Upper Thames River Conservation Authority

UPPER THAMES RIVER
CONSERVATION AUTHORITY

DAM SAFETY ASSESSMENT REPORT FOR
Fullarton Dam
Final
August, 2007

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Executive Summary

The Fullarton Dam is located approximately 2 km south of the town of Fullarton on an unnamed tributary of the North Thames River. The dam and reservoir were built for recreational purposes and is adjacent to Country Road 163. The Fullarton Dam is a small fill dam approximately 100 m long which, at the time of the visit, had a head of water of approximately 3 m acting across the dam. Freeboard was approximately 0.7 m. The dam is provided with an emergency spillway.

The dam controls a very small drainage area of 4 km^2 comprising mostly agricultural land. The conservation reservoir surface area is small and is impounded by a low earth-fill embankment dam located at the northern end of the reservoir. Flow releases from the dam outlet enter a narrow channel, and flow in a northeasterly direction for approximately 0.45 km before entering the main stem of the North Thames River. The North Thames River continues to flow in a northerly direction some 16.5 km before reaching St. Marys Dam in the town of St. Marys.

The discharge facilities at the dam consist of a concrete drop inlet structure with a set of stop logs at the upstream face and an inverted V-shaped trashrack anchored to the top of the inlet. There is an emergency spillway located on the right[*] or east bank. This is a lower section at the end of the embankment dam which is covered with cable-connected concrete blocks. The mouth of the spillway measured 9.5 m in length and appeared to be in good condition. The emergency spillway has a grassed discharge channel that runs parallel to the creek before joining it.

The upland terrain is rolling and locally hummocky. Relief is about 35 m. Glacial features and the North Thames River control the physiography.

Fullarton pond has a surface area of 0.02 km^2. The embankment dam is approximately 3.4 m high and impounds a total estimated storage volume of 0.02 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.

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[*] The orientations of all structures are given in terms of left and right as looking downstream. All geological orientations are given in terms of dip direction/dip degree with respect to True North.

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, 3-day summer storm event
- the dam is overtopped during passage of the IDF and has inadequate freeboard. The dam is deemed to have adequate spillway capacity to pass the IDF if the crest of the dam is raised.
- upstream and downstream slopes do not meet acceptance criteria during normal conditions.

The costs associated with the maintenance repairs recommended to ensure the ongoing safe operation of this dam are in the order of $21,000.

Figure ES-1

Fullarton Dam

Description: Earth Embankment

Original Construction: 1955

Last Upgrade: 2000

Last Repairs: 2000

Height: approx. 3.4 m

Length: approx. 100 m

Reservoir Area: 0.02 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, 3-day summer storm

Spillway Capacity: Adequate if crest of dam raised

Issues

General Condition: Crest is too low

Stability: Upstream and downstream slopes do not meet all criteria

Safety and Operating Issues

Operations: Not operated

Signage: Inadequate

Debris Boom: Not applicable

Fall Arrest Systems: Not applicable

Recommendations

- Raise crest of dam to prevent overtopping and to provide adequate freeboard.
- Install additional signs to satisfy Ministry of Natural Resources’ draft standards.
- Install riprap on upstream slope to prevent erosion.
- Regrade downstream slope on the right bank to fill in sinkholes.
- Test the emergency preparedness plan.
- Monitor erosion of concrete on inside faces of inlet structure.
- Survey upstream slope, particularly below waterline.
- Perform shear strength tests on existing samples to assess angle of internal friction.

Costs $21,000

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 Fullarton Dam located on an unnamed tributary of the North Thames River near the town of Fullarton, Ontario (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 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

Figure 1.1
Upper Thames River Conservation Authority
Dam Safety Assessment Report

