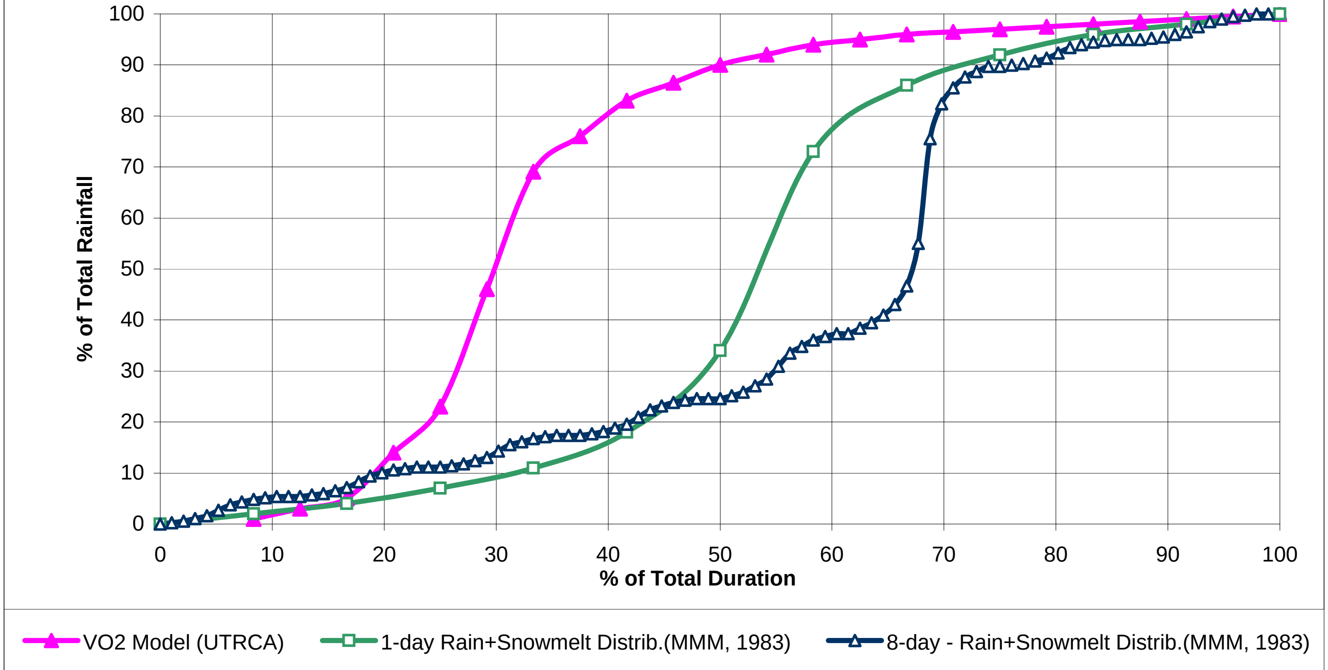
temperatures, daily rainfall total and daily depth of fresh snow measurements by ruler. A snow density of 0.1 was assumed to convert snow depth into its water equivalent. Daily snowmelt estimates were calculated using degree-day type equations. Five different snowmelt models can be used; Model 4, which is suitable for southern Ontario and the Upper Thames River basin, was selected.

UTRCA has used 8-day rainfall-plus-snowmelt distributions at four gauge locations in their Visual Otthymo, Version 2 (VO2) modeling for the Upper Thames River basin (MMM, 1983; UTRCA, 1995; M. Wood personal communication, 2003). The rain-on-snowmelt distribution pattern for Gauge D that covers the drainage area of the South Thames River basin below Woodstock to the confluence with the Middle Thames River and Cedar Creek was selected and used in the analysis of the Embro watershed. Table C2 in Appendix C summarizes the 1-day, 3-day and 8-day rain-plus-snowmelt distributions.

The 1-day, 3-day and 8-day rain-plus-snowmelt depths derived from AES data were applied to the above storm distribution patterns for Gauge D. Because these would be longer duration storms (up to 8 days for the Embro Dam), they are expected to behave differently than the shorter duration storms. The distribution of the 8-day rain-on-snowmelt storm has its high intensity portion placed during the sixth day of the storm. The daily hyetograph follows a sinusoidal pattern while the distribution of the peak day follows a winter rainfall distribution (MMM, 1983).

The rainfall-plus-snowmelt distributions for 1 day and 8 days are denoted in Figure 6.2. Figures 6.4 (a) and 6.4 (b) illustrate the 1:50-yr rainfall-plus-snowmelt hyetographs for a 1-day and 8-day duration, respectively. The rain-plus-snowmelt event DDF data for Woodstock is summarized in Table 6.3.

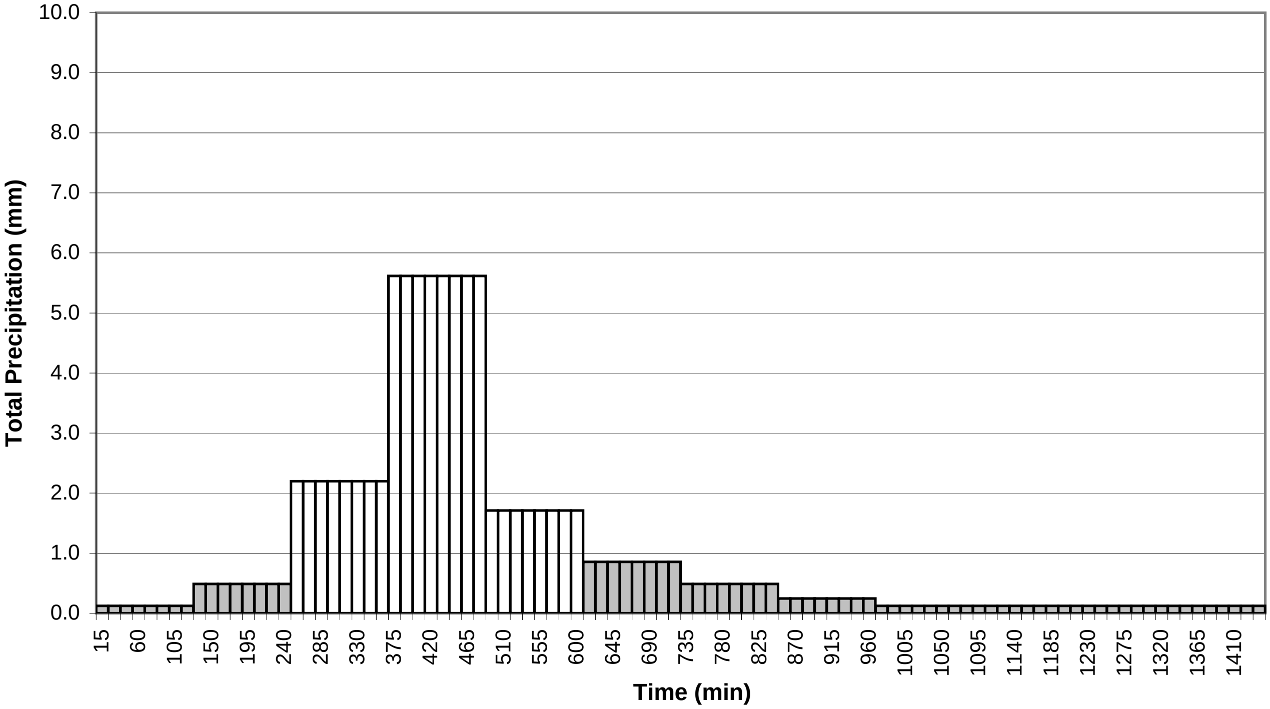
Figure 6.2: Dam Safety Assessment Report - Embro Dam
Comparison of Rainfall and Snowmelt Plus Rainfall Storm Distributions



Note: Rainfall and Rainfall Plus Snowmelt Depths based on the analysis results for Woodstock, Meterological Station (6149625); 1871 - 2002 (1 day, 2 days, etc) and 1962 - 1971 (6 hours, 12 hours, 24 hours)

FIGURE 6.2 – BACK

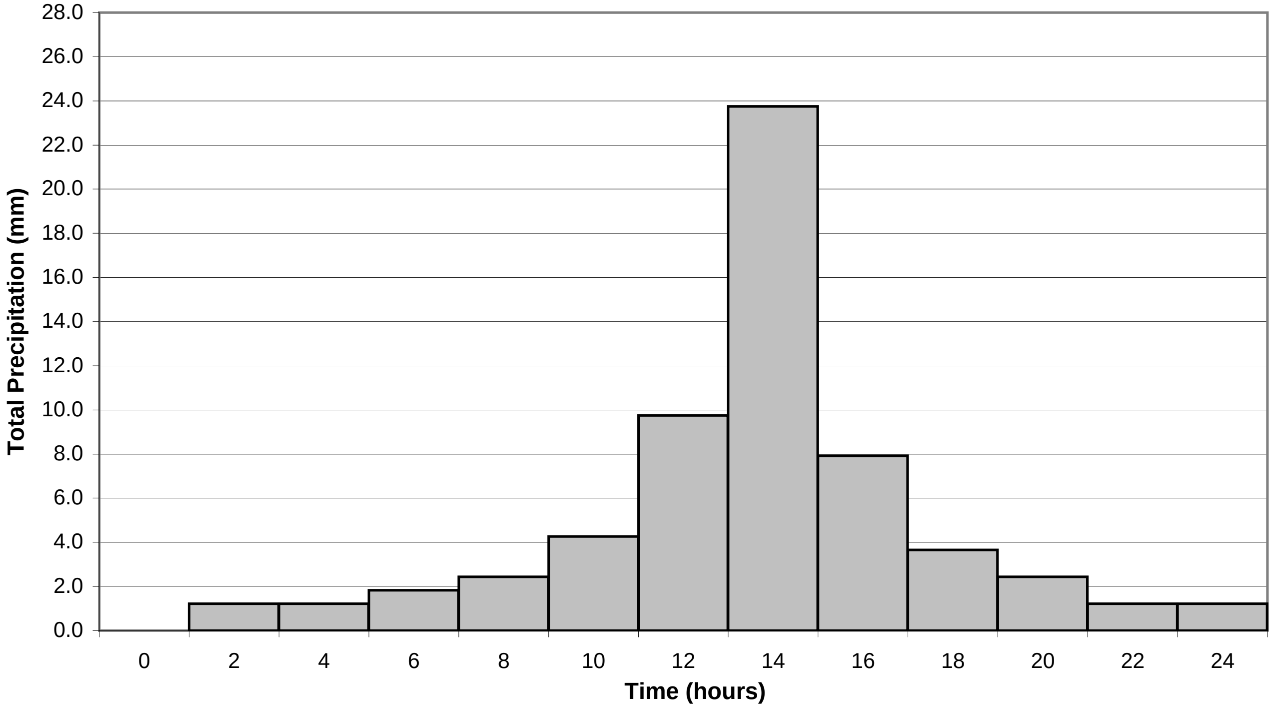
Figure 6.3: Dam Safety Assessment Report - Embro Dam
50-Yr, 24-Hr Rainfall Distribution



Note: Rainfall distribution is based on VO2 model (UTRCA), as shown in Figure 6.2.

FIGURE 6.3 – BACK

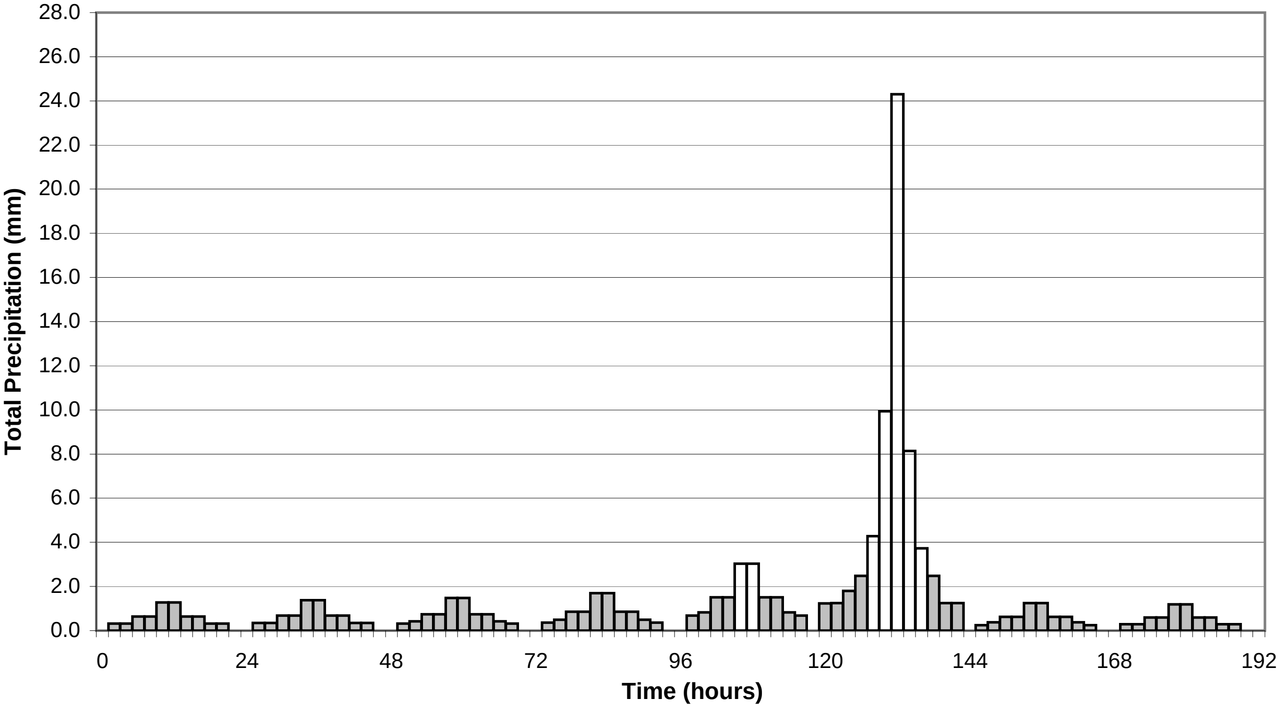
Figure 6.4 (a): Dam Safety Assessment Report - Embro Dam
50-Yr, 1-Day Rainfall Plus Snowmelt Distribution



Note: Rainfall and snowmelt hyetograph is based on MMM 1983 Study, as shown in Figure 6.2.

FIGURE 6.4 (A) – BACK

Figure 6.4 (b): Dam Safety Assessment Report - Embro Dam
50-Yr, 8-Day Rainfall Plus Snowmelt Distribution



Note: Rainfall and snowmelt hyetograph is based on MMM 1983 Study, as shown in Figure 6.2.

FIGURE 6.4 (B) – BACK OF PAGE

Table 6.3

AES Rainfall and Snowmelt Events for Woodstock Station 6149625 (1871 to 2002)

Return Period (yrs)	Total Precipitation (Rainfall and Snowmelt) (mm)							
	1-Day	2-Day	3-Day	4-Day	5-Day	6-Day	7-Day	8-Day
2	27.0	34.9	39.5	43.6	46.9	50.5	54.3	57.2
5	37.8	48.5	54.4	59.1	63.4	68.1	73.0	76.9
10	45.1	57.5	64.2	69.4	74.4	79.7	85.4	89.9
25	54.1	68.8	76.6	82.4	88.2	94.4	101.1	106.4
50	60.9	77.3	85.8	92.0	98.4	105.3	112.7	118.6
100	67.6	85.7	94.9	101.6	108.6	116.1	124.2	130.7
250	76.4	96.7	107.0	114.2	122.1	130.4	139.5	146.8

6.1.4.1 Regional Storm

The regional storm for the study area is the Hurricane Hazel storm based on the Floodplain Management Guidelines (MNR, 1986).

This 12-hr design storm (37 to 48 hours) was developed from rainfall gauge data located at Snelgrove just north of Brampton, Ontario. It is the largest recorded rainfall for any location within 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 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. Towards the end of the storm, 53 mm of rain fell in one hour while 91 mm was recorded during a 2-hr period.

6.1.4.2 Event Modeling

The HEC-HMS model was used to evaluate the Embro basin discharge behavior under a wide range of precipitation events, with return periods of 2, 5, 10, 25, 50, 100 and 250 years. The Hurricane Hazel storm (with appropriate areal reduction factors) was also modeled.

The dam and outlet structure are used to regulate the relatively small storage pond. This makes the volume component of a storm event more important, in comparison to the peak flow generated by the event. It is possible that a precipitation event, with a given return period, may yield different flood flow conditions with the same probability of occurrence depending on the starting water level and discharge facilities' settings, the storm durations, the temporal patterns and intensities of the storms.

6.2 Hydrological/Hydraulic Assessment

6.2.1 Rainfall-Runoff Modeling

6.2.1.1 General

Hydrologic analysis of potential flood events at the damsite included the assessment of regional flood frequency characteristics along with deterministic rainfall-runoff modeling using the HEC-HMS simulation package, as described in Section 6.1.

6.2.1.2 Model Setup

(a) Basin Physiographic and Hydrologic Characteristics
Input to the HEC-HMS model as described previously, consisted of physiographic characteristics for the damsite basin and the storm distribution curve. Physiographic parameters were determined from topographic and soil maps and UTRCA data files. These parameters consisted of catchment area, drainage characteristics, lake area and estimates of live storage and main watercourse slope and length. The storm rainfall data comprised historical storms with their temporal distributions covering the summer/fall seasons. These were obtained from AES.

(b) Calibration of HEC-HMS Model
Successful application of the HEC-HMS model depends on the various derived parameters and relationships specific to the basin or river system. Calibration is ideally performed on the study river system to optimize these parameters and match the model results with recorded data. Since no WSC streamflow stations or UTRCA meteorological

monitoring were located directly on Spring Creek, a representative-gauged, unregulated river which had similar runoff characteristics to the Embro drainage basin was chosen for calibrating the model. The river basin that met these criteria was the Waubuno Creek basin (UTRCA Meteorological Monitoring System station, covering the period 1984 to present). Waubuno Creek basin is between the city of London and the community of Thamesford, and is located west of the study basin. This gauging station was also operated by WSC over the period 1966 to 1999 as Station No. 02GD020.

The calibration procedure in HEC-HMS involves the automatic adjustment of parameters, which affect the transformation of rainfall to runoff in a river catchment in order to achieve a best fit between the simulated flows and the observed discharge in the river. The user may choose to optimize the fit between computed and gauge hydrographs by manipulating any combination of parameters within specified ranges such as the AMC of the basins (the CN value), basin lag time, and initial losses in order that the differences between the simulated and observed flows are reduced to acceptable limits. Hourly rainfall and flow data are available for the Waubuno Creek station for a period of record of 20 years, providing a range of rainfall events and catchment’s responses, which permitted accurate parameter optimization.

For application of the HEC-HMS model to river catchments where storage is present at the damsites, considerable attenuation of the inflow hydrograph can occur. This will result in a reduction of the magnitude of the outflow peak discharge in comparison to the peak of the inflow flood. Therefore, good agreement with storm event runoff volume must be considered in the calibration exercise, as well as reasonable correspondence with peak discharge. IDFs, by convention, are associated with a peak flood magnitude. For application to structures associated with little or no upstream storage, peak inflow is the key parameter to assessing the conveyance capacity. This latter condition applies to the Waubuno watershed in this calibration. Therefore, available hourly rainfall and hourly recorded flows were used for the calibration exercise.

Appropriate storm events were selected from the historical rainfall and streamflow database using the following selection criteria.

- The storm event should be a 24-hr or 1-day event occurring in the summer/fall period of the year, such that transformation of rainfall is accomplished with no snowfall or snowmelt present.
- The storm event should be preceded by at least one week of no rainfall such that average AMCs are present in the drainage basin.

Candidate storm events were selected from a review of the joint databases of hourly rainfall totals and hourly average discharge at Waubuno Creek near Dorchester.

A total of three potential storm events were identified for the study site, and these are presented in Table 6.4. These included one fall and two summer events of which two occurred following relatively dry periods, corresponding to an AMC between I and II. The third event occurred following a relatively wet period, corresponding to an AMC between II and III. It proved very difficult to select individual storm events that entirely met the selection criteria given above.

The storm event that occurred on August 27 and 28, 1992 was selected as a summer event. This storm event took place following antecedent rainfall and corresponded to the ideal 24-hr duration target event. This event was selected for calibration at Waubuno Creek because of the strong observed catchment response to the resulting runoff from the storm.

Another summer event, which occurred on June 11, 2000, was also selected on the basis of strong observed catchment response, although it was under the 24-hr duration criterion. This event took place following a relatively dry period without antecedent rainfall.

The selected fall storm, which occurred on September 29 and 30, 1986, consisted of a large 24-hr rainfall event followed by a well-developed runoff hydrograph at the Waubuno Creek site.