Aerial Photograph of Dam and Surrounding Area

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BACK OF FIGURE 1.1

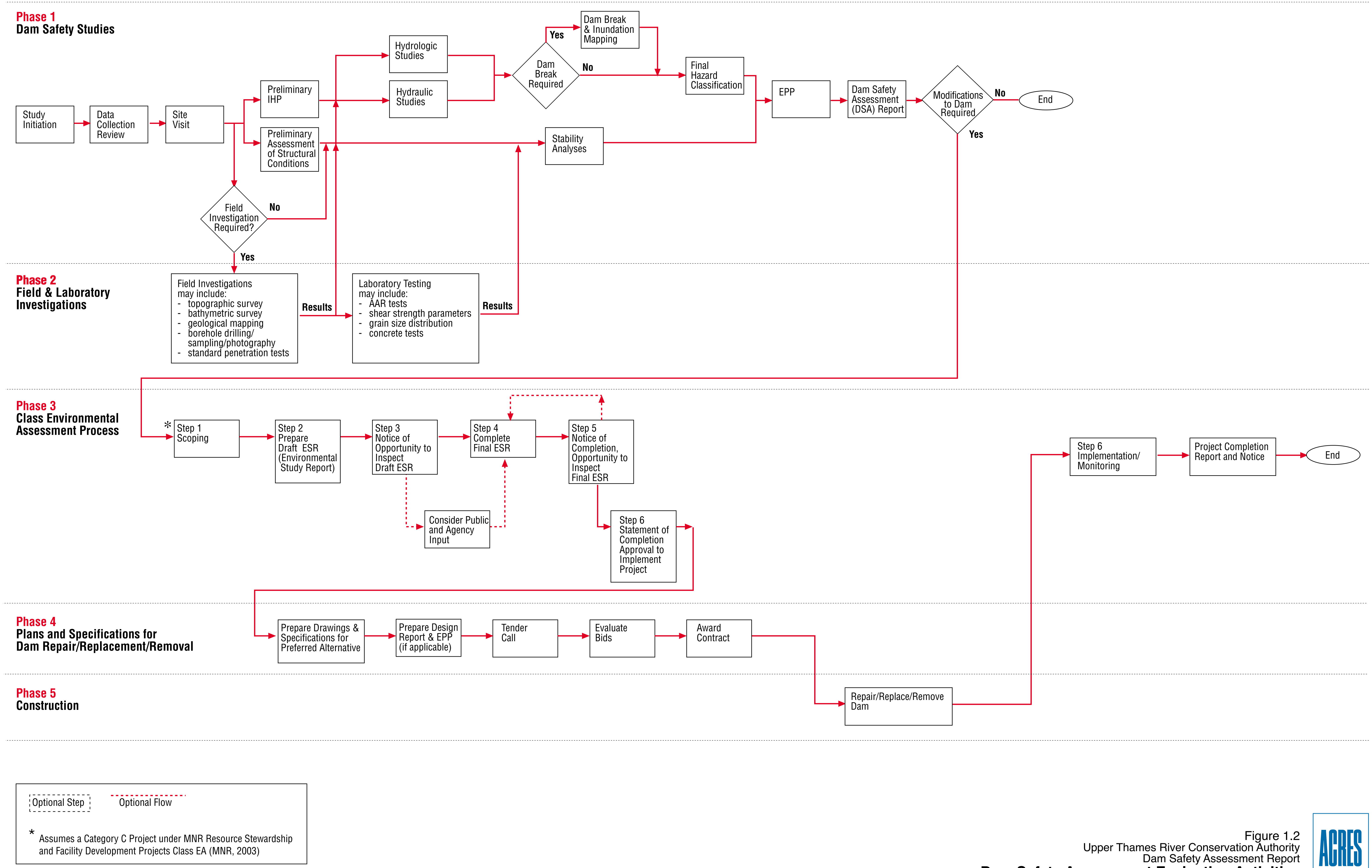

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Figure 1.2
Upper Thames River Conservation Authority
Dam Safety Assessment Report
Dam Safety Assessment Evaluation Activities

ACRES International

FIGURE 1.2 – BACK OF PAGE

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.

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 Fullarton Dam Safety Assessment

The Fullarton Dam is located on an unnamed tributary of the North Thames River near the town of Fullarton, Ontario, as shown in Figure 1.3.

Characteristics of this dam are shown in Table 1.4.

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
Fullarton Dam	Off County Road 163a	4.0	0.02	3.4	~100	1 drop inlet with a concrete pipe outlet + 1 emergency spillway

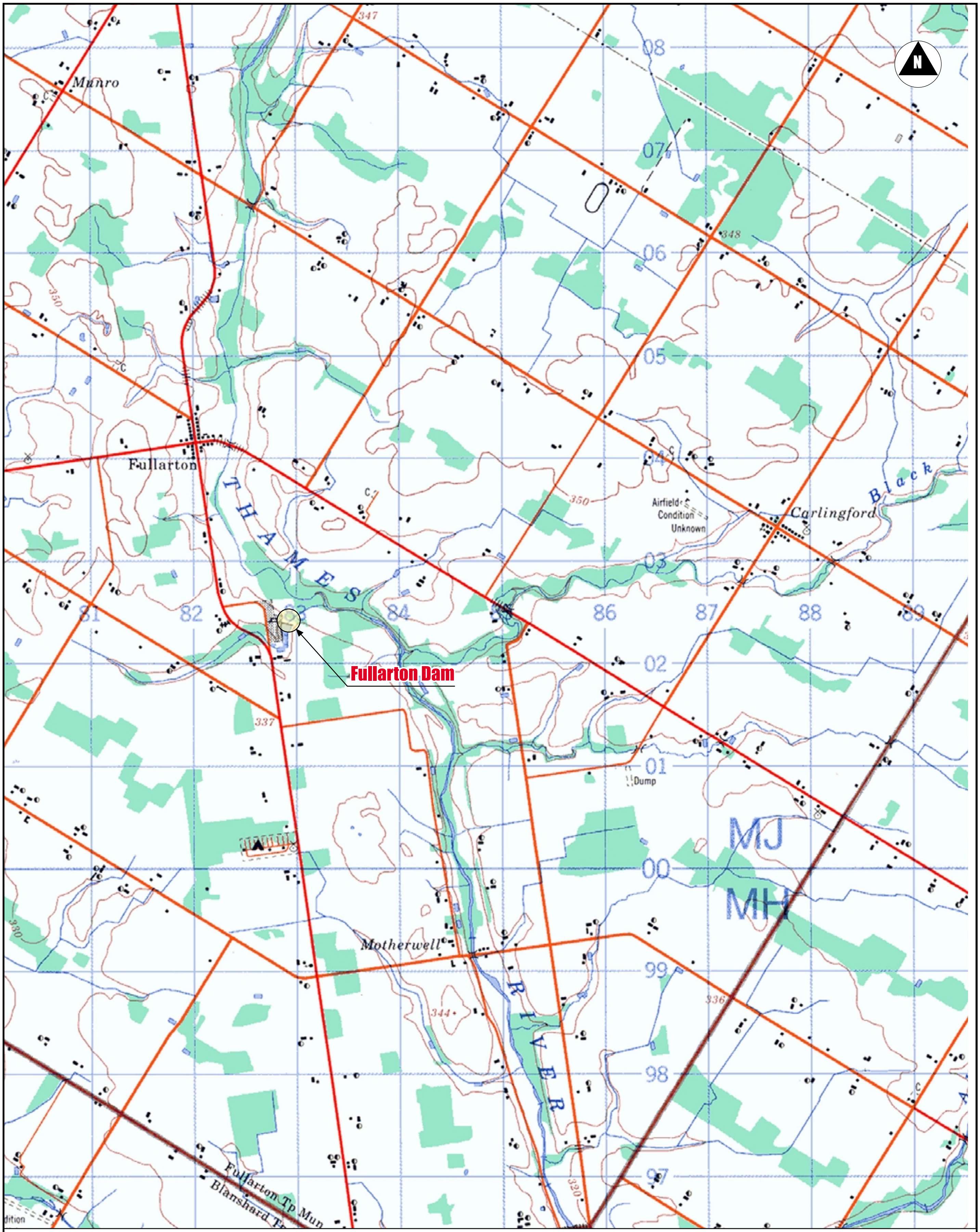

Figure 1.3
Upper Thames River Conservation Authority
Dam Safety Assessment Report
Location of Fullarton Dam

FIGURE 1.3 - BACK

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 Fullarton 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 Fullarton Dam

2 The Fullarton Dam

2.1 History[*]

Initial steps toward creating the Fullarton conservation area were taken in October 1952, when J. Wilson Brown reported that 77 acres of property, containing a good trout stream, on the Perth County road south of the village, were for sale. The property was purchased in 1953 from the Alonzo Hart estate, and development was started in September 1955.