Table 6.4

Storm Event Candidate Data for HEC-HMS Calibration

Date	Hour	Waubuno Creek		Date	Hour	Waubuno Creek		Date	Hour	Waubuno Creek	
		Rainfall (mm)	Flows (m^3/s)			Rainfall (mm)	Flows (m^3/s)			Rainfall (mm)	Flows (m^3/s)
11-Jun-00	1	0.00	0.4	28-Aug-92	1	0.25	0.8	29-Sep-86	1	0.00	1.9
	2	0.00	0.4		2	0.25	0.8		2	0.00	1.9
	3	0.00	0.4		3	2.00	0.8		3	0.00	1.9
	4	0.00	0.4		4	0.75	0.9		4	0.00	1.8
	5	0.00	0.4		5	0.00	1.0		5	6.50	1.8
	6	0.00	0.4		6	1.75	1.0		6	7.50	1.9
	7	0.00	0.4		7	3.00	1.0		7	0.00	2.5
	8	0.00	0.4		8	16.50	1.3		8	1.75	2.5
	9	0.00	0.4		9	24.75	3.7		9	5.75	2.5
	10	0.00	0.4		10	1.25	8.0		10	0.00	2.7
	11	0.25	0.4		11	0.50	8.2		11	2.00	3.1
	12	23.75	0.5		12	0.25	6.6		12	0.25	3.3
	13	10.00	0.8		13	1.25	6.0		13	23.50	4.2
	14	0.00	1.2		14	0.25	6.8		14	1.00	5.7
	15	18.50	1.7		15	0.25	8.7		15	1.50	7.1
	16	2.00	2.1		16	0.00	11.2		16	0.25	8.5
	17	0.50	1.9		17	0.00	13.0		17	4.00	9.3
	18	6.00	1.9		18	0.25	14.2		18	1.00	10.0
	19	19.25	2.9		19	0.00	15.3		19	0.00	10.7
	20	6.75	6.5		20	0.00	16.3		20	30.50	11.4
	21	22.00	18.8		21	0.00	16.9		21	5.50	13.2
	22	0.00	18.3		22	0.00	17.7		22	0.00	15.1
	23	2.00	15.7		23	0.00	18.1		23	0.00	16.5
	24	0.75	16.4		24	0.50	18.3		24	0.00	17.4
12-Jun-00	25	0.00	17.7	29-Aug-92	25	0.25	18.4	30-Sep-86	25	0.00	18.0
	26	0.00	19.3		26	0.00	18.6		26	3.00	18.4
	27	0.00	21.4		27	0.00	18.5		27	6.50	19.1
	28	0.00	23.3		28	0.00	18.6		28	0.25	19.9
	29	0.00	25.1		29	0.25	18.7		29	0.25	20.3
	30	0.00	26.7		30	0.00	18.7		30	0.25	20.5
	31	0.00	28.7		31	0.00	18.6		31	0.00	20.5
	32	0.00	31.2		32	0.25	18.3		32	0.00	20.5
	33	0.00	33.2		33	0.00	17.7		33	0.25	20.6
	34	0.00	34.6		34	0.25	17.0		34	0.00	20.7
	35	0.00	35.1		35	0.00	15.9		35	1.75	20.8
	36	0.00	33.9		36	0.00	14.9		36	0.50	21.3
	37	0.00	33.3		37	0.00	13.9		37	3.75	22.1
	38	0.00	32.3		38	0.25	12.7		38	2.25	23.3
	39	0.00	31.2		39	0.00	11.7		39	1.25	24.6
	40	0.00	30.1		40	0.00	10.8		40	0.50	25.4
	41	0.00	28.7		41	0.00	10.1		41	0.75	25.8
	42	0.00	27.0		42	0.00	9.3		42	2.75	25.8
	43	0.00	24.9		43	0.00	8.8		43	1.25	23.4
	44	5.25	22.7		44	0.00	8.2		44	0.25	22.4
	45	1.00	20.1		45	0.00	7.9		45	0.00	21.5
	46	0.00	17.3		46	0.00	7.5		46	0.75	20.9
	47	0.00	15.0		47	0.00	7.2		47	0.25	20.5
	48	0.25	13.7		48	0.00	6.8		48	0.00	20.0
								1-Oct-86	49	0.00	19.8
									50	0.00	19.5
									51	0.00	19.2
									52	0.00	18.8
									53	0.25	18.0
									54	0.50	17.2
									55	0.00	16.4
									56	0.00	15.5
									57	0.00	14.7
									58	0.00	14.1
									59	0.00	13.5
									60	0.00	12.8
									61	0.00	12.2
									62	0.00	11.6
									63	0.00	11.0
									64	0.00	10.4
									65	0.25	9.8
									66	0.00	9.3
									67	0.00	8.8
									68	0.00	8.3
									69	0.00	8.0
									70	0.50	7.6
									71	0.00	7.3
									72	0.50	7.0

Note:
Source data form UTRCA's HEC-DSS database.

The above-selected storm events formed the basis of the calibration of the HEC-HMS model and concentrated on obtaining an acceptable agreement between the simulated and observed storm event volume and the average hourly recorded flows.

The HEC-HMS model calibration required the following key parameters.

- The contributing river basin drainage area. Verification of the WSC drainage area for Waubuno Creek of 108 km^2 was performed by digitizing the delineated drainage area off 1:50 000-scale topographic mapping.
- The basin response to rainfall, which is referred to as basin lag. This is defined as the elapsed time from the center of mass of the rainfall event to the peak outflow of the runoff hydrograph. This is typically a function of basin area and slope, and can be calculated from a number of empirical equations or determined from storm event analysis. Basin lag for the selected calibration basin was determined from the storm event analysis as discussed in the section below, Waubuno Creek Calibration Results.
- An initial estimate of potential runoff potential of the catchment based on the relationship between CN value and hydrologic soil-cover complexes and soil group designation. This was estimated using the Soil Map of Middlesex County, Ontario (UTRCA). The CN value for AMC II condition was estimated to be 77 for this basin.
- Base flow amounts that reflect the antecedent flow conditions in the river and watershed, prior to a response to the storm event.

(c) Waubuno Creek Calibration Results

Results of the Waubuno Creek basin calibration are presented as follows.

The Waubuno Creek watershed was modeled as a single basin. Catchment parameters are summarized in Table 6.5. The optimization utility in HEC-HMS was used to fine-tune the estimated CN values, initial losses, and the computed basin lag to achieve the best agreement between observed and calculated flood event hydrographs. Initially, basin lag was established using an empirical relationship. However, empirical formulae such as Kirpich's equation severely underestimated basin lag, as observed in the recorded hydrographs.

Table 6.5

Summary of HEC-HMS Input Data and Calibrated Parameters

	Input Data							Calibration						
											Peak Flow		Discharge Volume	
Basin Name	Drainage Area (km^2)	Basin Lag (hrs)	Stream Length (km)	Average Slope (m/m)	Base Flow (m^3/s)	Event Year	Total Rainfall (mm)	Storm Event	Curve Number (CN)	AMC Conditions	Observed (m^3/s)	HEC-HMS (m^3/s)	Observed (mm)	HEC-HMS (mm)
Waubuno Creek near Dorchester	108 [*]	17 [**]	31.0	0.0043	0.38	2000	112	Summer	60.5	I	35.1	37.5	27.1	27.8
					0.60	1992	55	Summer	81.5	II	18.7	20.7	22.6	19.0
					1.80	1986	118	Fall	60.5	I	25.8	27.6	32.4	32.6

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[*] Drainage area from WSC.

[**] Basin lag was calibrated from observed basin rainfall and discharge.

The June 2000 event calibration yielded a CN value of 60.5, a basin lag of 17 hours, and an initial loss of 25 mm. The comparison between observed flow peak and event volume is given in Table 6.5. These results showed good agreement; the antecedent flows in the river prior to this event represent normal summer flow conditions and antecedent rainfall was low, suggesting a low AMC condition between I and II.

The August 1992 storm event occurred during a wetter than usual period with antecedent flows in Waubuno Creek well above the long-term average flow for this time of year, and was used for verification of the basin lag parameter. The calibration yielded a CN value of 81.5 and initial losses of 30 mm. This event yielded good agreement between simulated and recorded flows, with the basin lag of 17 hours obtained from the calibration of the June 2000 event, as shown in Table 6.5.

The fall event of September 1986 followed a very dry month of August. Calibration using the basin lag of 17 hours yielded a CN value of 60.5 and an initial loss of 30 mm, consistent with antecedent soil moisture conditions.

Results of the calibration are presented in Figures 6.5, 6.6 and 6.7. The HEC-HMS computed hourly outflow has been plotted beside the recorded flow for the storm events to provide a graphical comparison.

Generally, the storm volumes agreed quite well. There were some variations between the hourly average flows from the computed output hydrographs and the recorded flows. The computed peak discharges of the three storms used in the calibrations were within 7% and 10.4% of the recorded peaks while the computed runoff volumes were within 2.6% of the measured runoff volumes. In one event, this variation was about 15% for a low value input rainfall hyetograph. The CN value of 77 for AMC II conditions is within the expected values based on the soil and land-use cover in the area. It is, therefore, concluded that the calibration of the Waubuno Creek catchment falls within acceptable limits and can be appropriately applied to the Embro basin.

(d) Storm Event Precipitation

Summer/fall storm rainfall amounts corresponding to the Woodstock station for the shorter durations (6 hours, 12 hours and 24 hours) and the longer durations (1 day and 2 days) were used in the HEC-HMS model. Summer/fall rainfall storms for the required frequencies are summarized in Tables 6.1 and 6.2. Spring snowmelt-plus-rainfall events for 1-day, 3-day and 8-day durations at Woodstock were also used in the HEC-HMS model and are summarized in Table 6.3, over the required range of frequencies. Each precipitation event was assumed to apply individually and entirely on the study basin and no area reduction factor was applied to the point rainfall amounts.

For the summer/fall storm event analysis, average AMC II CN conditions were adopted. Spring snowmelt-plus-rainfall events were initially evaluated under AMC III CN conditions to account for ground conditions being partially or completely frozen.

(e) Regional Flood (Hurricane Hazel)

Although the IHP of the Embro Dam is classified as VERY LOW with the corresponding 50-yr flood assigned as the IDF, the regional flood was routed through the watershed as required in the terms of reference of this study. The regional flood designation for the study dam, which

Figure 6.5: Dam Safety Assessment Report - Embro Dam
HEC-HMS Model Event Calibration Waubuno Creek
Period 11-June-2000 to 13-June-2000

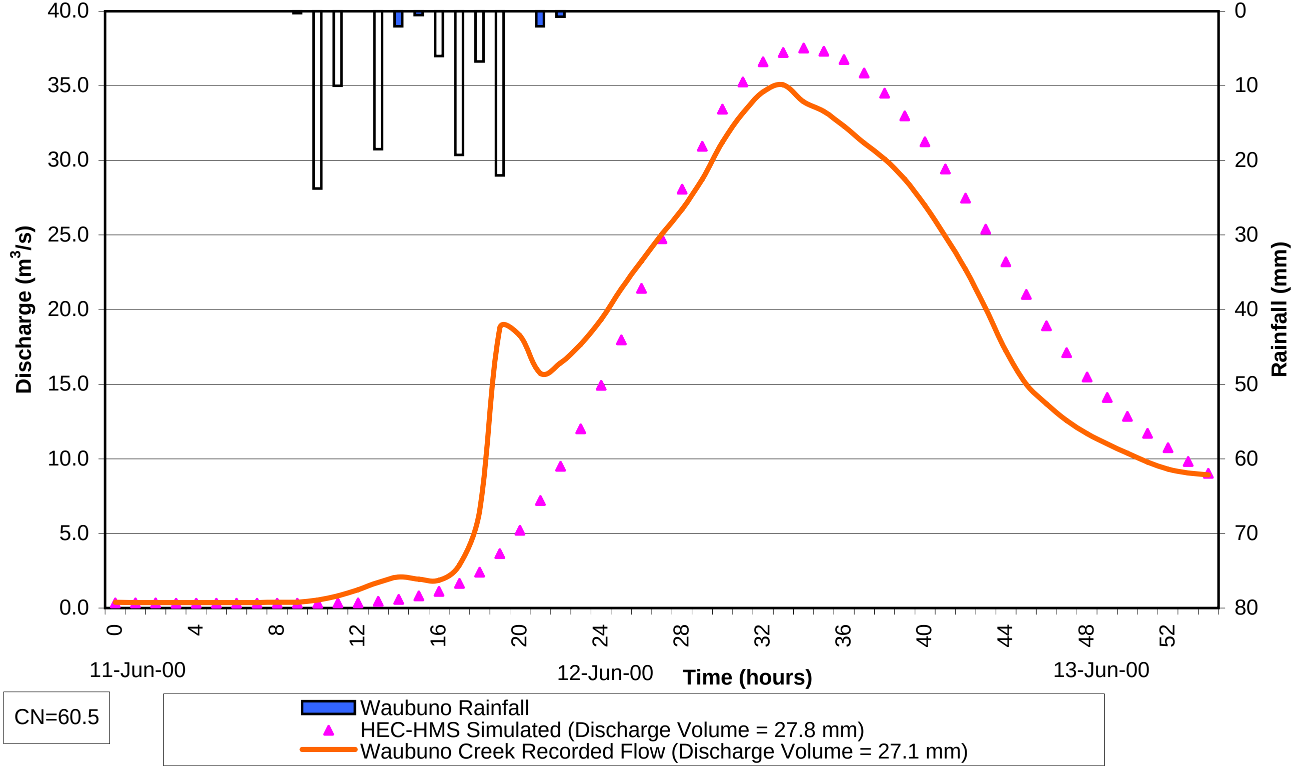

FIGURE 6.5 – BACK

Figure 6.6: Dam Safety Assessment Report - Embro Dam
HEC-HMS Model Event Calibration Waubuno Creek
Period 28-August-1992 to 30-August-1992

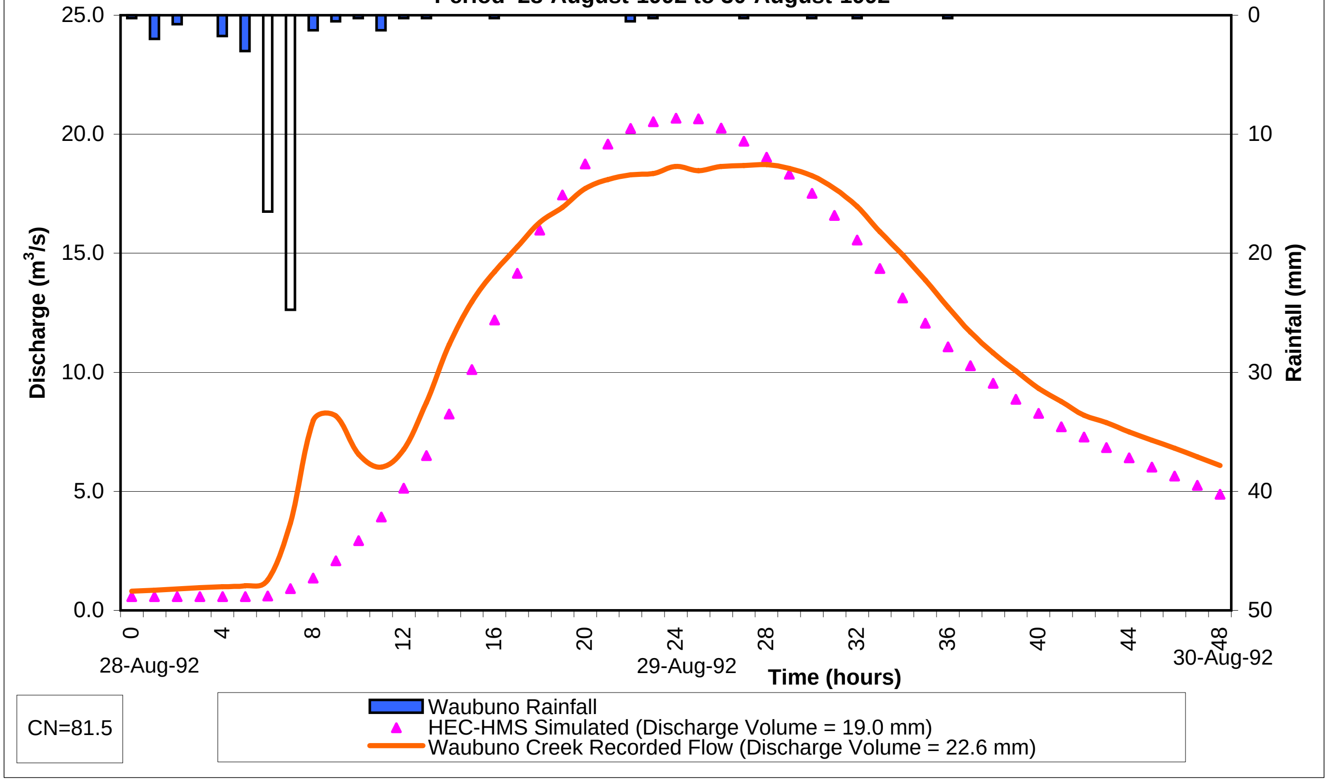

FIGURE 6.6 – BACK

Figure 6.7: Dam Safety Assessment Report - Embro Dam
HEC-HMS Model Event Calibration Waubuno Creek
Period 29-September-1986 to 01-October-1986

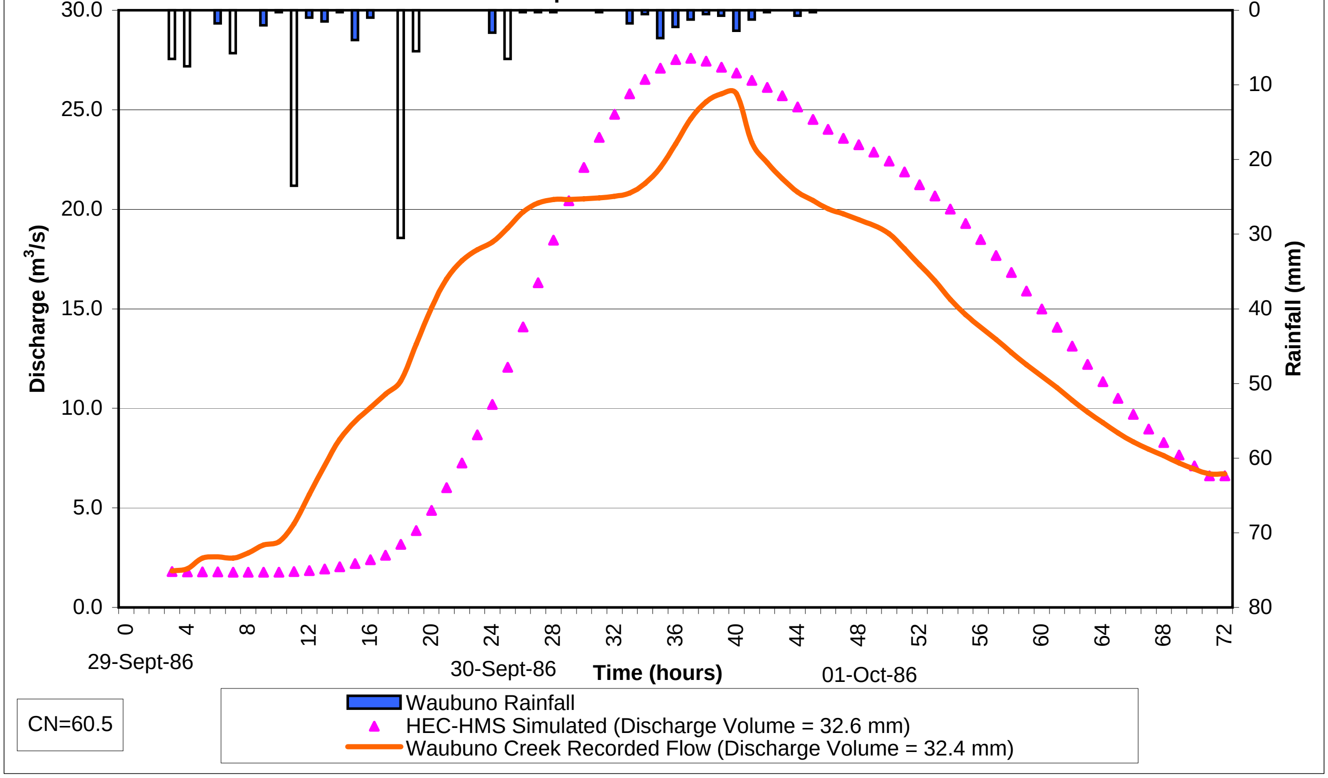

FIGURE 6.7 – BACK

lies within the Regional Storm - Zone 1 is the Hurricane Hazel storm. This storm is a 12-hr summer precipitation event with temporal distribution documented in the MNR guidelines. Areal reduction of the total event precipitation, which is dependent on the size of the study drainage basin, was derived by applying either the circular area-watershed length method or the elliptical area technique. The drainage area of Embro Dam is smaller than 25 km^2; therefore, no areal reduction was required.

AMC III CN conditions were applied to account for ground conditions being saturated at the beginning of the regional storm.

(f) Site Datum

UTRCA provided Acres with a drawing of Embro Dam, which was part of the Dam Hazard Identification studies completed in July 2001. The elevations given on this drawing are to a local datum. The same datum was used in the present analyses.