An earth-fill dam, 9 ft high and 300 ft long, was built and a 5-acre pond created. The dam was completed in November 1955 and the pond in the spring of 1958.

In its development plans, the Authority enjoyed the cooperation of Fullarton Township and the County of Perth. In 1962, the county sold to the Authority, for the sum of $1.00, approximately 5-1/2 acres of land across from the conservation area, for use as a roadside park. Downed timber and weeds were removed, and in 1964, the property was used as a test plot for the control of thorn trees, a scourge of farm lands in that section.

In 1964, the Authority turned over to the township 4 acres of property, under a 99-yr lease, for creation of a recreation center as a Centennial project. The Authority also assisted by providing a water supply for the center and shared in the cost of constructing rest rooms. The center was officially opened on June 25, 1966, when K. E. Lantz, Assistant Deputy Minister of Agriculture for Ontario, unveiled an inscribed plaque.

During the winter of 1966/67, a large quantity of silt was removed from the pond to increase fish habitat. The pond was deepened and, in the spring of 1967, was restocked with trout.

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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
- previous internal inspection reports
- Ontario Geological Survey maps and documents
- historical records
- meteorological data from selected stations
- data from selected streamflow gauging stations from Water Survey of Canada (WSC)
- selected topographic maps (1:50 000-scale).

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
Fullarton	2001	Drawing	R.P. of UTRCA	Dam Hazard Identification, Fullarton Dam (WECS Program)	-	Dam	No	Yes	-	8-Nov-02	-	
Fullarton	2000	Inspection Report	Chris Tasker and Al Merry (UTRCA)	-		Dam	No	Yes	No	8-Nov-02	-	2 page report
Fullarton	1985	Inspection Report	B. Bevan and J.C. Campbell (UTRCA)	Fullarton Inspection Report		Multi	No	No	Yes	-	-	
Fullarton	1985	Response Summary	B. Bevan (UTRCA)	Summary of UTRCA Responses RE: 1985 MNR Inspections	WM.2.1	Multi	No	Yes	No	8-Nov-02	-	
Fullarton	1982	Inspection Report	J. Jilek and P. Bane (UTRCA)	-		Multi	No	No	Yes	-	-	low areas on crest should be filled

4 Comprehensive Site Inspections and Condition Assessments

4 Comprehensive Site Inspections and Condition Assessments

4.1 Introduction

A site evaluation of the Fullarton Dam was made on November 14, 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 several locations close to Fullarton damsite. The Avon River gauge is geographically closest to the site. From the table, it can be seen that 0.80 mm of rain fell on November 14, the day of the civil/geotechnical inspection, and precipitation occurred on November 10 and 11 (total of 1.3 mm during the week prior to the civil/geotechnical inspection).

4.3 Record of Observations

4.3.1 General Description

The Fullarton Dam is located approximately 2 km south of the town of Fullarton on an unnamed tributary of the North Thames River. The conservation dam is close to two baseball diamonds which are located on a hill. An agricultural processing factory is located at the south (upstream) end of the reservoir (Photo 1). The dam and reservoir were built for recreational purposes and are adjacent to Country Road 163.

Table 4.1

Summary of Daily Precipitation Records from UTRCA's HEC-DSS Database

Year	Day, Month	Trout Creek (mm)	St. Marys (mm)	Avon River (mm)
2002	1-Nov	0.40	0.00	1.00
	2-Nov	7.00	0.00	0.20
	3-Nov	3.40	0.00	0.00
	4-Nov	0.00	0.00	0.00
	5-Nov	1.00	0.00	0.50
	6-Nov	7.60	0.00	1.20
	7-Nov	0.20	0.00	0.00
	8-Nov	0.00	0.00	0.00
	9-Nov	0.00	0.00	0.00
	10-Nov	32.20	13.50	0.50
	11-Nov	1.80	0.50	0.80
	12-Nov	0.00	0.00	0.00
	13-Nov	0.00	0.00	0.00
	14-Nov	2.40	0.50	0.80
	15-Nov	1.20	0.00	0.50
	16-Nov	0.00	0.00	0.00
	17-Nov	0.00	0.00	0.00
	18-Nov	2.40	0.00	1.50
	19-Nov	1.60	0.00	7.80
	20-Nov	0.00	0.00	0.00
	21-Nov	0.00	0.00	0.00
	22-Nov	0.80	0.00	2.20
	23-Nov	0.00	0.00	0.00
	24-Nov	0.00	0.00	0.00
	25-Nov	4.00	0.00	1.00
	26-Nov	0.00	0.00	0.00
	27-Nov	0.00	0.00	0.00
	28-Nov	0.00	0.00	0.00
	29-Nov	0.60	0.00	1.00
	30-Nov	2.80	0.00	1.20

Note: No recordings for Waubuno Creek station during the month of November 2002.

The Fullarton Dam is a small fill dam approximately 100 m long (Photo 2) which, at the time of the visit, had a head of water of approximately 3 m acting across the dam. Freeboard was approximately 0.7 m. The dam is provided with an emergency spillway. A track leads off the right end of the dam.

4.3.2 Hydrotechnical Aspects

The dam controls a very small drainage area of 4 km^2 comprising mostly agricultural land. The conservation reservoir surface area is small and is impounded by a low earth-fill embankment dam located at the northern end of the reservoir (Photo 2). Flow releases from the dam outlet enter a narrow channel (Photo 3), and flow in a northeasterly direction for approximately 0.45 km before entering the main stem of the North Thames River. The North Thames River continues to flow in a southerly direction some 16.5 km before reaching St. Marys Dam in the town of St. Marys.

The reservoir has a limited fetch and, therefore, has negligible wind-generated wave heights. A significant volume of the original reservoir has been filled with sediments, and the water quality does not appear to be very good based on a visual inspection.

The discharge facilities at the dam consist of a concrete drop inlet structure with a set of stop logs at the upstream face and an inverted V-shaped trashrack anchored to the top of the inlet (Photo 1). At the time of the site visit, water was flowing over the stop logs. The dam is approximately 3.4 m high from its crest to the invert of the downstream channel/creek (Photo 4). There is an emergency spillway located on the right[*] or east bank (Photo 5). This is a lower section at the end of the embankment dam which is covered with cable-connected concrete blocks (Photo 6). The concrete blocks are cable-connected across the mouth of the spillway which measured 9.5 m in length and appeared to be in good condition. The emergency spillway has a grassed discharge channel that runs parallel to the creek before joining it. The spillway channel has variable slopes and widths, and is 2.4 m wide at its narrowest section. The upper reaches of the emergency spillway channel were

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[*] The orientations of all structures are given in terms of left and right as looking downstream. All geological orientations are given in terms of dip direction/dip degree with respect to True North.

clear (Photo 7) and grassed. The lower portion of the spillway was not well-defined and was overgrown with larger brushes.

The entire damsite is founded on overburden, and the discharge from the drop inlet and circular pipe has formed a deep pool at its outlet point (Photo 3). The channel is armored with large cobbles and boulders around the outlet, and both banks are lined with grasses, brushes and mature trees. The channel downstream appears to slope relatively gently, and there is a small wooden footbridge (Photo 8) (planks) that crosses it some distance downstream. There is no evidence that high tailwater levels are present or that a problem exists at this dam. The dam was overtopped in the summer of 2000, and the high outflows eroded the embankment in the area of the outflow pipe. There are no permanent dwellings or development in the immediate vicinity of the downstream reach of the discharge channel.

4.3.3 Geotechnical Aspects

Both reservoir banks and both banks downstream from the dam consisted of overburden. No bedrock was observed.

4.3.3.1 Upstream Slope

The upstream slope (Photo 9) was not protected with riprap and was overgrown with cottontail growth. Benching due to erosion by wave action had occurred locally up to 0.5 m. No sloughing, displacement, settlement, cracking or sinkholes were evident. No floating debris had accumulated.

4.3.3.2 Crest

The crest (Photo 10) showed no cracking, displacement, sinkholes or settlement. Camber was not apparent.

4.3.3.3 Downstream Slope and Leakage

The downstream slope (Photo 4) at the outer thirds of the dam was flatter than the center third and the latter was provided with rock-fill slope protection material. This extended from crest to toe. The reason for the different slopes and the rock fill was for embankment protection.

The concrete conduit was partially blocked (Photo 11). The right-side downstream slope was noted to be raised somewhat, very irregular, hummocky and densely grassed. A few sinkholes were evident up to 0.2 m wide and 0.4 m deep. The cause of this condition is unknown. No other signs of internal erosion were evident. It may be due to unauthorized dumping on the slope. The left-side slope was normal and in good condition.

Locally, the ground on the right side of the downstream slope was slightly mushy and soft, suggesting high groundwater. No leakage was seen. No leakage was evident at the contact between the concrete sluice and the fill.

4.3.3.4 Abutments

Nothing unusual, i.e., leakage, cracking or movement was noted in the abutments.

4.3.3.5 Instrumentation

At the time of inspection, no instrumentation was seen. A piezometer was installed at a later date as part of the investigation program (see Section 5).

4.3.4 Civil/Structural Aspects

The intake structure consisted of a three-sided concrete drop box with a set of stop logs at the upstream face (Photo 12). At the time of inspection, water was flowing over the stop logs. The trashrack was constructed of galvanized steel and appeared to be in good condition and well-anchored to the concrete intake. The concrete intake structure showed some signs of deterioration. A portion of the upstream right corner (upstream from the stop log gains) was delaminated from the top surface to a depth of approximately 300 mm below water level. Erosion of the concrete has taken place on the downstream wall within the intake above the precast concrete pipe (Photo 13).

Stop logs consist of sets of two 38-mm x 140-mm (2-in. x 6-in.) planks vertically fastened together. The condition of the stop logs was difficult to assess due to the volume of water entering the structure at the time of inspection. No stop logs were present on shore, and it is assumed that all logs were in place in the gains.

The outlet conduit consisted of a precast concrete pipe with a measured inside diameter of 762 mm (30 in.) (Photo 11). The downstream edge of the pipe was chipped and damaged from the removal of a pipe section. Riprap had been placed in the end of the outlet pipe effectively blocking about half of the pipe. The outlet pipe was in good condition with the individual precast elements visibly aligned through the dam structure.

No signs are posted indicating the potential hazards of the dam. This dam is accessible to the public, and shows indications that a hiking trail passes downstream of the dam by means of tree markers and a makeshift pedestrian bridge (Photo 8).

5 Site Investigations

5 Site Investigations

Fullarton Dam was investigated with two boreholes which were drilled on November 24 to 26, 2003. The boreholes were located just left of the culvert in the embankment as shown on Drawing 14504-FT-002. Borehole BH-1 was located near the dam centerline, while Borehole BH-2 was located on the downstream slope. A CME 55 hollow-stem auger was used for drilling. Close-spaced sampling was done.