(g) Model Setup and Initial Conditions – Study Basin

The HEC-HMS model was set up for Embro Dam to allow transformation of storm precipitation into runoff. The watershed was modeled as a single basin. The contributing drainage area, along with basin parameters pertaining to watercourse length and slope were determined from both 1:50 000-scale topographical maps and 1:10 000 OBMs of the catchment. These values are summarized in Table 6.6.

Table 6.6

Summary of HEC-HMS Input Data for Embro Dam

Watershed	Local Drainage Area (km^2)	Total Drainage Area (km^2)	Pond Area (km^2)	Basin Lag[*] (hrs)	Curve Numbers (CN) II	Curve Numbers (CN) III[**]	Stream Length (km)	Average Slope (m/m)	Storm Event	Base Flow (m^3/s)
Spring Creek, A Tributary of the North Branch Creek	7.0	7.0	0.008	5.1	69	84	5.9	0.0033	Spring Fall	0.21 0.02

---

[*] Basin lag was computed based on the following formula after Watt/Chow. Reference: Canadian Flood Hydrology, 1995.
Basin lag = C1 $[L/(S)^{0.5}]^{C2}$ where,
L is the length of the flow path from basin divide to the outlet in metres
S is the average channel slope in metres per metre
C1 = 0.000559, C2 = 0.790; C1 and C2 are calibrated constants to yield basin lag in hours.

[**] Reference: National Engineering Handbook. NEH 4 Hydrology. Soil Conservation Service. March 1985.

Basin lag was computed for the study basin based on the adjusted Watt/Chow empirical relationship and adopted values are given in Table 6.6. CNs were assigned to the basin based on the HEC-HMS calibration results and corresponded to the antecedent conditions specified for the storm event being analyzed. Assigned values are summarized in Table 6.6. Weighted CN values were determined based on an assessment of hydrologic soil-cover complexes and soil group designation. These were estimated from the Soil Map of Oxford County, Ontario and mapping units' properties supplied by UTRCA.

Initial base flow in the study basin was set in accordance with average runoff conditions for the time of year during which the storm event was most likely to occur. For the spring storm events, which typically occur in early April, long-term average monthly discharge for March was adopted. The averaged discharge of August and September was used for the fall storm events. For the study basin, the average flow value for the Waubuno Creek gauged catchment was reduced to a specific runoff (cubic metres per second per square kilometres), then applied to the study basin. Adopted base flow values for Embro are summarized in Table 6.6.

The elevation-volume relationship for the Embro pond upstream storage was derived and used in the HEC-HMS modeling. The discharge capacity of the dam was input to the HEC-HMS model as an elevation-discharge rating curve. The stop logs are removed at the Embro Dam in the fall and replaced in the summer in accordance with the structure operation guidelines submitted by UTRCA to Acres (see Appendix E – Dam Operator Questionnaire). Any additional spillway capacity at the dam, such as the emergency spillway and the flow over the embankment dam, were factored into the rating curves. A description of the discharge facilities and storage relationship for the site is given in Section 6.2.2. Initial water levels that corresponded to the base flow discharges for both the spring and summer/fall events are given in Table 6.7.

Table 6.7

Initial Water Levels for HEC-HMS Analysis

Dam Name	**Name of Pond**	**Spring**		**Summer/Fall**	
		Level (m)	**Stop Log Settings**[*]	**Level (m)**	**Stop Log Settings**[*]
Embro	Embro	98.57	3 logs out	98.83	3 logs in

Note: All elevations referred to a local datum.

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[*] Sill elevation of inlet structure = 98.36 m (6-in. stop logs).

6.2.1.3 Model Flood Results

(a) Storm Event Flood Results

The results of the HEC-HMS simulations are presented in Tables 6.8 (a) and 6.8 (b). The storm rainfall return period values, the corresponding total precipitation and the peak inflows and outflows resulting from the rainfall transformation are summarized in these tables. The resulting peak water levels at the damsite, as the routed

Table 6.8 (a)

HEC-HMS Simulation Results for Spring Creek, A Tributary of the North Branch Creek Subbasin

1-Day (Rain-on-Snowmelt-AMC III)	Spring	2	27.0	1.1	1.1	98.98
		5	37.8	2.3	2.2	99.16
		10	45.1	3.2	2.7	99.54
		25	54.1	4.5	4.5	99.75
		50	60.9	5.5	5.5	99.82
		100	67.6	6.6	6.6	99.86
		250	76.4	8.1	8.0	99.92
3-Day (Rain-on-Snowmelt-AMC III)	Spring	2	39.5	2.4	2.3	99.18
		5	54.4	4.2	4.1	99.73
		10	64.2	5.4	5.4	99.81
		25	76.6	7.1	7.1	99.88
		50	85.8	8.3	8.3	99.93
		100	94.9	9.6	9.6	99.97
		250	107.0	11.3	11.3	99.99
8-Day (Rain-on-Snowmelt-AMC III)	Spring	2	57.2	3.3	2.9	99.60
		5	76.9	5.2	5.2	99.80
		10	89.9	6.5	6.5	99.86
		25	106.4	8.1	8.1	99.92
		50	118.6	9.4	9.4	🟧
		100	130.7	10.6	10.6	99.98
		250	146.8	12.2	12.2	100.00

Notes:
All elevations referred to a local datum.
Average crest elevation of earth-fill embankment = 100.05 m; lowest crest elevation = 99.95 m.

🟧 IDF water level

Table 6.8 (b)

HEC-HMS Simulation Results for Spring Creek, A Tributary of the North Branch Creek Subbasin

Event Duration	Event Timing	Storm Return Period (yrs)	Embro Dam			
			Total Precipitation (mm)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak Water Level (m)
6-Hr Rainfall (AMC II)	Summer	2	39.5	0.6	0.1	98.83
		5	52.8	1.7	1.5	99.15
		10	61.6	2.6	2.4	99.31
		25	72.7	4.0	3.8	99.70
		50	81.0	5.2	5.2	99.80
		100	89.2	6.5	6.5	99.86
		250	100.1	8.3	8.3	99.93
12-Hr Rainfall (AMC II)	Summer	2	44.0	0.8	0.4	98.96
		5	57.9	1.9	1.8	99.20
		10	67.1	2.9	2.6	99.39
		25	78.8	4.3	4.2	99.74
		50	87.4	5.5	5.5	99.81
		100	96.0	6.8	6.8	99.87
		250	107.1	8.5	8.5	99.94
24-Hr Rainfall (AMC II)	Summer	2	50.3	1.0	0.8	99.04
		5	65.5	2.2	2.1	99.25
		10	75.6	3.2	2.8	99.56
		25	88.3	4.6	4.6	99.76
		50	97.7	5.8	5.8	99.83
		100	107.1	7.0	7.0	99.88
		250	119.4	8.7	8.7	99.94
(AMC III)		Hazel	211.1	35.9	35.9	100.18
1-Day Rainfall (AMC II)	Summer (May to November)	2	50.9	1.0	0.9	99.05
		5	69.3	2.5	2.4	99.31
		10	81.5	3.8	3.7	99.69
		25	96.9	5.7	5.7	99.82
		50	108.3	7.2	7.2	99.89
		100	119.7	8.8	8.8	99.95
		250	134.5	11.0	11.0	99.98
2-Day Rainfall (AMC II)	Summer (May to November)	2	56.4	1.0	0.9	99.06
		5	75.6	2.3	2.3	99.28
		10	88.4	3.4	3.2	99.65
		25	104.4	5.0	5.0	99.78
		50	116.3	6.2	6.2	99.85
		100	128.2	7.6	7.6	99.90
		250	143.6	9.4	9.4	99.96

Notes:
All elevations referred to a local datum.
Average crest elevation of earth-fill embankment = 100.05 m; lowest crest elevation = 99.95 m.

floods pass through the outlet structures, are also included in these tables.

(b) Comparison of HEC-HMS Floods With Regional Flood Estimates

The deterministic flood estimates from the HEC-HMS analysis for the ungauged river basin can be compared with regional flood estimates. The regional analysis consists of an examination of flood frequency characteristics for the basin using the Index Flood Method, as outlined in Appendix 5, MNR Technical Guidelines (MNR, 1986)*. The study dam is located in Region 4, as defined in the Technical Guidelines. The index flood or 2-yr flood can be computed as a function of the drainage area of the damsite. Regional flood indices are then applied to the 2-yr flood to estimate floods of greater return periods. The regional flood estimates are summarized in Table 6.9 for the Embro Dam. It is cautioned that while the parameters used for the regional analysis are based on historical flow data from a number of area gauging stations, the range of drainage basin sizes and their degree of regulation are very variable.

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* The primary reference is Moin and Shaw, “Regional Flood Frequency Analysis for Ontario Streams”, 1986.

Table 6.9

Summary of Flood Regional Frequency Analysis Region 4 – Southcentral Ontario[*]

Return Period (yrs)	Regional Index Flood[*]	Flood Peak (m^3/s)
		Embro Subbasin
2	1.00	3.7
5	1.32	4.8
10	1.57	5.7
20	1.80	6.6
50	2.13	7.8
100	2.37	8.7
200	2.60	9.5
500	2.92	10.7
Drainage Area (km^2)		7.0
Regional Q2y		3.7
Unit Runoff for the 1:2-yr flood ($m^3/s/km^2$)		0.5221

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[*] MNR Technical Guidelines.

The results of the computed peak floods are compared to those from the Index Flood Method in Figure 6.8. Generally, the comparison shows that computed floods from the 3-day and 8-day spring snowmelt storms for the 2-yr return period are lower than the corresponding regional index flood estimates. However, this situation reverses for computed 8-day spring floods with return periods equal to and greater than 5 years. The 100-yr, 8-day spring snowmelt storm yields the most severe flood conditions at the dam in terms of water level rise and outflows. In general, the agreement of the 3-day and 8-day spring snowmelt floods do not deviate substantially from those for the Index Flood Method. Because of the inherent variation in drainage basin morphology and degree of both natural and regulated storage, some deviations about the regional estimates are expected.

6.2.2 Hydraulic Analysis

6.2.2.1 Discharge Capabilities

A hydraulic analysis of the Embro damsite was performed to evaluate the existing spillway capacity and to check on tailwater levels. Information obtained during the site visits as well as existing data and reports were reviewed. The present spillway capacity at the site was reviewed using recent structure surveys completed in 2001. The impacts of any upstream or downstream hydraulic conveyance constraints were also evaluated.

The details of the pond impounded behind the Embro Dam were reviewed and an elevation-volume curve was developed using the water surface area of the pond and estimated side slopes.

Tailwater levels were computed from estimated downstream channel geometry assuming normal flow depths at various discharges.

The spillway capacities for the bottom draw inlet structure and the emergency spillway along with the respective reservoir elevation-volume relationship are summarized in Table 6.10. The combined discharge rating curve for Embro Dam is plotted in Figure 6.9.

6.3 Assessment and Confirmation of the Final IHP and IDF Assessment

6.3.1 General

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

Figure 6.8: Dam Safety Assessment Report - Embro Dam
Flood Estimate Using Regional Index Flood versus Deterministic Modeling

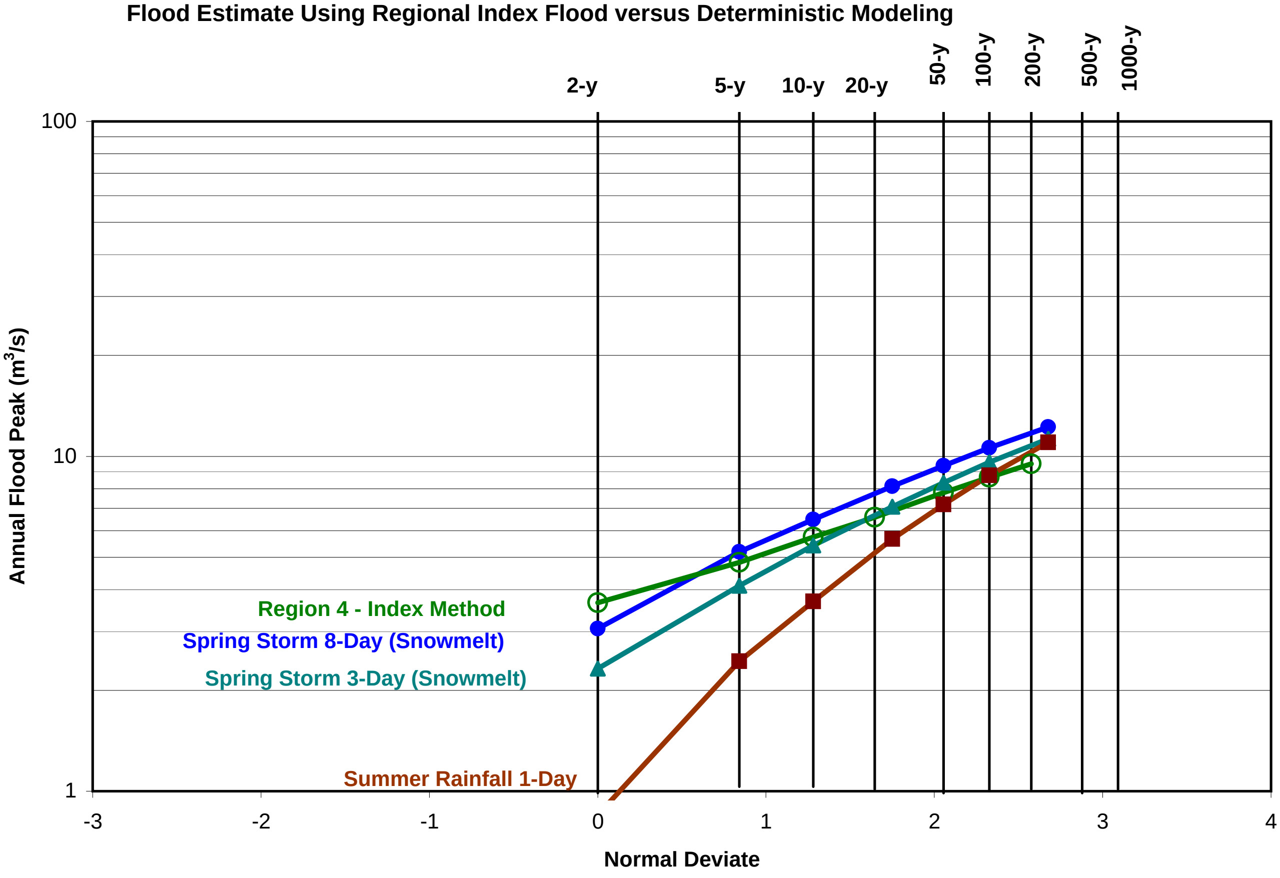

FIGURE 6.8 – BACK OF PAGE

Table 6.10

Embro Dam –
Spillway Capacity and Storage Relationship

Bottom Draw Inlet Structure				Emergency Spillway[*]		Flow Over Embankment Dam[**]		Headpond Storage	
Logs - 0 In		Logs - 3 In							
Elevation (m)	Discharge (m^3/s)	Elevation (m)	Discharge (m^3/s)	Elevation (m)	Discharge (m^3/s)	Elevation (m)	Discharge (m^3/s)	Elevation (m)	Storage (m^{3}x10^6)
98.36	0.00	98.36	0.00	98.36	0.0	98.36	0.0	95.8	0.000
98.44	0.03	98.44	0.00	98.44	0.0	98.44	0.0	98.8	0.014
98.51	0.08	98.51	0.00	98.51	0.0	98.51	0.0	100.8	0.047
98.59	0.15	98.59	0.00	98.59	0.0	98.59	0.0		
98.66	0.22	98.66	0.00	98.66	0.0	98.66	0.0		
98.74	0.31	98.82	0.00	98.82	0.0	98.82	0.0		
98.90	0.66	98.90	0.16	98.90	0.0	98.90	0.0		
98.97	1.01	98.97	0.45	98.97	0.0	98.97	0.0		
99.05	1.43	99.05	0.82	99.05	0.0	99.05	0.0		
99.12	1.92	99.12	1.26	99.12	0.0	99.12	0.0		
99.20	2.46	99.20	1.75	99.20	0.0	99.20	0.0		
99.28	2.58	99.28	2.30	99.28	0.0	99.28	0.0		
99.35	2.62	99.35	2.62	99.35	0.0	99.35	0.0		
99.43	2.66	99.43	2.66	99.43	0.0	99.43	0.0		
99.50	2.70	99.50	2.70	99.50	0.0	99.50	0.0		
99.58	2.74	99.58	2.74	99.58	0.1	99.58	0.0		
99.66	2.78	99.66	2.78	99.66	0.6	99.66	0.0		
99.73	2.81	99.73	2.81	99.73	1.3	99.73	0.0		
99.81	2.85	99.81	2.85	99.81	2.5	99.81	0.0		
99.88	2.88	99.88	2.89	99.88	4.2	99.88	0.0		
99.96	2.92	99.96	2.92	99.96	6.2	99.96	0.0		
100.04	2.96	100.04	2.96	100.04	11.0	100.04	1.7		
100.11	2.99	100.11	2.99	100.11	14.2	100.11	6.9		
100.19	3.02	100.19	3.03	100.19	17.8	100.19	16.1		
100.26	3.06	100.26	3.06	100.26	22.2	100.26	27.1		

Notes:
All elevations referred to a local datum.
Rating curves plotted in Figure 6.9.

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[*] Average crest elevation of emergency spillway is 99.55 m.

[**] Crest elevation of embankment dam varies; mean crest elevation = 100.05 m, lowest crest elevation = 99.95 m.

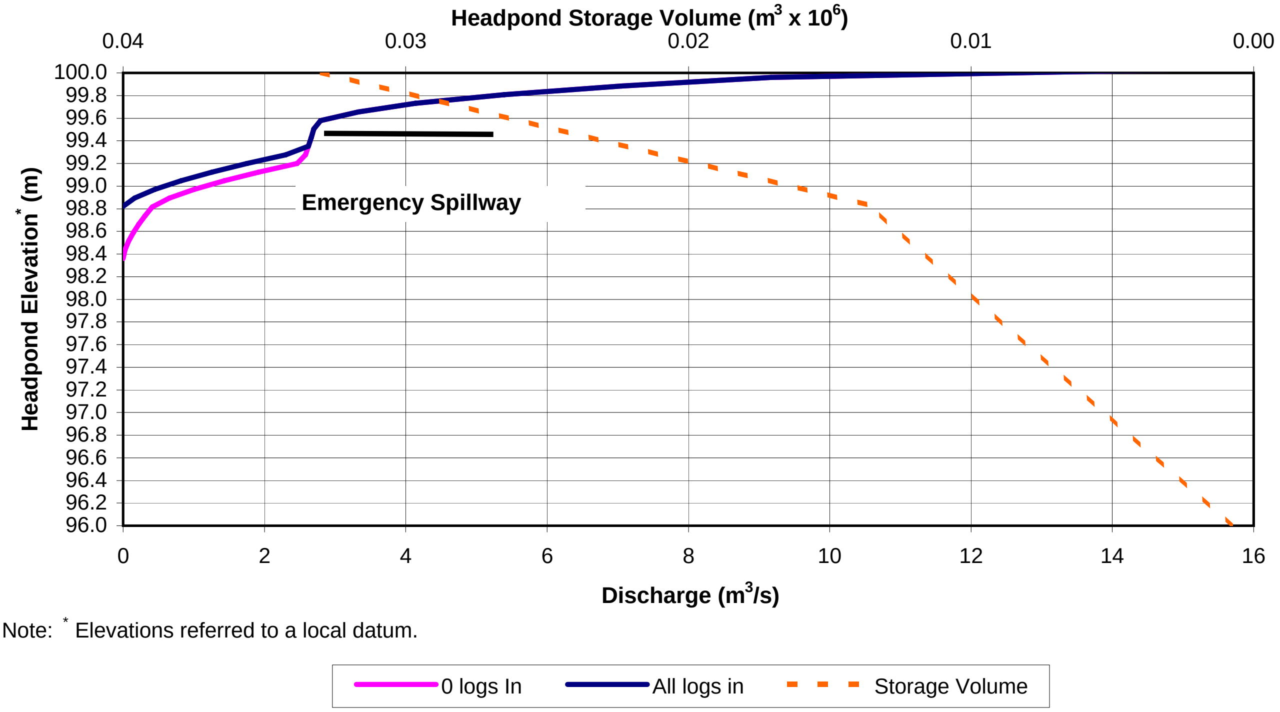
For this study, a preliminary IHP classification at the damsite was initially selected on the basis of available information. The information consisted of the characteristics of the dam, reservoir, watershed, discharge facilities, downstream development and ecology, recreational activities, historical flooding, and supplemental data gained from the site visits. This preliminary IHP was assessed using the selection criteria summarized in Table 1.1, which was then used to determine the IDF for a particular site considering the guidelines presented in Table 1.2.

6.3.2 Embro Dam – Preliminary IHP and IDF

The Embro Dam is located approximately 2 km south of the town of Embro on Spring Creek, a tributary of the North Branch Creek. Embro pond has a surface area of 0.008 km^2 (0.08 ha) and controls a total drainage area of 7 km^2. The primary discharge structure at the dam consists of a bottom draw inlet to a concrete discharge pipe through the dam. Flow releases discharge into Spring Creek below the dam. The creek continues to flow for approximately 1.6 km before entering the main stem of the North Branch Creek. There is an emergency spillway located on the left or east bank, which forms part of the abutment of the dam. The embankment dam is approximately 3.8 m high and impounds a total estimated storage volume of 0.03×10^6 m^3. This classifies the structure as a SMALL dam on the basis of height and a SMALL dam on the basis of storage impounded.

There are no permanent dwellings or development in the immediate downstream reach of the discharge channel. Overall, no potential incremental loss of life under flood conditions is expected. Incremental economic, social and environmental losses are not expected to exceed the VERY LOW category. The dam has, therefore, been designated as a VERY LOW IHP structure and the corresponding IDF lies between 1:25 years to 1:50 years. Based on the assessment of expected economic, social and environmental damages, the 50-yr flood which is the greater of the two floods in this range was selected as the IDF. The salient features of the damsite, along with a summary of the preliminary IHP and IDF classification parameters according to dam height and reservoir volume, are given in Table 6.11.

Figure 6.9: Dam Safety Assessment Report - Embro Dam
Outlet Structure Rating Curve and Reservoir Elevation/Volume Curve
Combined Rating Curve for Bottom Draw Inlet Structure and Emergency Spillway



Note: * Elevations referred to a local datum.

FIGURE 6.9 – BACK

Table 6.11

Preliminary IHP and IDF Classifications for Embro Dam

	Description						Preliminary IHP and IDF					
	Drainage Area						Dam Class		Potential Dam Failure Impacts			
Watercourse	Local (km^2)	Total (km^2)	Reservoir Area (km^2)	Dam Height (m)	Storage (m^{3}x10^6)	Spillway Facilities	By Height	By Storage	Loss of Life	Economic, Social & Environmental	IHP	IDF
Spring Creek, A Tributary of the North Branch Creek	7.0	7.0	0.008	≈ 4.5	0.03	- bottom draw inlet structure - emergency spillway	SMALL	SMALL	None expected	Minor flood damages downstream	VERY LOW	25-yr flood to 50-yr flood

6.3.3 Embro Dam – Final IHP and IDF Assessment

The results of the hydrologic and hydraulic assessments for the study damsite were used to verify the preliminary IHP and IDF classifications in Section 6.3.2. During passage of the 50-yr, 8-day spring snowmelt IDF event, most of the discharge would be conveyed through the emergency overflow spillway.

The inflow flood for the 1:50-yr frequency was estimated at 9.4 m^3/s while the peak outflow was also 9.4 m^3/s due to negligible attenuation by the Embro pond. The dam discharge facilities would not be able to pass this flood without overtopping the main embankment dam by 0.01 m at an upstream water level of 99.96 m.

Based on the above results, the dam does not have adequate spillway capacity to pass the IDF on the basis of three stop logs removed in the bottom draw inlet structure. Presently, the Embro Dam is confirmed as a VERY LOW hazard structure, and the corresponding IDF is the 50-yr, 8-day spring snowmelt event. The final IHP and IDF classifications are presented in Table 6.12.

Table 6.12

Final IHP and IDF Assessments for Embro Dam

Watercourse	Final		Event Duration and Timing	Start W.L. Condition (m)	Inflow (m^3/s)	Outflow (m^3/s)	Maximum Water Level (m)	Change in W.L. from Start W.L. (m)	Tailwater Level (m)
	IHP	IDF							
Spring Creek, A Tributary of the North Branch Creek	VERY LOW	1:50-yr	Spring (snowmelt) 8-day	98.91	9.4	9.4	99.96	1.05	97.1

Note: All elevations referred to a local datum.

6.3.4 Freeboard

Freeboard at the dam was estimated by calculating wind setup, wave height and wave run-up for 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 procedures in the US Army Corps of Engineers Shore Protection Manual (SPM) (US Army, 1984). Wind direction data was reviewed at both London Airport (located 10 km east of the city) from the Canadian Climate Normals, 1951 to 1980, and at UTRCA’s Waubuno station from 1995 to 2001. To obtain conservative estimates of normal freeboard requirements, the effective fetch in the reservoir was calculated with the primary wind direction (north) aligned with the longest fetch length or radial in the vicinity of the dam structure. Wind direction for the spring period was found to be predominantly from the southwest to northwest directions. The fetch lengths were recomputed based on this direction and used to determine effective fetch lengths for the IDF. Since the reservoir is relatively small, no corrections were made from overland to overwater wind speeds.

A Gumbel extreme value extrapolation of the wind frequency data (NRC-CNRC, 1995) for the station at Embro was used to estimate both the 100-yr and 1000-yr wind speeds. Because the reservoir is relatively small, the wave height would have a limited fetch and not be restricted by wind duration. The wind durations at either 104 km/h (100 years) or 127 km/h (1000 years) would both be long enough to establish steady-state wind/wave conditions in the headpond.

The computed effective fetch lengths for both the normal and IDF conditions for the Embro Dam were 0.09 km and 0.08 km, respectively. The effective fetches at the dam center were computed by

$$F_e = \sum X_i \text{ Cos } a_i$$

where,

a_i = the angle between the central radial and radial 'i'
X_i = the projection of radial 'i' on the central radial.

The resulting calculated wind setups were negligible in all 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). The resulting wave heights and wave run-ups for the 100-yr and 1000-yr wind speeds are summarized in Table 6.13.

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

- Under maximum normal headpond water levels and 1000-yr wind condition, normal freeboard requirements at the damsite is given in Table 6.13.
- Under peak IDF water level conditions, minimum freeboard requirements at the damsite has been conservatively established for specified 100-yr wind conditions. The minimum freeboard heights are given in Table 6.13.

These results show that, during passage of the IDF, the dam would be overtopped.

6.3.5 Embro Dam – Proposed Emergency Spillway Modification

Modifications to the existing Embro Dam emergency spillway were outlined in a letter to UTRCA dated February 2, 2004 with accompanying sketches. The grassed emergency spillway was redesigned and realigned to discharge into the existing channel at a point 60 m downstream of the centerline of the

Table 6.13

Freeboard Requirements for Embro Dam

Structure	Abutment Conditions	Crest Elevation (m)	Normal Water Level (m)	1:1000 Wind			Required Normal Freeboard (m)	IDF Water Level (4) (m)	1:100 Wind			Required Minimum Freeboard (m)	Available Freeboard		Remarks
				Design Wave Height (m)	Wind Setup (m)	Wave Run-Up (m)			Design Wave Height (m)	Wind Setup (m)	Wave Run-Up (m)		Excess Crest (1) Normal (m)	Crest (2) IDF (m)	
Embankment Dam	Earth fill	99.95 (5)	98.82	0.32	0.01	0.24 (3)	0.25	99.96	0.27	0.01	0.21 (3)	0.22	0.88	-0.23	Overtopped during IDF

Notes:

Normal freeboard is measured above the normal water level of the reservoir.

Minimum freeboard is measured above the inflow design flood reservoir water level.

All elevations referred to a local datum.

(1) Normal available freeboard = crest elevation - (NWL + 1:1000-yr wind setup + 1:1000-yr wave run-up).

(2) Minimum available freeboard = crest elevation - (IDF + 1:100-yr wind setup + 1:100-yr wave run-up). A negative value indicates overtopping.

(3) Conservatively estimated as the design wave height; waves expected to break before reaching the structure.

(4) Water level based on three logs removed in the bottom draw inlet structure.

(5) Average crest elevation of earth fill embankment = 100.05 m; lowest crest elevation = 99.95 m.

spillway. The recommended width of the spillway entrance should be 10 m at an elevation of 99.36 m (local datum) with the entrance invert and downstream channel curve lined with cable-connected concrete blocks underlaid with a suitable geotextile to prevent the loss of fines from the parent foundation material. The emergency spillway was designed to discharge approximately 4.4 m^3/s or 61% of the routed IDF at Embro Dam. A trapezoidal section spillway channel to convey spill flows to the existing drain was designed with a 5.0-m bed width, 1.5H:1V side slopes and a bed slope of 0.010 to prevent channel erosion by limiting its flow velocity to a permissible value.

Based on a revision of the IDF using the 8-day rain and snowmelt event, the corresponding outflow flood was calculated to be about 31% higher than previously estimated. The portion of the flood to be handled by the emergency spillway increased by almost 48%, and this required revisions to the spillway crest and channel dimensions.

The revised excavated width of the emergency spillway crest should be 11.0 m at an elevation of 99.26 m (local datum). The previously recommended lining details would be the same. The trapezoidal section spillway channel bed width should be increased to 6.50 m from the previous 5.0 m and the channel bed slope limited to 0.008 to prevent channel erosion. The 2-m sloping drop at the end of the spillway channel would be flared horizontally and vertically over a sufficient distance from the channel to the creek to reduce the flow depth and maintain the velocity within the maximum permissible limit.

7 Civil/Structural Assessment

7 Civil/Structural Assessment

The Embro Dam is essentially an earth embankment with a bottom draw inlet structure located in the pond connected to a concrete pipe passing through the dam and an emergency spillway structure constructed on native material on the left bank. These structures do not lend themselves to stability analyses and thus none were performed.

Assessment of the earth structures is covered in Section 8.

8 Geotechnical Assessment

8 Geotechnical Assessment

8.1 Geology

8.1.1 Regional Geology

The Embro Dam is on Spring Creek, a southward-flowing stream which is a tributary to the North Branch Creek. The North Branch Creek flows into the Middle Thames River just south of the dam. The latter and the Thames River to the southeast form the main drainage courses in the area.

The upland terrain is rolling and has a regional relief of about 30 m. The regional physiography has developed as a result of the latest glaciation.

According to government geological mapping (Min. Nor. Dev., 1991; Ont. Div. Mines, 1973), the area is characterized by thick deposits of glacial till which were deposited during the Wisconsin glaciation. Silty to sandy silt till with minor clay content, known as the Tavistock Till, predominates.

Deposits of sand and gravel glaciofluvial outwash and recent streambed alluvium exist along the Middle Thames River and the Thames River. These generally overlie the till. Local ice contact deposits such as drumlins also exist on the upland. Ancient beaches exist to the south.

Limestone bedrock underlies the area, but is only locally exposed.

8.1.2 Site Geology

The site is located in a relatively flat area of cultivated land. Overburden consisting of silt and sand, some limestone fragments and minor clay exists in the dam area and underlies the reservoir. The overburden overlies bedrock. Depth to top of bedrock is unknown.

8.2 Embankment Structure

8.2.1 Cross-Section Geometry

A typical cross section of the embankment is shown on the July 2001 Dam Hazard Identification drawing provided to Acres. This has been assumed to be typical. It is noted, however, that the upstream slope was not surveyed below the reservoir level. It is noted also that the survey showed erosion of the upstream face at the waterline. This erosion is wave-induced and was not previously noted during the site inspection due to the presence of thick reeds and grass along the waterline.

8.2.2 Foundation Preparation and Characteristics

There are no records of dam construction and of the foundation preparation. Based on the log of Borehole EM BH1, the absence of contaminating organics and topsoil suggests that all loose materials were removed prior to placement of the embankment fill.

8.3 Shear Strength Parameters

The embankment fill consists of clay with sandy gravel. Based on information presented in Section 5 and on an empirical correlation between effective angle of friction for clay-rich material and plasticity index (Holtz and Kovacs, 1981), an angle of friction of 31° was selected for the embankment fill. The material comprising the fill was assumed to be of a normally consolidated origin and, therefore, a cohesion of zero was assigned.

The foundation consists of glacial till comprising dense to very dense, sandy silt with gravel. Accordingly, an angle of friction of 38° and no cohesion were selected for the foundation, based on the in situ density and on Acres experience with the shear strength of tills in Ontario.

8.4 Bearing Capacity

The allowable bearing capacity of the glacial till foundation is estimated to be in the order of 600 kPa. The embankment exerts a maximum total pressure of approximately 80 kPa and, hence, the foundation has more than adequate bearing capacity.

8.5 Settlement

Embro Dam exhibited no signs of settlement, indicating no differential vertical movements since construction. Provided the vertical loads are not significantly increased and given the low seismicity potential of the site area, settlement of the embankment fill is not likely to occur in the future. The same applies to the foundation.

8.6 Liquefaction

The soils that comprise the embankment and the foundation are not considered to be liquefiable due to their clayey nature, grain-size, moisture content and liquid limit characteristics (Arumoli et al., 1999). The low seismicity potential in the site area also reduces the risk of liquefaction.

8.7 Seepage and Uplift

The water table in a homogeneous fill dam of this height is normally gently sloping from the reservoir to just above the tailwater. The inspection did not indicate any significant seepage. Very minor seepage may have existed, but may have been obscured by grass vegetation or may not have been evident due to evaporation.

8.8 Instrumentation

The only instrumentation in this dam is the piezometer referred to above. This monitors the phreatic surface. No other instrumentation is recommended.

8.9 Embankment Stability

8.9.1 Location of Sections

The stability of the earth embankment was examined. The section location taken for the stability analyses is through the highest portion of the dam at about its midpoint. At this section, the embankment is about 4.5 m high. Figure 8.1 shows the section used in the stability analysis.

8.9.2 Method of Analysis

Stability analyses were performed according to the limit equilibrium method of slope analysis utilizing the proprietary slope stability software SLOPE/W (GEO-SLOPE International Ltd.). All calculations were based on the effective strength method and analysis was performed according to the Morgenstern-Price method of slices with a half-sine function selected for the interslice force function. Several methods exist to perform slope stability calculations; however, the Morgenstern-Price method was selected since the appropriate factor of safety should be obtained from a slope stability method that satisfies both force and moment equilibrium.

8.9.3 Material Properties

Material properties were assigned based on typical values for these materials or as provided in the literature, since there were no laboratory tests performed to establish the shear strength of the embankment and foundation materials. Table 8.1 describes the properties for the various materials used in the stability analyses.

8.9.4 Phreatic Surface

A graphical method, as described by Craig (1997), was used to establish the phreatic surface and correspondingly the pore pressures within the embankment dam and foundation material. The graphical solution requires the plotting of a basic parabola. Figure 8.2 shows the details of generating the basic parabola for the dam. The phreatic surface is then obtained by applying the prescribed corrections to the basic parabola. Figure 8.2 shows the produced phreatic surface.

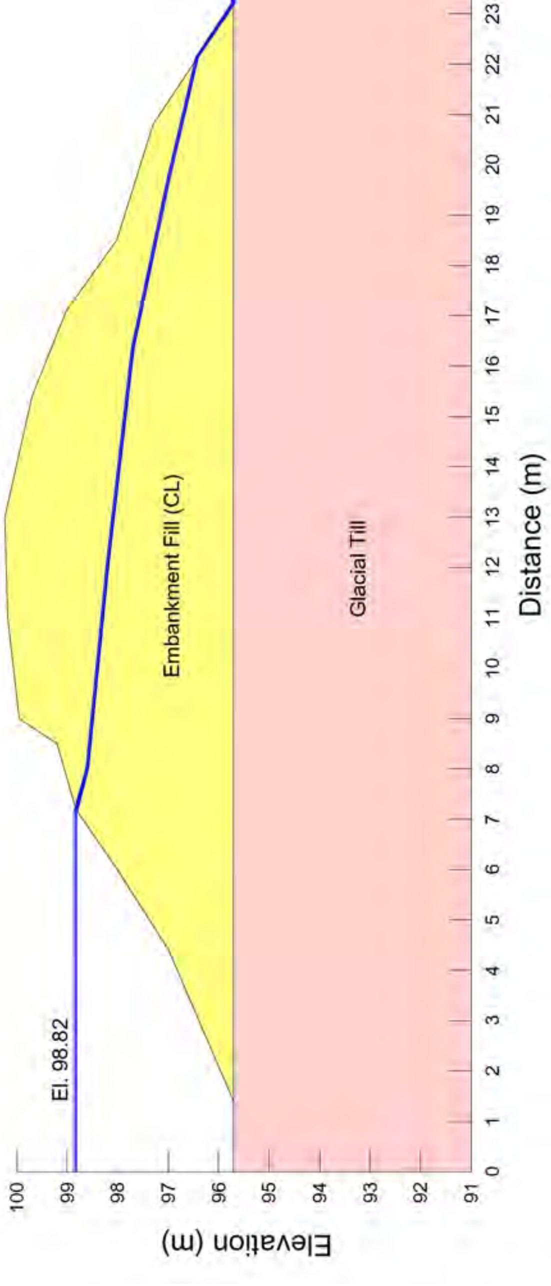


Figure 8.1
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Embro Dam
Stability Section

ACRES International

BACK OF FIGURE 8.1

Table 8.1

Stability Analysis of Earth Embankments

Item	Acceptance Criteria	Calculated	Comments
General			
IHP			
Flood Conditions		Very Low	
IDF		50-yr flood	
Materials			
Embankment			
- embankment fill (CL)			
cohesion (kPa)		0	
ϕ (deg)		31	
moist unit weight (kN/m^3)		17.8	
saturated unit weight (kN/m^3)		19	
Foundation			
- glacial till			
cohesion (kPa)		0	
ϕ (deg)		38	
moist unit weight (kN/m^3)		18.5	
saturated unit weight (kN/m^3)		20.3	
Loads			
Normal water level (NWL)		98.82	
IDF water level		99.96	
Seismic, horizontal (S_h)		0.021*	* 2/3, i.e., 0.014*g*, was used in pseudostatic analyses
Load Combinations			
Upstream Slope			
NWL	1.5	1.24	Does not meet the criteria
Extreme (NWL, S_h)	1.1	1.18	
Extreme (IDF)	1.3	1.39	
Rapid drawdown	1.2	N/A	
Downstream Slope			
Normal (NWL)	1.5	1.16	Does not meet the criteria
Extreme (NWL, S_h)	1.1	1.12	
Extreme (IDF)	1.3	1.16	Does not meet the criteria
Rapid drawdown	N/A	N/A	

The piezometer installed in November 2003 indicates a water level slightly lower than what is determined from a parabolic analysis (Craig, 1997). This is possibly due to the relatively long length of equalization time required for the clay-rich embankment fill.

8.9.5 Seismic Parameters

The draft ODSG requires that dams withstand ground motions associated with a MDE. The MDE is selected based on the hazard potential classification and consequences of dam failure. In the case of the Embro Dam, an earthquake event with 1:100-yr return period was selected as the design load case for stability assessment. This selection was on the basis that the dam has a VERY LOW IHP classification.

Probabilistic earthquake parameters for the damsite, up to 1:1000-yr return period, were established based on data obtained from the Geological Society of Canada, and are shown in Table 8.2. The horizontal peak ground acceleration (PGA) is 0.021 for the 1:100-yr return period.

Table 8.2

Probabilistic Earthquake Parameters

Probability of Exceedance per Year	**0.010**	**0.005**	**0.0021**	**0.001**
Peak horizontal ground acceleration (*g*)	0.021	0.029	0.040	0.052

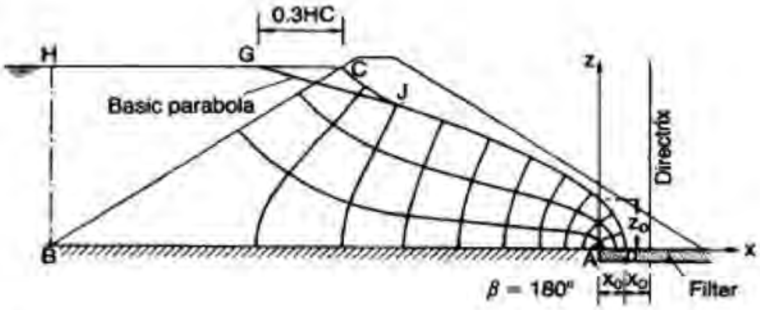
The pseudostatic method of analysis requires an equivalent sustained ground motion, and hence, two thirds of the PGA is considered appropriate. A ground acceleration of 0.014*g* was, therefore, applied in the stability analysis.

8.9.6 Load Cases

Load cases considered for the upstream and downstream slopes in the stability assessment are summarized in Table 8.1. The cases considered are normal, extreme (normal water level with earthquake or IDF) and rapid drawdown.