Borehole BH-1 penetrated the embankment fill and foundation material to a total depth of 6.85 m. Similarly, Borehole BH-2 penetrated embankment fill and dam foundation material to a total depth of 4.57 m. Assuming a crest elevation of 100 m, the foundation level was found to be at el 96.18 m in BH-1 (3.7 m below crest) and at el 97.01 m in BH-2. A standpipe piezometer was installed in BH-1.

The borehole logs attached present 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 and grain-size distribution. Results are shown in Table 5.1, and grain-size plots and the plasticity charts are attached.

Table 5.1

Laboratory Test Results for Fullarton Dam

Bore-hole	Sample*	Depth (m)	% Moist	LL (%)	PL (%)	PI (%)	Gravel (%)	Sand (%)	Fines (%)
BH-1	AQ3	2.28-2.89	17.6	16	20	5			
BH-1	AQ4	3.05-3.65	10.9						
BH-1	AQ6	4.57-5.18					36.8	55.8	7.4
BH-2	AQ3	1.52-2.13		15	20	5			

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* In BH-1, AQ3 to AQ6 were in embankment fill. In BH-2, sample AQ3 was in the foundation.

In Borehole BH-1, four split-spoon samples were taken in the fill and four in the foundation, along with standard penetration tests (SPTs). In Borehole BH-2, a grab sample and a Shelby tube were taken in the embankment fill and four split-spoon samples and SPTs were taken in the foundation.

Sampling indicates that the fill comprises brown and gray clay. The material is classified as CL and ML. SPT ‘N’ values in the fill range from 8 to 13, i.e., stiff consistency. Liquid limits, plasticity limits and plasticity index of the fines in the embankment are 20%, 16% and 5%, respectively, indicating low plasticity. Moisture content of the embankment fill ranges from 11% to 18% approximately.

Sampling in the foundation indicates variable material ranging from silty, gravelly glacial till underlain by well-graded gravel and sand (BH-2) to well-graded sand underlain by silty, sandy and gravelly glacial till (BH-1). SPT ‘N’ values range from 11 to 23 (medium) for the upper foundation material and are 35 for the lower foundation material (dense). Liquid limits, plasticity limits and plasticity index for the foundation material are 20%, 15% and 5%, respectively, indicating low plasticity.

Atterberg limits for the embankment fill and the foundation material are almost identical.

The groundwater level in the dam was found to be 2.5 m down from the crest at the time of the work.

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

ACRES
International

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

ACRES International

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

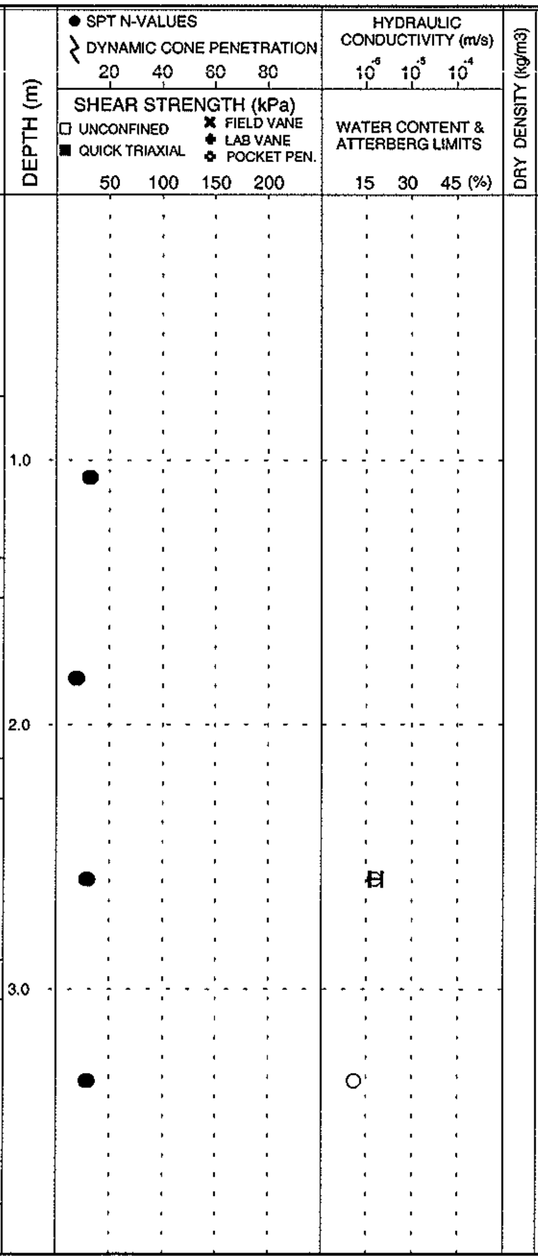
CLIENT: Upper Thames River Conservation Authority **HOLE:** FT BH1
PROJECT: Dam Safety Assessment **PAGE:** 1 **OF:** 3

SITE: Fullarton Dam

COORDINATES:	4.1m to left of culvert ctrline 2.0m d/s of dam centerline	**CONTRACTOR:**	Atcost Soil Drilling Inc.	**STARTED:**	24 Nov 2003
		DRILL TYPE:	CME 55	**FINISHED:**	24 Nov 2003
DIP DIRECTION:	0	**METHOD SOIL:**	Hollow stem auger	**INSPECTOR:**	D. Besaw
DIP:	90	**ROCK:**		**LOGGED BY:**	D. Besaw
ELEVATIONS		**CASING:**	Auger 200mm OD	**REVIEWED:**	BSinclair
DATUM:	Crest elev 100m assumed				
PLATFORM:				**DATE:**	18/3/04
GROUND:	99.95	**CORE:**			
END OF HOLE:	93.06				