Project No.	P14504.02
Subject:	UTRCA Embro Dam
Date:	16-Feb-04
By:	A. Touhidi

$$x = x_o - \frac{z^2}{4\,x_o}$$



Point G:

x = -18.36

z = 3.12

Solve for x_o:

x_o = 0.1316

Downstream Slope Angle (Degrees) =	33.7
Downstream Correction to Basic Parabola =	0.36

z	*x*
0	0.13
0.5	-0.34
1	-1.77
1.5	-4.14
2	-7.47
2.5	-11.74
3	-16.96
3.12	-18.36

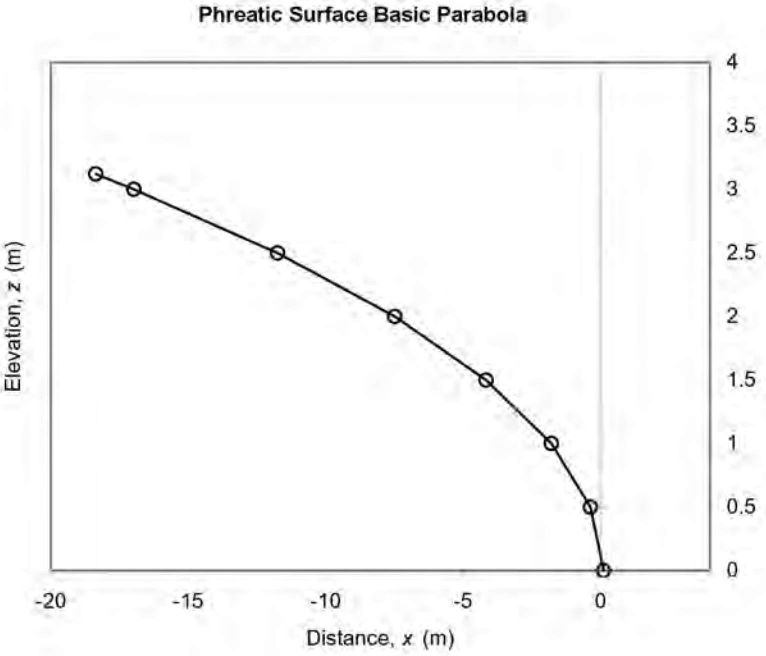


Figure 8.2
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Embro Dam

Embro Dam Basic Parabola

BACK OF FIGURE 8.2

However, the rapid drawdown case was deemed as being not applicable to this site based on the discharge facilities available.

8.9.7 Results of Stability Analyses

The results of the stability analyses are provided in Table 8.1, together with the acceptance criteria and calculated factors of stability. Figures 8.3 to 8.8 graphically depict the cross sections analyzed and the minimum factors of safety established for both the upstream and downstream sections.

The upstream slope fails to meet acceptance criteria for the normal water level condition. The downstream slope fails to meet acceptance criteria for the normal water level condition and the extreme (IDF) condition.

A parametric study indicates that an unrealistically high angle of friction for the embankment fill would be required to bring the dam stability into compliance. It appears that the stability is being adversely influenced by the high water table which is characteristic of homogeneous dams.

It is noted that the stability analysis of the upstream slope was based on an assumed profile, and hence, the analysis should be confirmed using a surveyed profile.

8.10 Assessment

There is no evidence of settlement, cracking, displacement or sinkholes in the dam or in the abutments. Embro Dam is, however, poorly maintained; for example, there is no riprap protection on the upstream slope and wave-induced erosion has occurred (this conflicts with the site inspection report as explained in Section 8.1). A gully/sinkhole has been eroded on the left bank at the downstream toe. This appears to have been caused by emergency spillway overflow redirected towards the downstream toe of the dam instead of taking the planned route which was overgrown with grass. Some minor washing/erosion of the right bank downstream of the pipe outlet has also occurred as a result of this redirected overflow.

The dam does not meet all the required stability criteria. Stability of the upstream slope should be reviewed based on a surveyed profile.

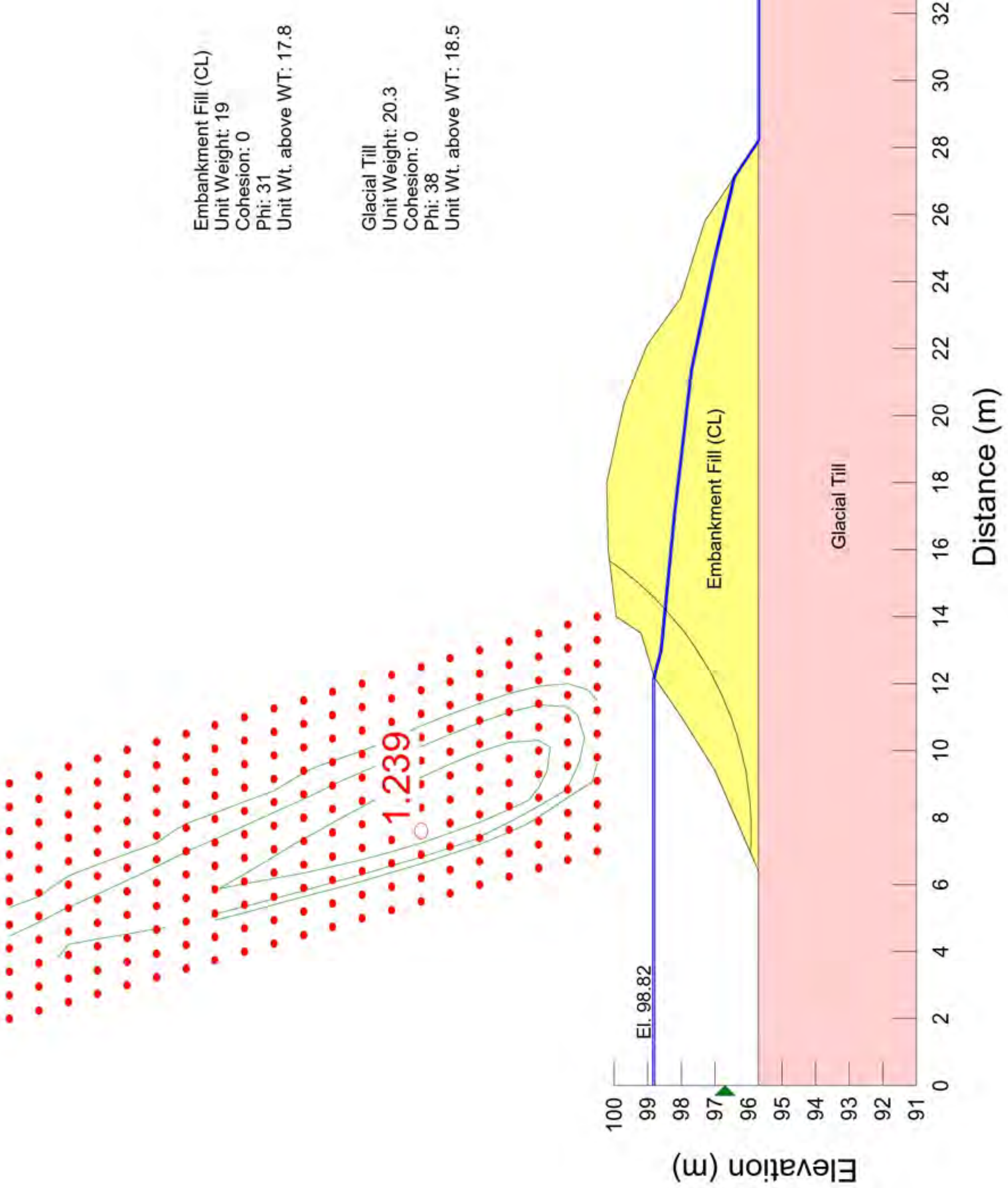


Figure 8.3
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Embro Dam

Upstream Stability, Normal Load Condition

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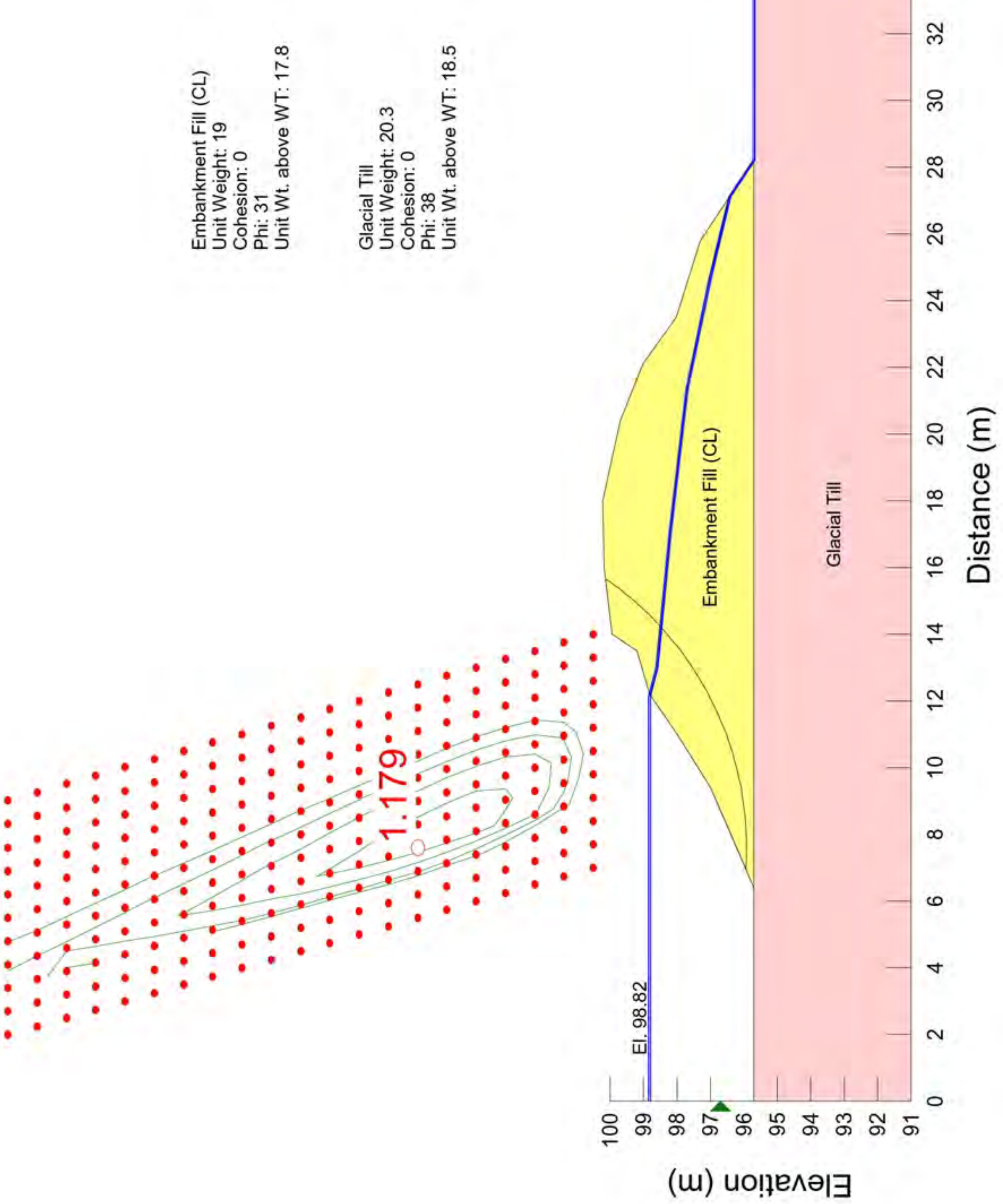

Figure 8.4
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Embro Dam

Upstream Stability, Extreme Load Condition (NWL, S_h)

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International

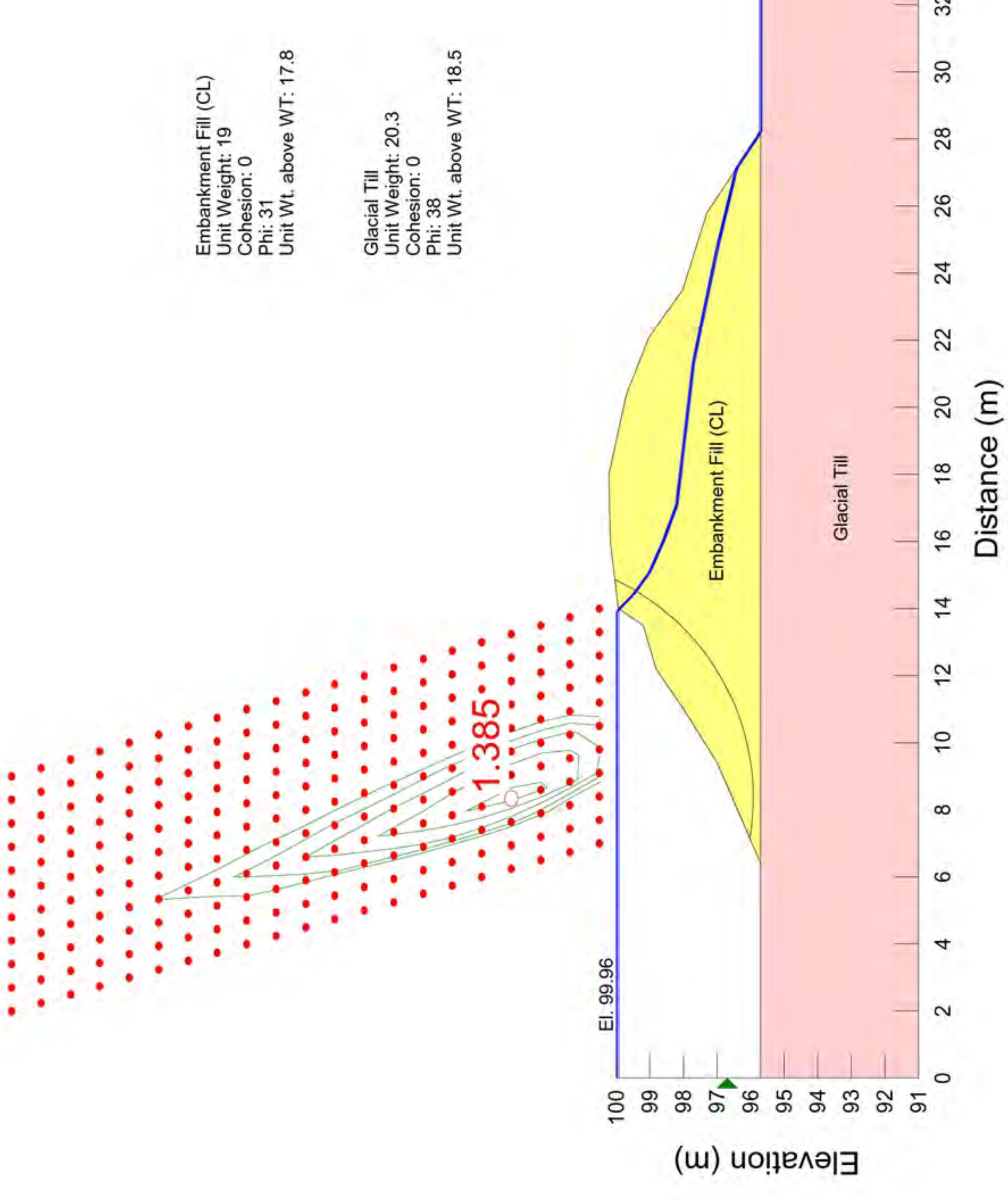

Figure 8.5
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Embro Dam

Upstream Stability, Extreme Load Condition (IDF)

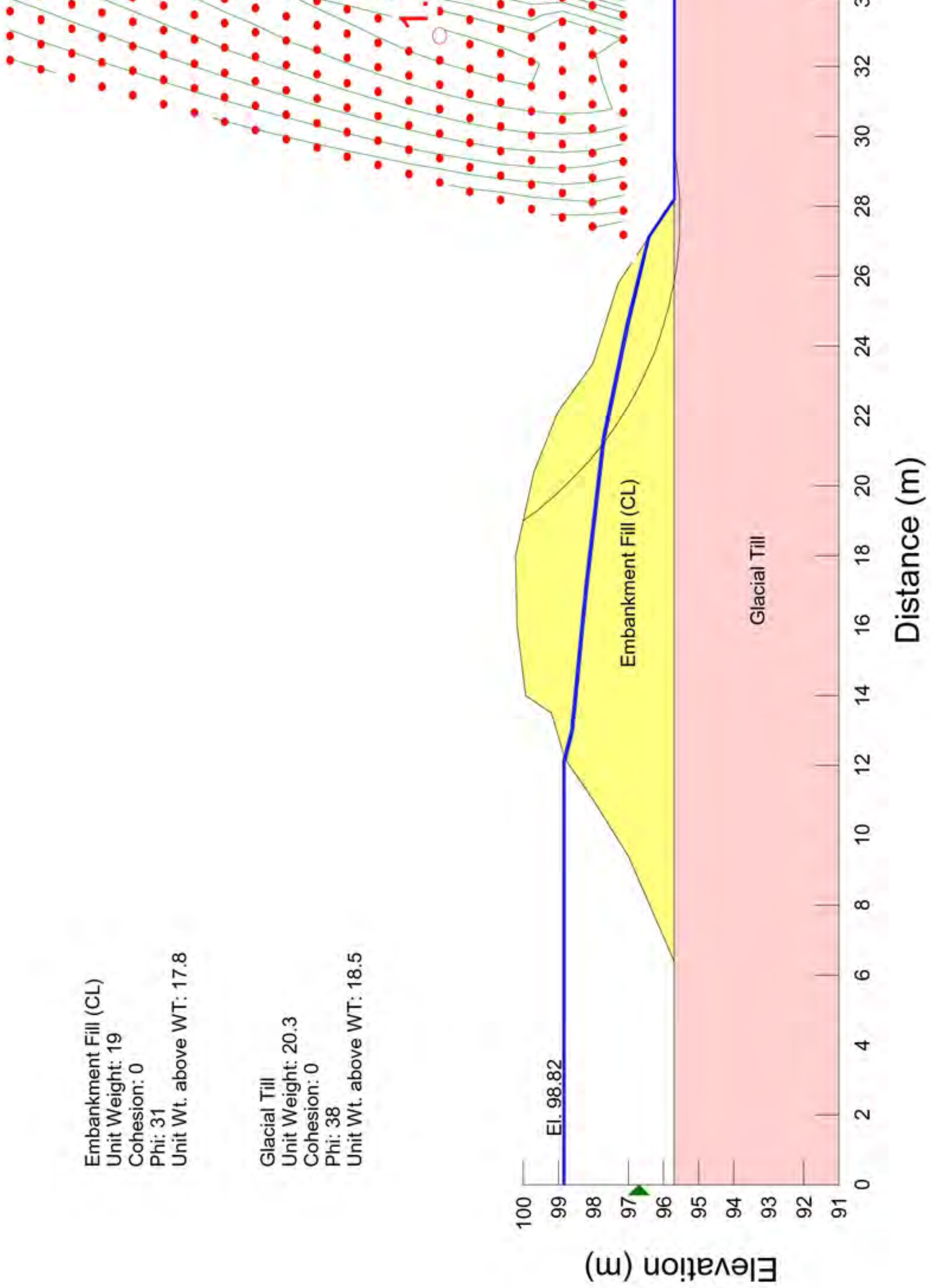

Figure 8.6
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Embro Dam

Downstream Stability, Normal Load Condition

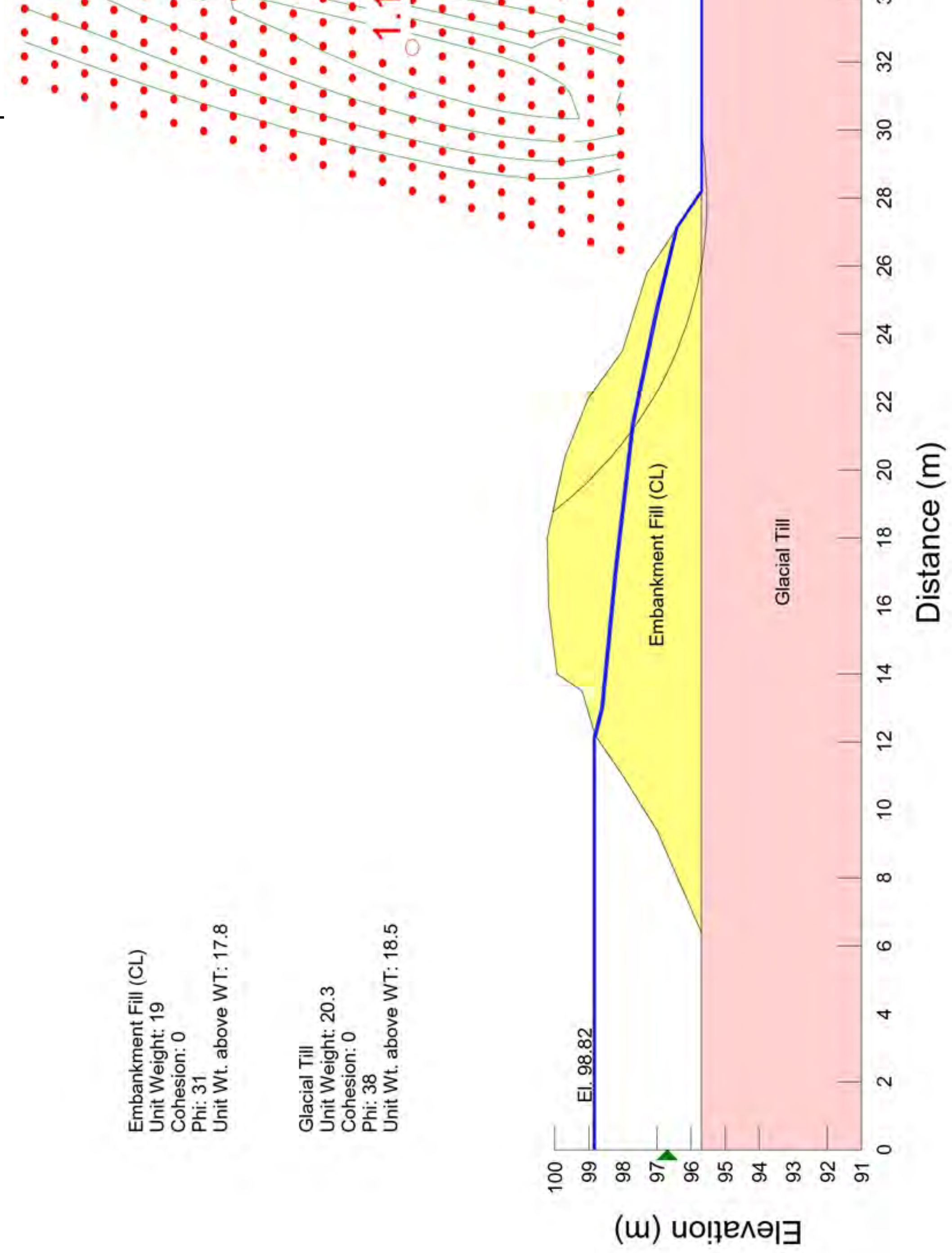

Figure 8.7
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Embro Dam
Downstream Stability, Extreme Load Condition (NWL, S_h)

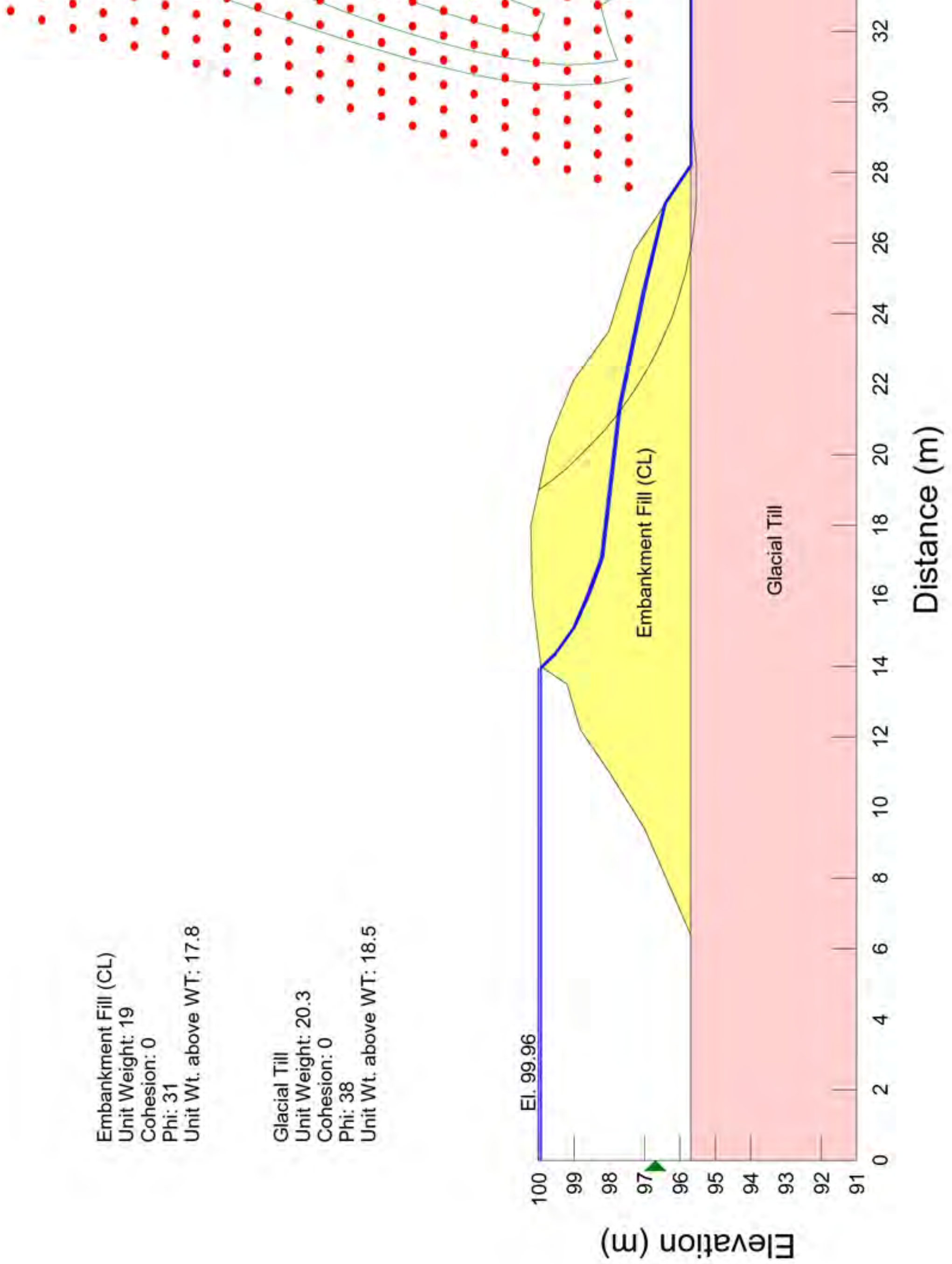

Figure 8.8
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Embro Dam
Downstream Stability, Extreme Load Condition (IDF)

ACRES International

9 Operations, Maintenance and Safety

No OMS manual has been prepared for the Embro Dam under the current dam safety assessment study.

9.1 Operation

The bottom draw inlet structure and concrete outlet pipe combined with the emergency spillway structure at Embro Dam are adequate to ensure the safe passage of the IDF but adequate freeboard is not maintained.

9.2 Access and Signage

The public has free access to the structure. Public access should conform to MNR draft Dam Safety Bulletin #3, ‘Public Access to Dams’, found in Appendix D.

At the Embro Dam, because the public has free access to the dam, there should be “Use At Your Own Risk” signs posted. A sign on the bottom draw inlet structure warning boaters and swimmers to keep away should be posted.

9.3 Fall Protection

Because the Embro Dam is an embankment structure and not operated, fall protection is not required.

9.4 Log Boom

There is no log/debris boom present at this site, and none is required.

Plan

10 Emergency Preparedness Plan

In the event of the failure of the Embro Dam, the UTRCA is responsible to warn residents of a hazardous situation, linking appropriate dam surveillance with emergency response procedures. The procedures that the dam operator is responsible for are defined in an EPP. The EPP is intended to guide the operator with respect to the procedures that are required to be performed in the event of an emergency. These procedures link with UTRCA's overall emergency response plan (ERP) to allow for planning by parties that might be affected in the event of a dam break flood, and the coordination of efforts between federal, provincial and municipal levels of government.

According to the requirements of the draft ODSG

> *"An EPP shall describe the actions to be taken by the dam owner and operator in an emergency. The EPP shall assign responsibility for each action to be taken by an individual (identified by organizational position) and/or a backup."*

For the dam considered under this study, the EPP is required to include the following procedures and information:

- emergency identification and evaluation
- preventative actions (where available)
- notification procedures
- notification flowchart
- communication systems
- access to site
- response during periods of darkness/adverse weather
- sources of equipment
- stockpiling supplies and materials
- inundation maps (where required).

The EPP for this dam was prepared under separate cover by Acres. For details, the reader is referred to this document.

It should be noted that these plans were prepared using the best information that was available at the time of preparation. These plans are, however, dynamic documents that must be reviewed and updated on an annual basis, particularly with respect to contact names, addresses and telephone numbers, in conjunction with UTRCA’s ERP. These notifications were summarized in an Emergency Action Table which is attached at the end of this section.

Testing of the EPP should be performed.

Problem	How to Evaluate	Notification	Data to Record	Action
Flooding	• Water level approaching crest of the dam but no waves overtopping the dam.	District Emergency Response Coordinator UTRCA	• Water flow discharge, headwater, tailwater elevations and rate of change • Weather conditions • Photographs • Dam and flow control equipment condition	• Monitor
	• Waves overtopping crest of Embankment Dam.	District Emergency Response Coordinator UTRCA	• Water discharge, headwater, tailwater elevations and rate of change • Weather conditions • Photographs • Dam and flow control equipment condition	• Monitor • Place sandbags or fill a the crest to increase fre
	• Water level exceeds crest of dam and downstream slopes eroding.	District Emergency Response Coordinator UTRCA Warn anyone in immediate area.	• Water discharge, headwater, tailwater elevations and rate of change • Weather conditions • Photographs • Dam and flow control equipment condition	• Follow procedures for I Dam Failure.
Imminent Dam Failure	• Slopes of Dam severely eroded • Excessive seepage • Whirlpool in headpond • Extensive cracking • Boils or springs downstream • Discharge of fines	District Emergency Response Coordinator UTRCA Warn anyone in immediate area.	• Water discharge, headwater, tailwater elevations and rate of change • Weather conditions • Photographs • Dam and flow control equipment condition	• Restrict boating access
Dam Failure	• Dam breached	District Emergency Response Coordinator UTRCA Warn anyone in immediate area.	• Water discharge, headwater, tailwater elevations and rate of change • Weather conditions • Photographs • Description and location of dam breach	• Restrict boating access
Non-dam Emergency	• Boating accident • Swimming emergency • Personal injury	Emergency Medical Response Team 911 District Emergency Response Coordinator UTRCA	• Nature of problem • Photographs • Names • Cause(s) of accident • Length of time for response	• Follow standard proced for First Aid

Embro Dam
Emergency Action Table

11 Recommendations and Costs

11 Recommendations and Costs

As a result of the 2002/2003 dam safety assessment, a number of recommended actions and maintenance activities were identified that are intended to ensure that the structure will satisfy current dam safety criteria within a 20-yr planning horizon. These ranged from routine monitoring to relatively major concrete rehabilitation works. In each case, an attempt was made to prioritize the remedial work requirements.

For each of the recommended issues, prefeasibility level cost estimates were developed based on an assessment of the general scope of work and typical unit price data from similar projects in Ontario. Note that the cost estimates that were developed were made on the basis of the actual estimated direct construction costs for the individual remedial action identified. As details of the contract packaging for a given dam are not known at this time, other costs (such as mobilization, control of water, increased access costs at remote damsites, contingency and engineering costs) were estimated on the basis of a percentage of the contract price according to the general guidelines summarized in Table 11.1.

Table 11.1

Summary of Additional Costs Associated With a Typical Remedial Repair Project

Item	Cost
Mobilization and demobilization	5% to 7% of capital cost
Control of water during construction	3% to 10% of capital cost (can vary significantly depending on complexity)
Barge access	10% to 15% of capital cost
Contingency	15% to 25% of capital cost
Engineering and supervision	8% to 15% of capital cost

In preparing cost estimates for repairing deteriorating concrete, it was generally anticipated that the scope of the repairs would include all of the deteriorated concrete and at least some of the concrete surrounding the repairs. It was usually assumed that, where necessary, the entire pier, upstream and downstream of the gains, would be repaired at one time. The actual timing of the repairs may, of course, vary. For example, it may be cost-effective where the extent of upstream deterioration is relatively minor to undertake these repairs under a separate, smaller contract, at a later date. There was no attempt made to address the timing of repair issues in this report. It is also noted that costs for repairing areas of relatively minor deterioration, that are not considered to require attention at this time, were not developed.

An explanation of the priority numbers and concrete repair classifications are shown in Tables 11.2 and 11.3. Details of the recommended action and associated costs for the Embro Dam are summarized in Table 11.4. An overall cost summary of the remedial repairs, including allowances for engineering, permitting and environmental costs, is provided in Table 11.5.

Table 11.2

Explanation of Priority Numbers

Priority	Description
1	Immediate - Corrective action required immediately due to safety concerns.
2	High - Corrective action required within 2 years.
3	Medium - Corrective action required within 5 years.
4	Low - Corrective action required within 10 years.
5	Monitoring - Defect should be monitored with corrective action to be taken only when required.

Note: Each level reflects the relative importance or urgency associated with taking some form of action. In cases in which the defects were observed to be safety related (mostly Priority 1 items), action means actual construction. It is noted that some of the Priority 5 items may need to be reassigned a higher priority once the areas have been monitored and investigated and any defects have been identified.

Table 11.3

Concrete Repair Classification

Description		Area (m^2)	Depth of Repair (mm)	Method
1	Sealing contraction joints (above water)	N/A	N/A	Remove existing cracked caulking by mechanical or other means. Clean joint of dirt and other residue. Apply backer rod if joint is deep. Apply primer. Apply polyurethane elastomeric sealant. Applicable to horizontal and vertical surfaces above waterline.
2	Sealing cracks and contraction joints below waterline	N/A	N/A	Requires diver. Remove existing sealant, if present. Clean joint of algae, etc, by wire brushing. Apply sealant such as Devclad 182 with ethafoam backing rod as required.
3	Bonding cracks (above waterline)	N/A	N/A	Required for structural bonding or to stop water leakage. Use epoxy injection for cracks less than 12 mm, cementitious injection for larger cracks. Where a crack is known to be damp or leaking water, use a water-reactive polyurethane resin.
4.1	Small vertical areas	0 – 2	1 – 50	Remove deteriorated concrete, saw cut, clean, trowel repair mortar
4.2	Horizontal areas	1 – 5	12 – 50	Remove deteriorated concrete, saw cut, pour free-flowing repair mortar
4.3	Large vertical areas	-	12 – 50	Remove deteriorated concrete, saw cut, shotcrete
4.4	Unlimited size vertical surfaces with deep deterioration	-	>75	Chip, saw cut, form and pour concrete. Dowels and rebar may be necessary.
4.5	Vertical areas with exposed rebar	-	12 – 50	Remove deteriorated concrete to 50 mm. Behind rebar, clean rebar of all rust, clean concrete and apply repair material.
4.6	Horizontal overlay with rebar	-	12 – 50	Remove deteriorated concrete to 50 mm. Behind rebar, clean rebar of rust, clean, apply overlay in accordance with manufacturer's directions.
4.7	Large areas of new reinforced facing concrete	-	>150	Roughen old concrete, dowel as required, place new rebar, form and pour concrete
5	Vertical grouting of masonry piers	-	-	Repoint masonry along wall faces. Drill vertically through pier from deck level. Grout using balanced, stable, cement-based suspension grouts to fill all voids and cracks in masonry.

Table 11.4

Estimated Remedial Repair Costs – Embro Dam

Item No.	Structure	Component	Defect Description	Repair Description	Repair Type	Estimated Quantity	Estimated Construction Cost (2004 $)	Priority	Remarks
1	Embankment	Upstream and downstream slopes	Stability of slopes do not meet criteria	Flatten slopes or add berms	-	1500 m^3	45,000	1	Survey first to verify inclination of slopes.
2	Emergency spillway	-	Flow does not stay within channel	Excavate clear path away from dam toe	-	420 m^3	8,000	2	During past floods, water has exited the channel and eroded the toe of the left bank.
3	Embankments	Upstream slope	Vegetation growing and some erosion occurring	Remove large vegetation and install riprap	-	22.5 m^3	2,250	2	Vegetation with a short root system such as grasses are beneficial in preventing erosion.
4	Embankments	Downstream slope	Vegetation growing	Remove large vegetation with deep roots	-	-	-	2	Vegetation with a short root system such as grasses are beneficial in preventing erosion. By UTRCA.
5	Downstream channel	Right bank	Erosion along bank	Repair erosion	-	-	Included in Item 2 above	2	From flow out of emergency spillway.
6	Drop inlet structure	Trashracks	Partially clogged with leaves, reeds and branches	Remove debris on a regular basis	-	-	-	5	By UTRCA staff.
7	Entire dam	-	Lack of signage	Install signs	-	4	1,500	2	Install “Use at Own Risk” signs at each end of dam and “Danger – Keep Away” signs on trashracks.

Table 11.4
Estimated Remedial Repair Costs – Embro Dam – 2

Item No.	Structure	Component	Defect Description	Repair Description	Repair Type	Estimated Quantity	Estimated Construction Cost (2004 $)	Priority	Remarks
8	Conduit outlet pipe	Pipe sections	Check alignment of pipe sections	First check visually	-	-	**-**	2	Perform during time of low flow. By UTRCA.
9	Entire dam	Crest	Overtopped during past floods	Redesign emergency spillway	-	-	Included in Item 2 above	2	Design already given to UTRCA.
10	Downstream channel	Banks and invert	Lack of erosion protection	Regrade and add riprap	-	160 m^3	5,600	2	As outlined in sketch by Mike Ragwen.
							62,350		

Table 11.5

Budget Estimate Summary of Construction Costs for Maintenance Repairs for the Embro Dam

Item No.	Description	Unit	Quantity	Unit Price ($)	Amount ($)
1	Mobilization and demobilization	LS	1	5,000	5,000
2	Repairs to dam and structures	LS	1	62,350	62,350
3	**Subtotal (Construction Costs)**				**67,350**
4	Contingency on Construction Costs (20%)				13,470
5	**TOTAL ESTIMATED CONSTRUCTION COSTS**				**80,820**

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Appendix A

Appendix A

Photographs

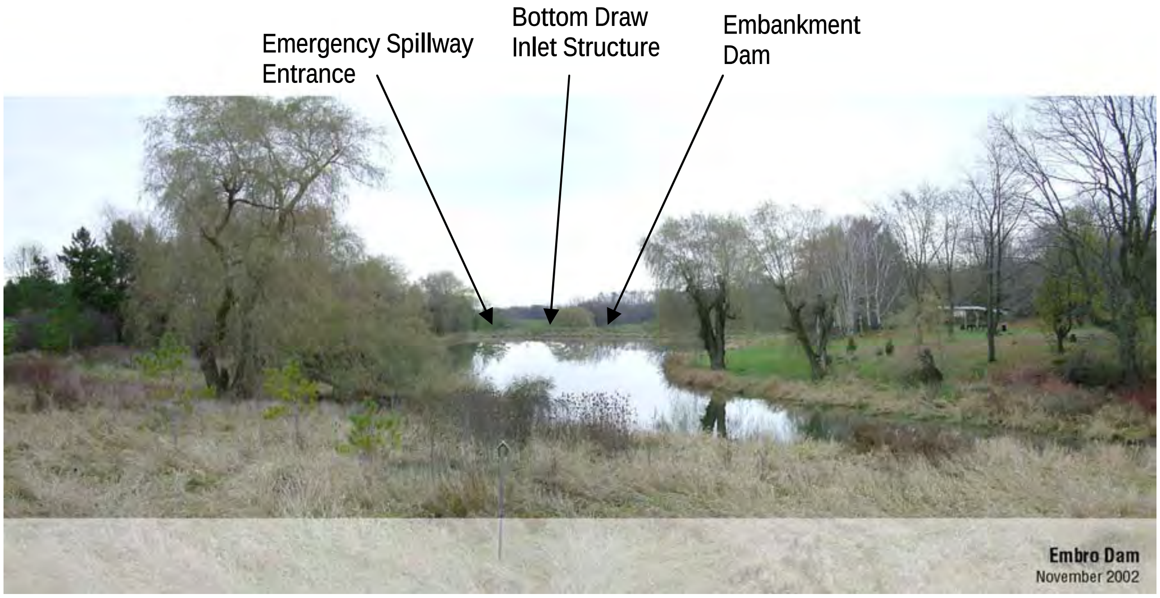
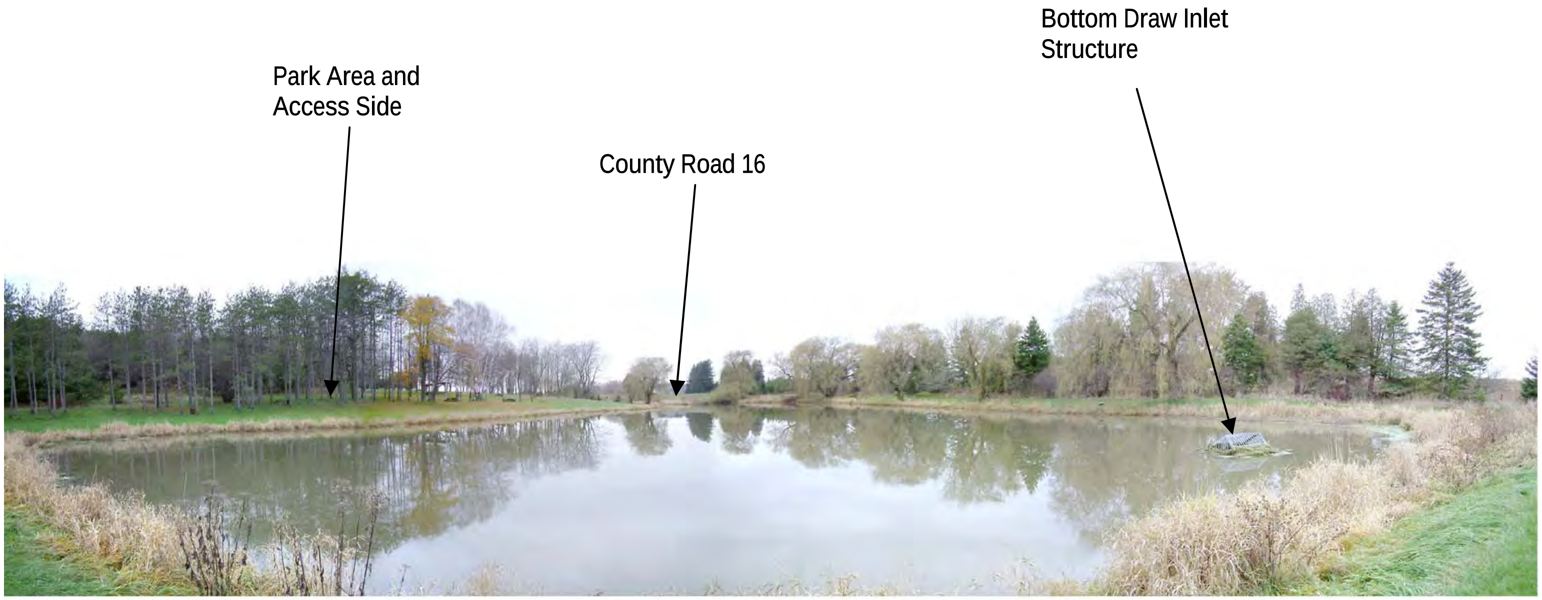


Photo 1 – View of Dam from Far End of Pond

Photo 2 – Downstream View of Embankment at Pipe Outlet

Photo 3 – Emergency Spillway Entrance

Photo 4 – Downstream Channel



Embro Dam
November 2002

Photo 5 – Upstream View of Pond

Photo 6 – Bottom Draw Inlet Structure

Photo 7 – Pool Area at Pipe Outlet

Photo 8 – Upstream Face of Dam

Photo 9 – View along Dam Crest

Large Hole

Photo 10 – Eroded Area at Downstream Toe, Left Bank

Eroded Areas

Photo 11 – Erosion along Right Downstream Channel

Appendix B

Appendix B

Forms B1 and B2

Form B1

Pre-Inspection Background Information

Prepared By: Acres International Limited

Name of Dam: Embro

Latest Construction:
? Stop logs replaced
? New trashracks
1985 Removal of excess weed growth on emergency spillway and embankment

Inspection Dates:
July 2001 UTRCA
July 2000 UTRCA
August 1985 UTRCA
May 1982 UTRCA

Access: 2.0 km south of the town of Embro on County Road 16, turn right into Embro Conservation Area

Lake Controlled: Embro Pond

Lake Area: 0.005 km^2

Watershed: Spring Creek, a tributary of the North Branch Creek, Middle Branch of the Thames Watershed

Drainage Area: 7 km^2

Gauge Info: None at the dam

Rule Curves: Not available

List of Drawings:
UTRCA:
\# ? Dam Hazard Identification, Embro Dam, July 2001

Meteorological and Hydrological Data: The following meteorological data are available from Stratford, Woodstock and London airport:

- daily precipitation amounts
- mean, maximum and minimum daily temperatures.