See end of log for detailed groundwater measurements

ELEV. / DEPTH (m)	SYMBOL	DESCRIPTION	SAMPLE DEPTH	TYPE/ NUMBER	REC'Y (mm)	RET'D (mm)	BLOW COUNTS	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
99.91 0.0	▨	Embankment fill - brown and gray clay (CL-ML), stiff, trace organics, moist, low plasticity, homogeneous.						
			0.76 1.37	AQ 1	355	355	3 7 6	
			1.52 2.13	AQ 2	380	380	3 4 4	
			2.28 2.89	AQ 3	460	460	6 6 6	Piezo water level shown for 11/26/2003.
			3.04 3.65	AQ 4	430	430	5 7 5	
96.18 3.73		Foundation level - well-graded sand (SW-SM) with	3.81					



PIEZOMETER INSTALLATION

SAMPLING METHOD		***SHIPPING CONTAINER***	
A - Split Tube	E - Auger	N - Insert	R - Cloth Bag
B - Thin Wall Tube	F - Wash	O - Tube	S - Plastic Bag
C - Piston Sample	G - Shovel Grab	P - Water Content Tin	U - Wooden Box
D - Core Barrel	K - Slotted	Q - Jar	Y - Core Box
			Z - Discarded

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

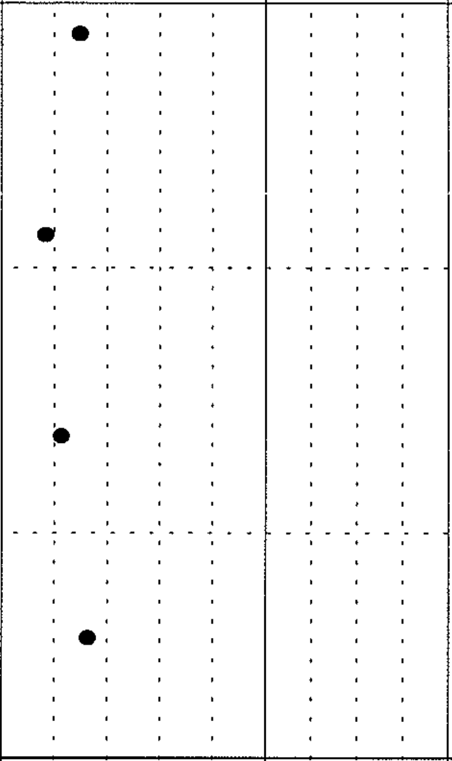
☐ Constant Head Test
▨ Falling Head Test
▩ Lab. Permeability

BOREHOLE REPORT

ACRES

CLIENT: Upper Thames River Conservation Authority
HOLE: FT BH1
PROJECT: Dam Safety Assessment
PAGE: 2 **OF:** 3

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
		subrounded gravel, 5-10% fines, medium to dense, wet.	4.42	AQ 5	330	330	7 15 15		●				
			4.57 5.18	AQ 6	405	405	8 8 11	5.0	●			See AQ6 gradation.	
			5.33 5.94	AQ 7	480	480	10 10 13	6.0	●				
93.66 6.25		Glacial till - gray silt (ML) with fine subrounded gravel and sand, hard, moist.	6.09 6.7	AQ 8	405	405	5 12 23		●			collected additional till sample from 6.09 - 6.85 with CME continuous sampler.	
93.06 6.85													



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

ACRES

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority **HOLE:** FT BH1
PROJECT: Dam Safety Assessment **PAGE:** 3 **OF:** 3

WATERLEVEL READINGS

11/24/2003 4:00:00 PM	3.0
11/26/2003 7:49:00 AM	2.52

NOTES/COMMENTS

1 Water Level Measurements

All water level measurements are referenced to ground level.

Reservoir level was 0.7m below crest for the 2.52 reading.

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2 Piezometer Installation

Surface - Flush mount cap embedded in Sakcrete

0.3-0.6 Bentonite chips
0.6-3.96 Bentonite slurry
3.96-5.18 Sloughed gravel and sand
5.18-6.09 Slottted screen in coarse sand pack
6.09-6.7 Coarse sand

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

BOREHOLE REPORT

CLIENT: Upper Thames River Conservation Authority
PROJECT: Dam Safety Assessment
HOLE: FT BH2
PAGE: 1 **OF:** 3

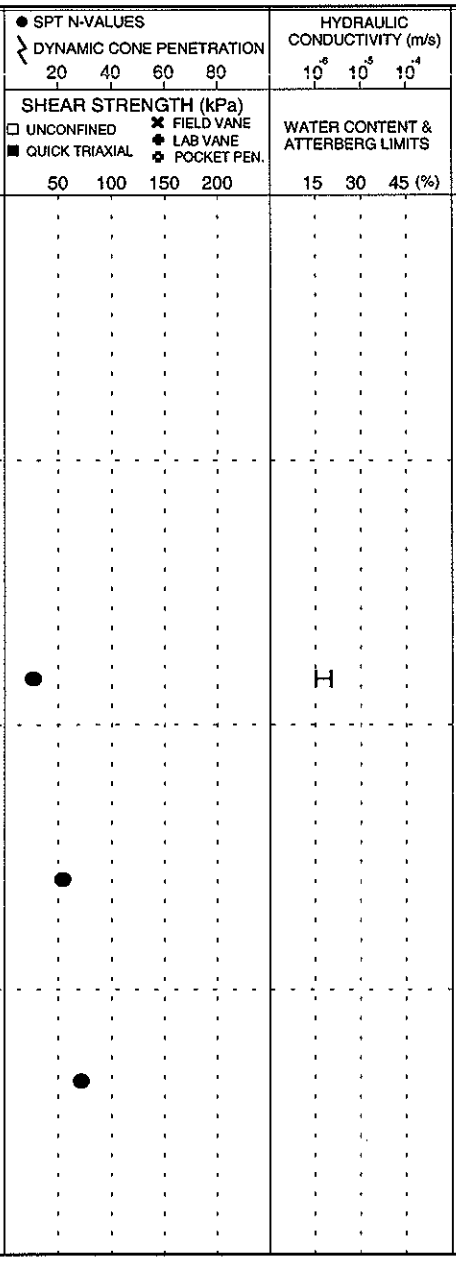
SITE: Fullarton Dam

COORDINATES: 6.2m to left of culvert ctrline 9.6m d/s of dam centerline
DIP DIRECTION: 0
DIP: 90
ELEVATIONS
DATUM: Crest elev 100m assumed
PLATFORM:
GROUND: 98.78
END OF HOLE: 93.83