The nearest regional streamflow gauging stations are

- Middle Thames River at Thamesford (Station No. 02GD004; drainage area = 306 km^2)
- Waubuno Creek near Dorchester (Station No. 02GD020; drainage area = 108 km^2)
- Nissouri Creek near Embro (Station No. 02GD022; drainage area = 29 km^2).

These gauging stations are in the same watershed area but their drainage areas are much larger and probably of limited value, except for gauge at Nissouri Creek but it only has 7 years of flow records.

Topographic Maps:	40 P/2 Woodstock (1:50 000-scale)
Soil and Land-Use Maps:	Soil Map of Oxford, Ontario (digitized, UTRCA) and The Upper Thames River Watershed Report Cards 2001
Dam Height:	Approximately 4.5 m
Dam Length:	Approximately 100 m
No. of Sluiceways:	None. One bottom draw inlet and one emergency spillway channel.
No. of Stop Logs per Bay:	Not applicable. Number of logs on upstream side of bottom draw inlet unknown.
Hydrologic Flows:	Nothing available in files
Hydraulic Analysis:	Nothing available in files
Dam Operation:	Three logs removed in fall and replaced in spring
Soils Reports:	None available
Underwater Inspections:	None available
Divestment Opportunities:	Annual agreement for area management with the Embro Pond Association
Property Ownership:	UTRCA
CA Maintenance:	Embro Pond Association

Dam Maintenance: UTRCA

Known Problems: Previous inspection reports indicated trees and other vegetation should be removed from the embankment slopes. Erosion and formation of a gully along the toe of the left downstream embankment was also noted.

Summary of File: See Table 3.1 documenting all dam safety reference information found in UTRCA files

Form B2

Dam Inspection Report

Date:	November 12 and 20, 2002
Structure:	Embro Dam
Municipality:	Zorra
Location:	County Road 16
GPS Coordinates:	UTM, NAD83: 17 506 888 E, 4 779 900 N Lat/Long: 43° 10’ 19” N, 80° 54’ 55” W
Inspected By:	B. Craig, T. Hartung, P. Last, M. Ragwen and B. Sinclair of Acres International Limited
Weather:	Overcast with occasional showers, air temperature approximately 6°C

---

1. Earth Embankment

For details, see the photographs in Appendix A.

- Upstream bank overgrown with reeds and grass. No riprap slope protection visible. No erosion or benching apparent.
- Emergency spillway channel on right bank. Upstream end grassed and open but lower section overgrown with trees with piles of debris. Drilling mud visible in lower section.
- Downstream slope heavily overgrown with grass and shrubs. Erosion is occurring along the gully which follows the toe of the slope on the left side. Erosion occurring along the slopes of the downstream channel, possibly due to discharges from the pipe outlet.

---

2. **Concrete Structures** (wingwalls, piers, deck, spillways, apron, etc)

For details, see the photographs in Appendix A.

- Concrete of the bottom draw inlet structure appeared in good condition.

- Concrete of the outlet conduit also appeared in good condition. Because of the flow in the pipe and the depth of the water at the outlet, a close examination along the length of the pipe was not performed to see if there was any misalignment along the length of the conduit.

---

3. Wooden and Metal Structures (decks, gains, railings, conduits, etc)

For details, see the photographs in Appendix A.

- Bottom draw inlet structure trashrack is fabricated of galvanized steel, is in good condition, and is well-fastened to the concrete base.

---

4. Gates and/or Stop Logs (identified looking downstream left to right)

For details, see the photographs in Appendix A.

Three stop logs are removed in the fall and replaced in the spring.

---

5. Water Level Gauge (reading and condition)

For details, see the photographs in Appendix A.

No water level gauge was seen at this dam.

---

6. Winches (type and number)

For details, see the photographs in Appendix A.

Stop logs are small enough that the top ones can be manipulated by hand. This occurs each fall and spring.

---

7. Valves (type and number)

None at this site.

---

8. Boom (driftwood, chains, anchors)

For details, see the photographs in Appendix A.

No boom present at this site.

---

9. **Erosion** (upstream and downstream)

Erosion damage was seen on the downstream slope as indicated in Item 1 above.

---

10. **Seepage or Leaks**

For details, see the photographs in Appendix A.

Close access to the pipe outlet was not possible for signs of leakage around the pipe. No other signs of leakage were visible.

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11. **Access Route** (location of gate keys, winch handles and keys)

Vehicular access is from the right, which is blocked by a locked gate. UTRCA and the Conservation Area staff have keys to unlock the gate. No keys or equipment are required to manipulate the stop logs. The parking lot and the path to the dam are not plowed in the winter season.

---

12. **Safety Issues** (public and operator)

- No guard screen on the conduit outlet to prevent entry.
- Logs cannot be removed under overflowing conditions.

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13. **Signage**

For details, see the photographs in Appendix A.

- The only sign is one warning of unstable ice in winter.

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14. **Divestment and/or Decommissioning Opportunities**

Annual agreement with the Embro Pond Association for area management.

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15. **General Remarks**

There was a considerable buildup of weeds in front of the trashracks of the bottom draw inlet structure. Concrete is in good condition. Emergency spillway needs to be excavated so that water flows down the alignment rather than down the toe of the left embankment. Slopes need to be cleared of vegetation.

---

16. Recommendations

- Install riprap on slopes of outlet channel to prevent erosion.
- Excavate emergency spillway so water can flow along the alignment of this channel.
- Fill sinkhole and reshape downstream channel banks.
- Remove large vegetation with deep roots from slopes and from the emergency spillway alignment.
- Perform a topographic/bathymetric survey to define elevation of dam and emergency spillway and volume of the pond.
- Drill one hole in the dam to obtain information about the embankment and the foundation.

Appendix C

Appendix C

VO2-UTRCA Storm Distributions

Table C1

VO2-UTRCA Storm Distributions

6 Hours		12 Hours		24 Hours		48 Hours	
Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.25	0.50	0.25	0.25	0.25	0.13	0.25	0.06
0.50	1.00	0.50	0.50	0.50	0.25	0.50	0.13
0.75	3.00	0.75	0.75	0.75	0.38	0.75	0.19
1.00	5.00	1.00	1.00	1.00	0.50	1.00	0.25
1.25	14.01	1.25	2.00	1.25	0.63	1.25	0.31
1.50	23.02	1.50	3.00	1.50	0.75	1.50	0.38
1.75	46.04	1.75	4.00	1.75	0.88	1.75	0.44
2.00	69.06	2.00	5.00	2.00	1.00	2.00	0.50
2.25	76.03	2.25	9.50	2.25	1.50	2.25	0.56
2.50	83.00	2.50	14.00	2.50	2.00	2.50	0.63
2.75	86.50	2.75	18.50	2.75	2.50	2.75	0.69
3.00	90.00	3.00	23.00	3.00	3.00	3.00	0.75
3.25	92.00	3.25	34.50	3.25	3.50	3.25	0.81
3.50	94.00	3.50	46.00	3.50	4.00	3.50	0.88
3.75	95.00	3.75	57.50	3.75	4.50	3.75	0.94
4.00	96.00	4.00	69.00	4.00	5.00	4.00	1.00
4.25	96.50	4.25	72.50	4.25	7.25	4.25	1.25
4.50	97.00	4.50	76.00	4.50	9.50	4.50	1.50
4.75	97.50	4.75	79.50	4.75	11.75	4.75	1.75
5.00	98.00	5.00	83.00	5.00	14.00	5.00	2.00
5.25	98.50	5.25	84.75	5.25	16.25	5.25	2.25
5.50	99.00	5.50	86.50	5.50	18.50	5.50	2.50
5.75	99.50	5.75	88.25	5.75	20.75	5.75	2.75
6.00	100.00	6.00	90.00	6.00	23.00	6.00	3.00
		6.25	91.00	6.25	28.75	6.25	3.25
		6.50	92.00	6.50	34.50	6.50	3.50
		6.75	93.00	6.75	40.25	6.75	3.75
		7.00	94.00	7.00	46.00	7.00	4.00
		7.25	94.50	7.25	51.75	7.25	4.25
		7.50	95.00	7.50	57.50	7.50	4.50
		7.75	95.50	7.75	63.25	7.75	4.75
		8.00	96.00	8.00	69.00	8.00	5.00
		8.25	96.25	8.25	70.75	8.25	6.13
		8.50	96.50	8.50	72.50	8.50	7.25
		8.75	96.75	8.75	74.25	8.75	8.38
		9.00	97.00	9.00	76.00	9.00	9.50
		9.25	97.25	9.25	77.75	9.25	10.63
		9.50	97.50	9.50	79.50	9.50	11.75
		9.75	97.75	9.75	81.25	9.75	12.88
		10.00	98.00	10.00	83.00	10.00	14.00
		10.25	98.25	10.25	83.88	10.25	15.13
		10.50	98.50	10.50	84.75	10.50	16.25
		10.75	98.75	10.75	85.63	10.75	17.38
		11.00	99.00	11.00	86.50	11.00	18.50
		11.25	99.25	11.25	87.38	11.25	19.63
		11.50	99.50	11.50	88.25	11.50	20.75
		11.75	99.75	11.75	89.13	11.75	21.88
		12.00	100.00	12.00	90.00	12.00	23.00
				12.25	90.50	12.25	25.88

Table C1

VO2-UTRCA Storm Distributions

6 Hours		12 Hours		24 Hours		48 Hours	
Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth
				12.50	91.00	12.50	28.75
				12.75	91.50	12.75	31.63
				13.00	92.00	13.00	34.50
				13.25	92.50	13.25	37.38
				13.50	93.00	13.50	40.25
				13.75	93.50	13.75	43.13
				14.00	94.00	14.00	46.00
				14.25	94.25	14.25	48.88
				14.50	94.50	14.50	51.75
				14.75	94.75	14.75	54.63
				15.00	95.00	15.00	57.50
				15.25	95.25	15.25	60.38
				15.50	95.50	15.50	63.25
				15.75	95.75	15.75	66.13
				16.00	96.00	16.00	69.00
				16.25	96.13	16.25	69.88
				16.50	96.25	16.50	70.75
				16.75	96.38	16.75	71.63
				17.00	96.50	17.00	72.50
				17.25	96.63	17.25	73.38
				17.50	96.75	17.50	74.25
				17.75	96.88	17.75	75.13
				18.00	97.00	18.00	76.00
				18.25	97.13	18.25	76.88
				18.50	97.25	18.50	77.75
				18.75	97.38	18.75	78.63
				19.00	97.50	19.00	79.50
				19.25	97.63	19.25	80.38
				19.50	97.75	19.50	81.25
				19.75	97.88	19.75	82.13
				20.00	98.00	20.00	83.00
				20.25	98.13	20.25	83.44
				20.50	98.25	20.50	83.88
				20.75	98.38	20.75	84.31
				21.00	98.50	21.00	84.75
				21.25	98.63	21.25	85.19
				21.50	98.75	21.50	85.63
				21.75	98.88	21.75	86.06
				22.00	99.00	22.00	86.50
				22.25	99.13	22.25	86.94
				22.50	99.25	22.50	87.38
				22.75	99.38	22.75	87.81
				23.00	99.50	23.00	88.25
				23.25	99.63	23.25	88.69
				23.50	99.75	23.50	89.13
				23.75	99.88	23.75	89.56
				24.00	100.00	24.00	90.00
						24.25	90.25
						24.50	90.50
						24.75	90.75

Table C1

VO2-UTRCA Storm Distributions

6 Hours		12 Hours		24 Hours		48 Hours	
Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth
						25.00	91.00
						25.25	91.25
						25.50	91.50
						25.75	91.75
						26.00	92.00
						26.25	92.25
						26.50	92.50
						26.75	92.75
						27.00	93.00
						27.25	93.25
						27.50	93.50
						27.75	93.75
						28.00	94.00
						28.25	94.13
						28.50	94.25
						28.75	94.38
						29.00	94.50
						29.25	94.63
						29.50	94.75
						29.75	94.88
						30.00	95.00
						30.25	95.13
						30.50	95.25
						30.75	95.38
						31.00	95.50
						31.25	95.63
						31.50	95.75
						31.75	95.88
						32.00	96.00
						32.25	96.06
						32.50	96.13
						32.75	96.19
						33.00	96.25
						33.25	96.31
						33.50	96.38
						33.75	96.44
						34.00	96.50
						34.25	96.56
						34.50	96.63
						34.75	96.69
						35.00	96.75
						35.25	96.81
						35.50	96.88
						35.75	96.94
						36.00	97.00
						36.25	97.06
						36.50	97.13
						36.75	97.19
						37.00	97.25
						37.25	97.31

Table C1

VO2-UTRCA Storm Distributions

6 Hours		12 Hours		24 Hours		48 Hours	
Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth
						37.50	97.38
						37.75	97.44
						38.00	97.50
						38.25	97.56
						38.50	97.63
						38.75	97.69
						39.00	97.75
						39.25	97.81
						39.50	97.88
						39.75	97.94
						40.00	98.00
						40.25	98.06
						40.50	98.13
						40.75	98.19
						41.00	98.25
						41.25	98.31
						41.50	98.38
						41.75	98.44
						42.00	98.50
						42.25	98.56
						42.50	98.63
						42.75	98.69
						43.00	98.75
						43.25	98.81
						43.50	98.88
						43.75	98.94
						44.00	99.00
						44.25	99.06
						44.50	99.13
						44.75	99.19
						45.00	99.25
						45.25	99.31
						45.50	99.38
						45.75	99.44
						46.00	99.50
						46.25	99.56
						46.50	99.63
						46.75	99.69
						47.00	99.75
						47.25	99.81
						47.50	99.88
						47.75	99.94
						48.00	100.00

Table C2

Rain-on-Snowmelt Distribution Pattern for Gauge D for 1 Day, 3 Days and 8 Days

1 Day		3 Days		8 Days	
Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth
0.0	0.00	0.0	0.00	0.0	0.00
1.0	1.00	2.0	0.43	2.0	0.27
2.0	2.00	4.0	0.99	4.0	0.54
3.0	3.00	6.0	1.98	6.0	1.08
4.0	4.00	8.0	2.98	8.0	1.62
5.0	5.50	10.0	4.96	10.0	2.69
6.0	7.00	12.0	6.95	12.0	3.77
7.0	9.00	14.0	7.94	14.0	4.31
8.0	11.00	16.0	8.93	16.0	4.85
9.0	14.50	18.0	9.50	18.0	5.12
10.0	18.00	20.0	9.92	20.0	5.39
11.0	26.00	22.0	9.92	22.0	5.39
12.0	34.00	24.0	9.92	24.0	5.39
13.0	53.50	26.0	10.72	26.0	5.68
14.0	73.00	28.0	11.69	28.0	5.97
15.0	79.50	30.0	13.45	30.0	6.55
16.0	86.00	32.0	15.22	32.0	7.12
17.0	89.00	34.0	18.75	34.0	8.28
18.0	92.00	36.0	22.28	36.0	9.44
19.0	94.00	38.0	24.05	38.0	10.02
20.0	96.00	40.0	25.82	40.0	10.59
21.0	97.00	42.0	26.78	42.0	10.88
22.0	98.00	44.0	27.58	44.0	11.17
23.0	99.00	46.0	27.58	46.0	11.17
24.0	100.00	48.0	29.03	48.0	11.17
		50.0	30.48	50.0	11.44
		52.0	32.57	52.0	11.80
		54.0	35.47	54.0	12.42
		56.0	40.45	56.0	13.04
		58.0	52.04	58.0	14.29
		60.0	80.37	60.0	15.53
		62.0	89.86	62.0	16.15
		64.0	94.21	64.0	16.77
		66.0	97.10	66.0	17.13
		68.0	98.55	68.0	17.40
		70.0	100.00	70.0	17.40
		72.0	100.00	72.0	17.40
				74.0	17.70
				76.0	18.11

Table C2

Rain-on-Snowmelt Distribution Pattern for Gauge D for 1 Day, 3 Days and 8 Days

1 Day		3 Days		8 Days	
Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth
				78.0	18.83
				80.0	19.55
				82.0	20.98
				84.0	22.42
				86.0	23.14
				88.0	23.85
				90.0	24.27
				92.0	24.57
				94.0	24.57
				96.0	24.57
				98.0	25.15
				100.0	25.85
				102.0	27.13
				104.0	28.41
				106.0	30.96
				108.0	33.52
				110.0	34.79
				112.0	36.07
				114.0	36.77
				116.0	37.35
				118.0	37.35
				120.0	38.39
				122.0	39.44
				124.0	40.95
				126.0	43.05
				128.0	46.66
				130.0	55.04
				132.0	75.53
				134.0	82.40
				136.0	85.54
				138.0	87.63
				140.0	88.68
				142.0	89.73
				144.0	89.73
				146.0	89.94
				148.0	90.26
				150.0	90.79
				152.0	91.31
				154.0	92.37
				156.0	93.42

Table C2

Rain-on-Snowmelt Distribution Pattern for Gauge D for 1 Day, 3 Days and 8 Days

1 Day		3 Days		8 Days	
Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth	Duration (hrs)	Cumulative Percentage of Storm Depth
				158.0	93.95
				160.0	94.48
				162.0	94.80
				164.0	95.01
				166.0	95.01
				168.0	95.01
				170.0	95.26
				172.0	95.51
				174.0	96.01
				176.0	96.51
				178.0	97.50
				180.0	98.50
				182.0	99.00
				184.0	99.50
				186.0	99.75
				188.0	100.00
				190.0	100.00
				192.0	100.00

Reference:
UTRCA's Visual Otthymo, Version 2 (VO2) modeling for the Upper Thames River basin (MMM, 1983; UTRCA, 1995; M. Wood personal communication, 2003).

Appendix D

Appendix D

MNR Dam Safety Bulletins

Dam Safety Bulletin #1

Boom Logs

MNR Policy and Procedure Directive (1970)

Where there is a large collection of debris and floodwood, MNR installs suitable booms upstream to protect the dam.

MNR Legal Opinion (1999)

MNR installs boom logs upstream of a dam to catch debris to protect the dam. From a legal perspective, MNR's corporate position is that the boom logs must be capable of collecting debris.

The color, of the boom logs, does not change MNR's corporate position. The fact that the colored logs are also used as navigational aids and serve as warning devices is irrelevant since MNR does not use them for this purpose.

Design Considerations

A boom type that performs well in one location may not perform well in another location that may have entirely different conditions.

Debris load design calculations must take into consideration the debris that might be expected during a flood event. The worst possible time for a boom log to fail would be during a flood.

Boom log type, size and cost can vary significantly. Capital costs should take in to consideration the reduced maintenance or extended life of the boom.

DRAFT

Dam Safety Bulletin #2

Signage

MNR Policy and Procedure Directive (1970)

Where hazardous boating conditions exist near dams, the Ministry installs warning signs.

MNR Legal Opinion (1994 Inquest)

In its management of the Crown lands and waters of Ontario, MNR has a duty to be aware of public hazards and to both minimize those hazards wherever possible and warn those that may be affected by them.

The Ministry frequently uses warning signs on dams to alert boaters of danger ahead. Where there is no hazard to the boating public at a dam, a sign may not be placed.

Considerations for placing signs at dams

Signs should be used to warn the public of hazardous conditions that exist and to discourage the public from continuing unsafe activities that have taken place.

Signs are to be placed where they are highly visible for the purpose in which they are intended.

Signs are often subject to vandalism. They could disappear without your knowledge. Take pictures of signs when newly installed and make note of its condition each time the dam is visited.

Signs must be maintained.

Sample sign wording

DANGER   Fast Water Keep Clear

No Trespassing
No Camping
No swimming
(if these have been known to take place)

Note: some dam decks have been designed to also serve as a bridge so "No Trespassing " would not be applicable in these cases.

Portage (if on an identified canoe route)

Dam Ahead (where dam is around a corner or where only a weir that is not very noticeable)

Sign Design

Danger signs are to have 8" high red letters on a white background
Danger signs are to be 4' x 8' in size

All signs are to be bilingual

Restricting Access

In addition to the "No Trespassing" sign a locked chain should be placed across the access to the dam deck so that a conscious effort would have to be made to trespass.

Dam Safety Bulletin #3

Public Access to Dams

General

Public access to MNR dams poses a significant challenge with respect to public safety because of the potential for falling either into the water on the upstream side, or to the ground surface or water below the dam. There may also be safety issues with public access to hoisting equipment (overhead gantry, pedestal or rail hoists) and gains openings.

While handrails conforming to the requirements of the OHSA for industrial work environments should already be installed at all dams where there is a potential to fall into the water, or where there is a potential to fall 1.2m or more, these do not necessarily protect against fall hazards in all cases. For example, kick plates along the bottom of handrails are not usually installed at dams because of the problem that they create for snow removal and water flow impedance during dam overtopping. A member of the general public unaware of the hazard could still fall through the railing if attempting to cross the dam while there is ice, snow or other slippery condition on the deck.

All dams should be equipped with gains covers that cover the entire gains opening, and are equipped with locks so that the public does not have access to the gains opening.

Dams where the deck doubles as a vehicle bridge:

It is not feasible to block access to the dam deck where the deck also serves as a bridge. However, a combination of gates, chains, guiderails or handrails can be used to block access to the portion of the dam deck incorporating the gains opening and hoist mechanisms. Some type of barrier should always be used to inhibit public access to these areas. The bridge deck and barrier between the bridge and the rest of the dam should conform to the Ontario Highway Bridge Design Code (OHBDC). Section 5 of the OHBDC deals with barriers.

Appropriate signage should also be used advising the public of any hazards (i.e., Danger - No Trespassing or other appropriate signage). Consult Dam Safety Bulletin #2 for details on signage.

The type of barrier used to block access to all or part of the dam should reflect the degree of hazard associated with public access. For example, a locked chain requires little effort to pass, and may be used in cases where the hazard is low, whereas a fence requires some more effort to climb, and could be used where the hazard is high.

Dams where the deck doubles as a pedestrian bridge:

As in the case where the dam serves as a bridge deck for vehicles, pedestrian access to the portion of the dam occupied by the hoist mechanism and gains opening should be restricted by an appropriate barrier and signage. Where it is not possible to restrict access to this portion of the dam while still leaving an area for pedestrian passage, a barrier should inhibit access to the entire dam, and other means of pedestrian passage used.

The portion of the dam accessible for pedestrian passage should conform to the standards stipulated in Section 5-4.5 of the OHBDC. Structures supporting pedestrian traffic should be designed to the loading stipulated in Section 2-4.3.3 of the OHBDC as a minimum.

Alternate means of pedestrian passage should be explored when major dam upgrades or dam replacement are being considered, or when the public safety risks are high. These may consist of pedestrian walkways attached to the dam, or completely separate pedestrian bridges. Walkways or bridges should conform to the OHBDC requirements.

The Regional Engineering Unit can facilitate procurement of any consulting services required for design of facilities appropriate for vehicle or pedestrian passage over dams.

Appendix E

Appendix E

Dam Operator Questionnaire

Embro

P14504.09.04

Dam Safety – General Dam Operator Questionnaire

It is recommended that the dam operator complete this questionnaire for each site at the start of a Dam Safety Review.

This questionnaire will update information on discharge facilities and operating equipment. The information will be used to conduct the Dam Safety Review. The information is broken down into the following categories:

- Part I - Site Description
- Part II - General Operational Information
- Part III - Hydraulic Discharge and Operating Facilities
  - A. Discharge Facilities
  - B. Operating Equipment
  - C. Operating Problems
- Part IV Past Dam Incidents
- Part V Emergency Preparedness Plan (EPP) Information

Throughout the questionnaire, the following definitions of spillway and sluice apply:

Spillway	A structure over which flood flows are discharged. The discharge is uncontrolled, i.e., an overflow structure.
Sluice	A structure through which flood flows are discharged; the flow is controlled by gates, stop logs or valves.
An emergency	Severe flooding, possible dam failure conditions or a person(s) in danger from a boating accident or drowning.

Office: UTRCA Watershed: North Branch / Mud Creek Site: Embro

Prepared by: Chris Tasker Date: Feb 17/2003

Person(s) to contact for additional information:

Name: Chris Tasker Telephone: 519 451-2800 x 238

Questions	Answers/Observations/Comments
Part I –Site Description (To be completed prior to distributing questionnaire. Data to be reviewed and confirmed by Operating Staff)	
1. Facilities Summary **Type** Sluices –gate Sluices –log Sluices –valve (Manufacturer, size, type, etc.) Debris boom Trash rack Non-overflow walls Spillways/overflow walls Upstream retaining walls Downstream retaining walls Other –	**Number** Bottom Draw drop inlet controlled by stop logs
2. Elevation Datum (Canadian Geodetic Datum (CGD) or other - specify)	~~B~~ top of drop inlet

Part II - General Operational Information

3. Please list any major repairs/maintenance since construction that you know of.	-concrete repairs -installation of emergency spillway
4. (a) Who operates this site? NA	☐ Contractor ☐ Other Contact person Legal Agreement in place?
(b) How many staff are normally available to operate the site?	NA
(c) Is this person/team responsible for operating other sites? (d) If yes, where?	☐Yes ☐No NA
(e) If answer to (c) is yes, is there sufficient staff to operate these sites simultaneously?	☐Yes ☐No NA
(f) If answer to (e) is no, is other assistance available? (g) If yes, who and from where?	☐Yes ☐No NA
5. (a) Is an operations log book kept at the dam? (b) Is an operations log book kept elsewhere? (c) If yes to either (a) or (b), where is it located and what information is logged?	☐Yes ☑No ☐Yes ☑No
(d) Do staff stay at this site during an emergency?	☐ Yes ☑No
(e) How are communications maintained with the area office? (i) at site (ii) traveling to/from site	MIKENET / CELL " "
6. Most likely means of access under emergency conditions during: (a) Spring (b) Summer/Fall (c) Winter	☑Road ☐Boat ☐Snowmobile ☐ATV ☐Helicopter ☑Walk ☑Road ☐Boat ☐Snowmobile ☐ATV ☐Helicopter ☑Walk ☑Road ☐Boat ☐Snowmobile ☐ATV ☐Helicopter ☑Walk

7. Are problems or restrictions for accessing the site in an emergency situation foreseen?	
(a) Spring	☐Yes ☑No
(b) Summer/Fall	☐Yes ☑No
(c) Winter	☑Yes ☐No
If yes, please describe (e.g. will the access road or a bridge be accessible if there is a major flood?)	Parking lot & path not plowed
8. Length of time it will take staff to access the site under emergency conditions.	
(a) Spring	☑Less than 1/2 h ☐1/2 to 2 h ☐2 h to 1/2 d ☐1/2 to 1 d ☐More than 1 d
(b) Summer/Fall	☑Less than 1/2 h ☐1/2 to 2 h ☐2 h to 1/2 d ☐1/2 to 1 d ☐More than 1 d
(c) Winter	☑Less than 1/2 h ☐1/2 to 2 h ☐2 h to 1/2 d ☐1/2 d to 1 d ☐More than 1 d
9. Once at the site, how long will it take staff to achieve maximum spill capacity (assuming headwater level is at Maximum Operating Level)?	☐Less than 1/2 h ☐1/2 to 1 h ☐1 h to 2 h ☐2 h to 1/2 d ☐1/2 d to 1 d ☐2 d ☐3 d ☐More than 3 d NA
10. How many staff members are required to achieve maximum spill capacity for the above time estimate?	NA
11. (a) Are there any emergency procedures in place to deal with a dam accident or extreme flood condition? (b) If yes, what is the name of the document?	☐Yes ☑No
12. How often is this dam operated?	0 /month ~~0~~ /year 2
13. (a) Is there a water level gauge at this site?	☐Yes ☑No
(b) If no, is there a gauge at a dock nearby?	☐Yes ☑No
(c) What is the location of the gauge (if applicable)?	top of drop inlet.
(d) To what is this gauge referenced?	☐CGD ☐Local structure datum ☐ Other datum
(e) Is the gauge metric or imperial?	☐ Metric ☐ Imperial

14. (a) Are there any recreational activities (such as boating, fishing, canoe portages, hiking or snowmobiling) in close proximity to the dam in either upstream or downstream areas? (b) If yes, please describe.	☑ Yes ☐ No fishing on pond
15. (a) What other agencies are involved with flow regulation along the river? (b) Who are the contact persons?	None
16. What else may be affected by changes in water levels?	☐ cottagers ☐ recreational boaters ☐ municipal water supply ☐ private water supply ☑ ~~sensitive~~ fisheries/habitat ☐ Float plane landing
17. (a) Are there any known operator safety issues or equipment deficiencies? (b) If yes, please explain. (c) Has the Ministry of Labor visited the site? (d) If yes, please list any comments they made.	☐ Yes ☑ No ☐ Yes ☑ No
18. Is the public allowed on the dam?	☑ Yes ☐ No
19. (a) Are there any public safety concerns? (b) If yes, please explain (c) Is vandalism a problem? Please elaborate.	☐ Yes ☑ No ☐ Yes ☑ No
20. What signage is provided at this dam?	☐ Danger – Fast Water ☐ No Trespassing ☐ No Swimming ☑ Other unstable ice
21. (a) Is there a debris boom upstream of the dam? (b) If yes, is it chained (logs) or cable-strung (manufactured)? (c) Is it permanent or seasonal? (d) Is there a safety boom upstream? (e) Is it permanent or seasonal?	☐ Yes ☑ No ☐ Chained ☐ Cable strung ☐ Permanent ☐ Seasonal ☐ Yes ☐ No ☐ Permanent ☐ Seasonal trash rack
22. What structural aspects of the dam do you inspect during operational visits?	no operations required - monthly inspections during summer - stop logs removed in fall & replace in summer

23. Log Settings	
(a) What is the normal regulated water level	Gauge CGD local
(b) How many logs are usually in for the normal summer setting?	
(c) How many logs are normally removed for the winter drawdown condition?	3 logs x 6"
(d) How many logs can actually be removed in an emergency?	
(e) Is the bottom log fixed in place and not removed?	☐ Yes ☐ No

Part III - Hydraulic Discharge and Operating Facilities

A Discharge Facilities	
24. (a) Is a rating curve/table available for this site?	☐ Yes ☑ No
(b) Have any structural or channel modifications been made since the date on the rating table? (e.g., different size stoplogs, additional stoplogs, shaved stoplogs, dredging, etc.)	☐ Yes ☑ No
(c) If yes, please describe these modifications and how they will affect the rating table?	
25. (a) Does fully open represent lifting the gates clear of the deck?	☐ Yes ☐ No ☑ Not applicable
(b) If no, can they be easily lifted clear of the deck during an emergency?	☐ Yes ☐ No ☑ Not applicable
26. (a) Have all log sluices and/or all gate sluices ever been fully opened? (b) If yes, under what headwater elevation and when? (c) If no, what is the constraint?	☐ Yes ☑ No ☐ Not applicable silt
B Operating Equipment	
27. Type of equipment used to operate the discharge facilities:	hand
(a) Sluice Operation	☐ crab winch ☐ spud winch ☐ other - specify with: ☐ diesel ☐ electric ☐ hand ☐ other - specify

(b) Log Chutes and other outlet works.	☐crab winch ☐spud winch ☐other - specify with: ☐diesel ☐electric ☑hand ☐other - specify
28. (a) Is primary (pole) power available at the site? (b) Is auxiliary power available? (c) If yes, specify source.	☐Yes ☑No ☐Not applicable ☐Yes ☑No ☐Not applicable
29. (a) Is the discharge facility operating equipment located at the site (keys, winch handles, chain falls, etc.)? (b) If no, where are they located? (c) Is there more than one set?	☐Yes ☐No ☑Not applicable ☐Yes ☐No
30. (a) If the gates are automated, is the operation remotely controlled? (b) If yes, from where?	☐Yes ☐No ☑Not applicable
31. (a) Have any backup provisions been made should the equipment fail? (b) If yes, what are the provisions? (c) If yes, is the backup located on site? (d) If no, where is backup located?	☐Yes ☐No ☑Not applicable ☐Yes ☐No
32. If the backup is located off-site, how much more time is required to achieve maximum discharge?	hrs
33. (a) Has the mechanical equipment ever failed? (b) If yes, when did the failure occur? (c) What was the nature and extent of the failure? (d) Has it been satisfactorily repaired?	☐Yes ☐No ☑Not applicable ☐Yes ☐No
C Operating Problems	
34. (a) Are there problems that may reduce the number of stop logs which can be removed or the number of gates that can be opened during normal or flood conditions? (b) If yes, please describe.	☑Yes ☐No ~~☑Not applicable~~ No logs can be removed under increased waterlevels

35. (a) Has debris blockage ever occurred at this site? (b) If yes, at what time of the year does blockage occur? (c) What was the nature & extent of the blockage?	☑ Yes ☐ No ☐ Not applicable ☐ All the time only ☐ During spring only ☑ During floods corn rubbler small branches
36. Is there potential for debris from upstream to interfere with operations at the site under: (a) Normal Operation (b) Flood/Emergency Operation (c) If the answer to (a) or (b) is yes, please describe the situation.	☑ Yes ☐ No ☐ Not applicable ☑ Yes ☐ No ☐ Not applicable
37. (a) Is there a debris management program in place (e.g. debris boom, regular removal of debris, etc.)? (b) If yes, briefly describe program.	☑ Yes ☐ No Cleaned when observed
38. (a) Do ice jams affect this site? (b) Are there special operations to accommodate ice jam inflows? (c) Do ice jams block/hinder discharge facilities? (d) Do ice jams break booms? (e) If answer to any of the above is yes, please describe the situation.	☐ Yes ☑ No ☐ Yes ☑ No ☐ Yes ☑ No ☐ Yes ☑ No
39. Has an ice sheet formation been observed: (a) in the headpond or reservoir area? (b) against the intake headworks? (c) against the gate sluices? (d) against the log sluices? (e) against gravity walls/bulkheads?	 ☑ Yes ☐ No ☑ Yes ☐ No ☐ Not applicable ☐ Yes ☐ No ☑ Not applicable ☑ Yes ☐ No ☐ Not applicable ☐ Yes ☐ No ☑ Not applicable
40. (a) Are there any measurements or other estimates of the ice thickness? (b) If yes, please indicate these.	☐ Yes ☑ No
41. What is the duration of the headpond/reservoir ice cover (months)?	Jan to MAR
42. Is the frozen headpond generally covered with snow?	☐ Yes ☐ No ?
43. (a) Are any photographs of the headpond ice conditions available? (b) If yes, where are they located and when were they taken?	☐ Yes ☑ No

44. (a) Are there any other observations regarding ice cover? (b) If yes, please describe.	☐Yes ☑No
45. (a) What is the deck surface? (b) Describe snow/ice removal concerns.	☐ Concrete ☐ Wood ☐ Metal grating NA
Part IV – Past Dam Incidents	
46. Describe any past dam incidents (such as seepage, overflow during flooding, sinkholes in the headpond, washout of an abutment, etc.)	- erosion of toe bank at channel in 2008 - over topping over 1/4 of embankment
Part V –EPP Information	
47. Please provide the following emergency contact phone numbers. (a) Dam Operator (b) Alternate Dam Operator (c) District Emergency Response Coordinator (d) Regional Engineer (e) Provincial Response Center (f) OPP (g) Medical Emergencies	Name Office # Home # Cell #
48. (a) Are there permanent residents living within 0.5 km downstream of the dam? (b) If yes, please indicate their names and phone numbers.	☐Yes ☑No Name Phone #
49. (a) Is there an access road to this site? (b) Who maintains the access road to the site? (c) Is this access road plowed in the winter and spring?	☑Yes ☐No UTRCA / Pond committee ☐Yes ☑No ☐Not applicable

50. (a) Is there emergency equipment available at the site such as life preservers and a first-aid kit? (b) If not available at the site, where are the nearest available ones?	☐Yes ☑No
51. Note and describe any physical features that use you use to cue yourself that water levels are abnormal (both during flood and drought).	

Discharge Facilities

(one line for each discharge structure - sluices, spillways, turbines, etc.)

Facility	Structure					Rating Table		Operation			
	Number/ ID	Width (m)	Crest/ Sill Elev. (m)	Log Height (m)	Capacity (m^3/s)	Table No.	Date	Log Sluices			Gate Sluices[1] Yes/No/ Unknown
								Logs Per Sluice	Logs that can be Removed		
									Normal Condition	Emergency Condition	
emergency spillway – grass											
bottom drain drop-inlet with logs								6" x ?			

1- Can gates be fully opened under emergency conditions? If no, to what percentage can they be opened?

Drawings

Plot Scale PLOTSCALE
Jul 30 , 2007 , 3:49pm Login name: Zhao114056
Drawing Name: R:\P14504\Civil\Aerial Dwgs\Embro\14504-EM-001 and 002 Embro.dwg

DRAWING No. 14504-EM-001 B

0 100 200 300 400 500 Metres

SCALE 1:5000 (plotted as A1 size sheet)

DATE	NO.	ISSUE / REVISION	CH.	APP.	APP.
JUL. 27, 2007	B	ISSUED WITH FINAL DAM SAFETY ASSESSMENT REPORT	B.C.	P.L.	M.M.
FEB. 23, 2004	A	ISSUED WITH DRAFT DAM SAFETY REPORT	B.C.	P.L.	M.M.

ACRES INTERNATIONAL

UPPER THAMES RIVER
CONSERVATION AUTHORITY

DAM SAFETY PROGRAM – REVIEW OF DAMS OWNED/OPERATED BY UTRCA AND ABCA

DESIGN
PREPARED ______
CHECKED ______
DRAWING
PREPARED ______
CHECKED ______
PROJECT DISCIPLINE LEAD
PROJECT ENGINEER
PROJECT MANAGER

EMBRO DAM
KEY PLAN

SCALE

ACRES PROJECT NO.
P14504.02

DRAWING NO.
14504–EM–001

REVISION
B

0 6 12 18 24 30 Metres

SCALE 1:300 (plotted as A1 size sheet)

LEGEND

● BH1 IDENTIFICATION NUMBER
2003 YEAR OF INVESTIGATION
BOREHOLE

Plot Scale PLOTSCALE
Jul 30 , 2007 , 3:49pm Login name: Zhao114056
Drawing Name: R:\P14504\Civil\Aerial Dwgs\Embro\14504-EM-001 and 002 Embro.dwg

DRAWING No. 14504-EM-002 B

JUL. 27, 2007	B	ISSUED WITH FINAL DAM SAFETY ASSESSMENT REPORT	B.C.	P.L.	M.M.
FEB. 23, 2004	A	ISSUED WITH DRAFT DAM SAFETY REPORT	B.C.	P.L.	M.M.
DATE	NO.	ISSUE / REVISION	CH.	APP.	APP.

ACRES
ACRES INTERNATIONAL

UPPER THAMES RIVER
CONSERVATION AUTHORITY

DAM SAFETY PROGRAM – REVIEW OF DAMS OWNED/OPERATED BY UTRCA AND ABCA

DESIGN
PREPARED ____________
CHECKED ____________
DRAWING
PREPARED ____________
CHECKED ____________
PROJECT DISCIPLINE LEAD
PROJECT ENGINEER
PROJECT MANAGER

EMBRO DAM
SITE PLAN

SCALE
ACRES PROJECT NO. P14504.02
DRAWING NO. 14504–EM–002
REVISION B