CONTRACTOR: Atcost Soil Drilling Inc.
DRILL TYPE: CME 55
METHOD SOIL: Hollow stem auger
ROCK:
CASING: Auger 200mm OD
CORE:

STARTED: 26 Nov 2003
FINISHED: 26 Nov 2003
INSPECTOR: D. Besaw
LOGGED BY: D. Besaw
REVIEWED: BJ Sinclair
DATE: 18/3/04

ELEV. DEPTH (m) 98.4	SYMBOL	DESCRIPTION	SAMPLE DEPTH	TYPE/ NUMBER	REC'Y (mm)	RET'D (mm)	BLOW COUNTS	DEPTH (m)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0		Embankment fill - dark brown clay (CL), moist, low plasticity, trace organics	0 0.61	ES 1	5kg				
			0.76 1.26	BO 2				1.0	Sample BO 2 is collected in 75 mm Shelby tube.
97.01 1.39		Foundation level - glacial till, brown and light gray silt (ML) with fine subrounded gravel, maximum size of 25mm, stiff to very stiff, moist.	1.52 2.13	AQ 3	420	420	3 4 7	2.0	
			2.28 2.89	AQ 4	480	480	4 6 16		
95.76 2.64		Gravel (GW) - well graded gravel with coarse sand, subrounded gravel fraction, maximum size40-50 mm, wet.	3.04 3.65	AQ 5	480	480	13 15 14	3.0	



DRY DENSITY (kg/m3)

PIEZOMETER INSTALLATION

SAMPLING METHOD		*SHIPPING CONTAINER*	
A - Split Tube	E - Auger	N - Insert	R - Cloth Bag
B - Thin Wall Tube	F - Wash	O - Tube	S - Plastic Bag
C - Piston Sample	G - Shovel Grab	P - Water Content Tin	U - Wooden Box
D - Core Barrel	K - Slotted	Q - Jar	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 **HOLE:** FT BH2
PROJECT: Dam Safety Assessment **PAGE:** 2 **OF:** 3

ELEV. / DEPTH (m)	SYMBOL	DESCRIPTION	SAMPLE DEPTH	SAMPLE TYPE/ NUMBER	SAMPLE RECY (mm)	SAMPLE RET'D (mm)	SAMPLE 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
93.83 4.57			3.96 4.57	AQ 6	530	460	12 13 15		●				
						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: FT BH2
PAGE: 3 **OF:** 3

WATERLEVEL READINGS

11/26/2003 10:30:00 AM
1.34

NOTES/COMMENTS

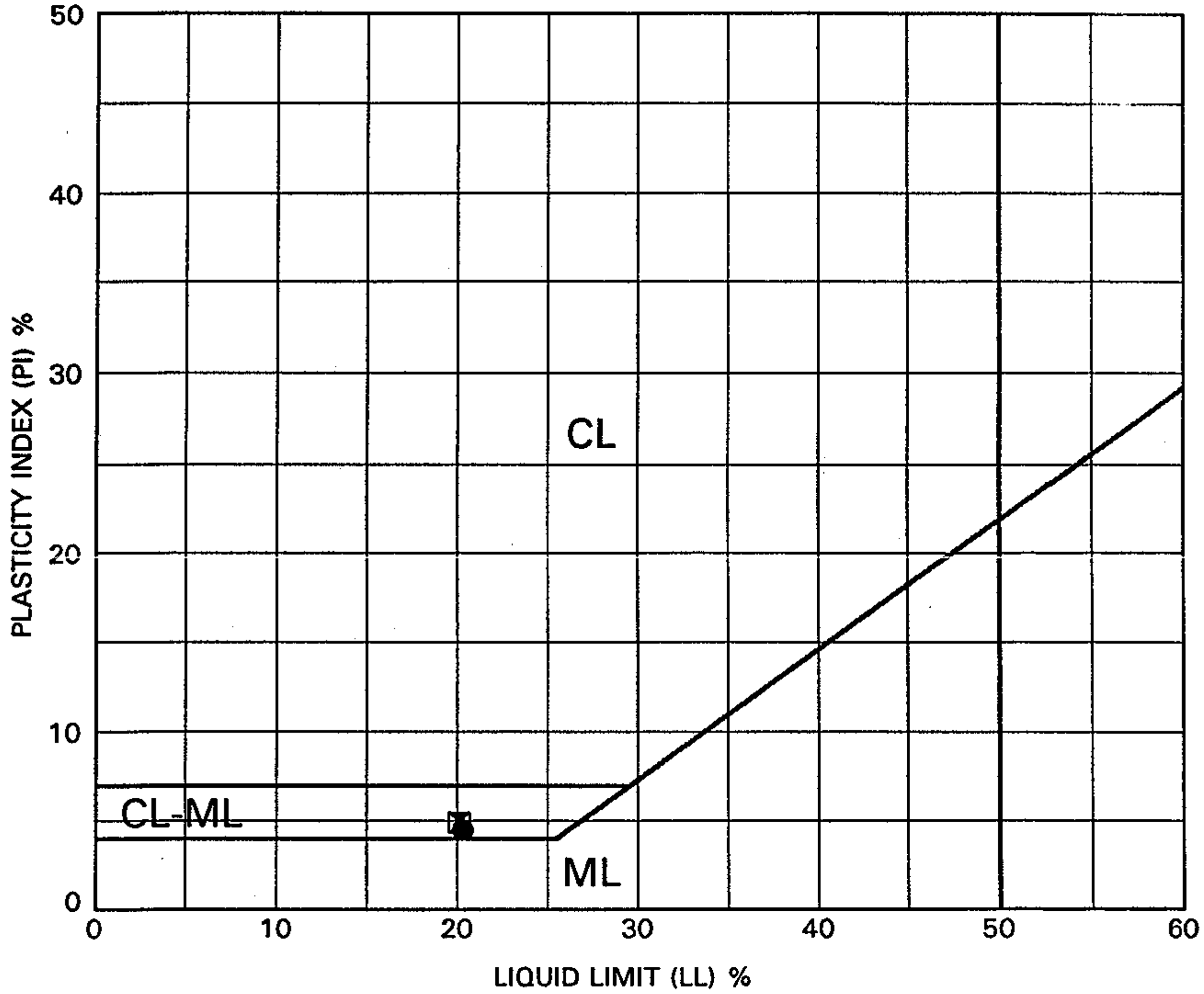
1 **Water Level Measurements**

All water level measurements are referenced to ground level.

.

2 **Piezometer Installation**

Piezometer not installed



LEGEND	BH	DEPTH (m)	LL%	PL%	PI%
●	FTBH1	2.28	20	16	5
☒	FTBH2	1.52	20	15	5

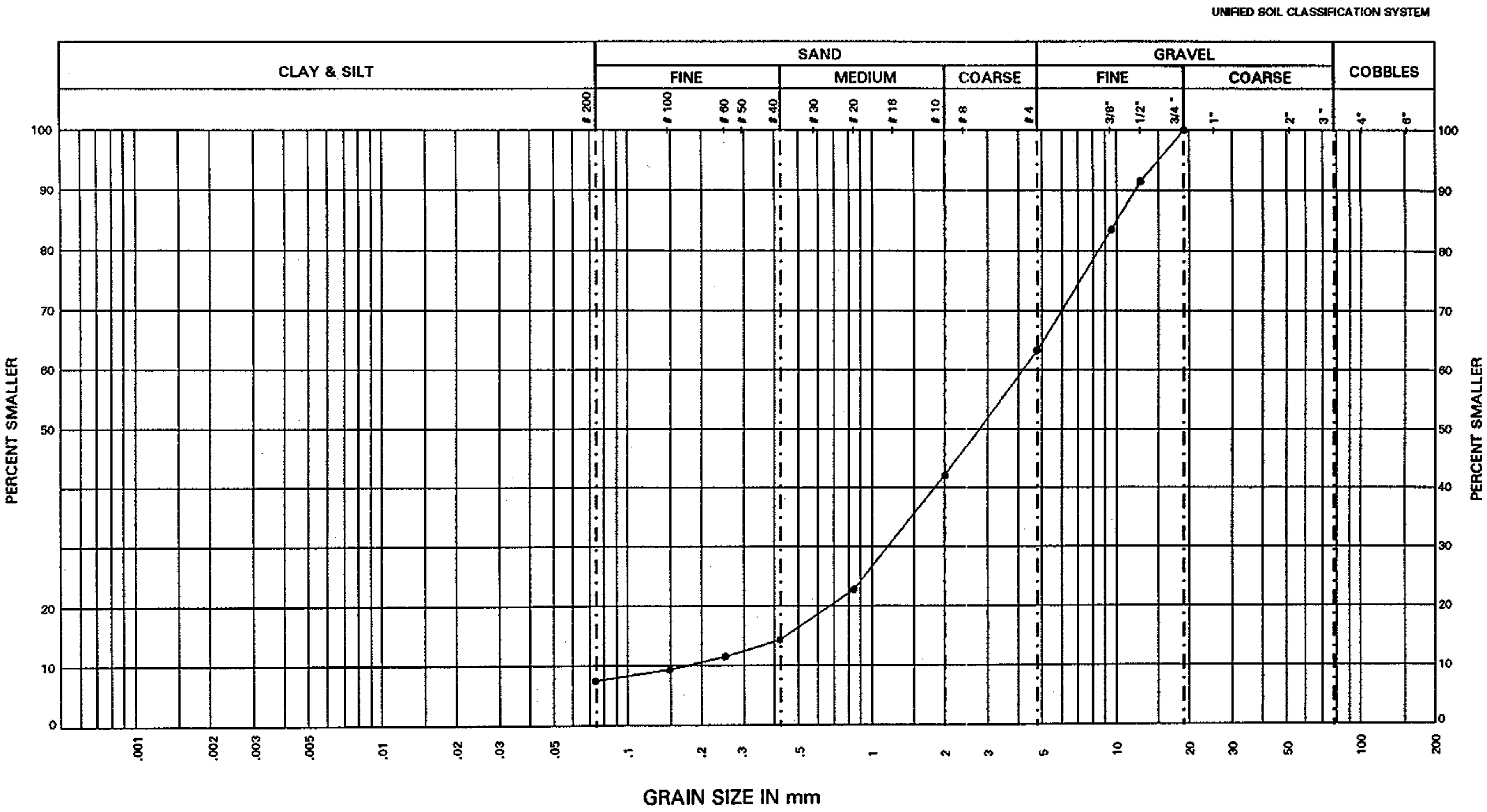
Geotechnical Investigations
Upper Thames Conservation Authority

NP = Non-Plastic

PLASTICITY CHART
Fullarton Dam

PROJECT NO. P1450404

ACRES



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

REMARKS: Fullarton 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.

For the Fullarton damsite, there is no gauge on the watercourse and the drainage area is so small that it cannot accurately be isolated from downstream (Thames) flow records; therefore, a frequency analysis on streamflow was not used. Application of transposed or regional runoff flood characteristics for dam safety use requires verification, which can be only accomplished by deterministic modeling. 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 Fullarton 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 used to establish CN values for the study basin.

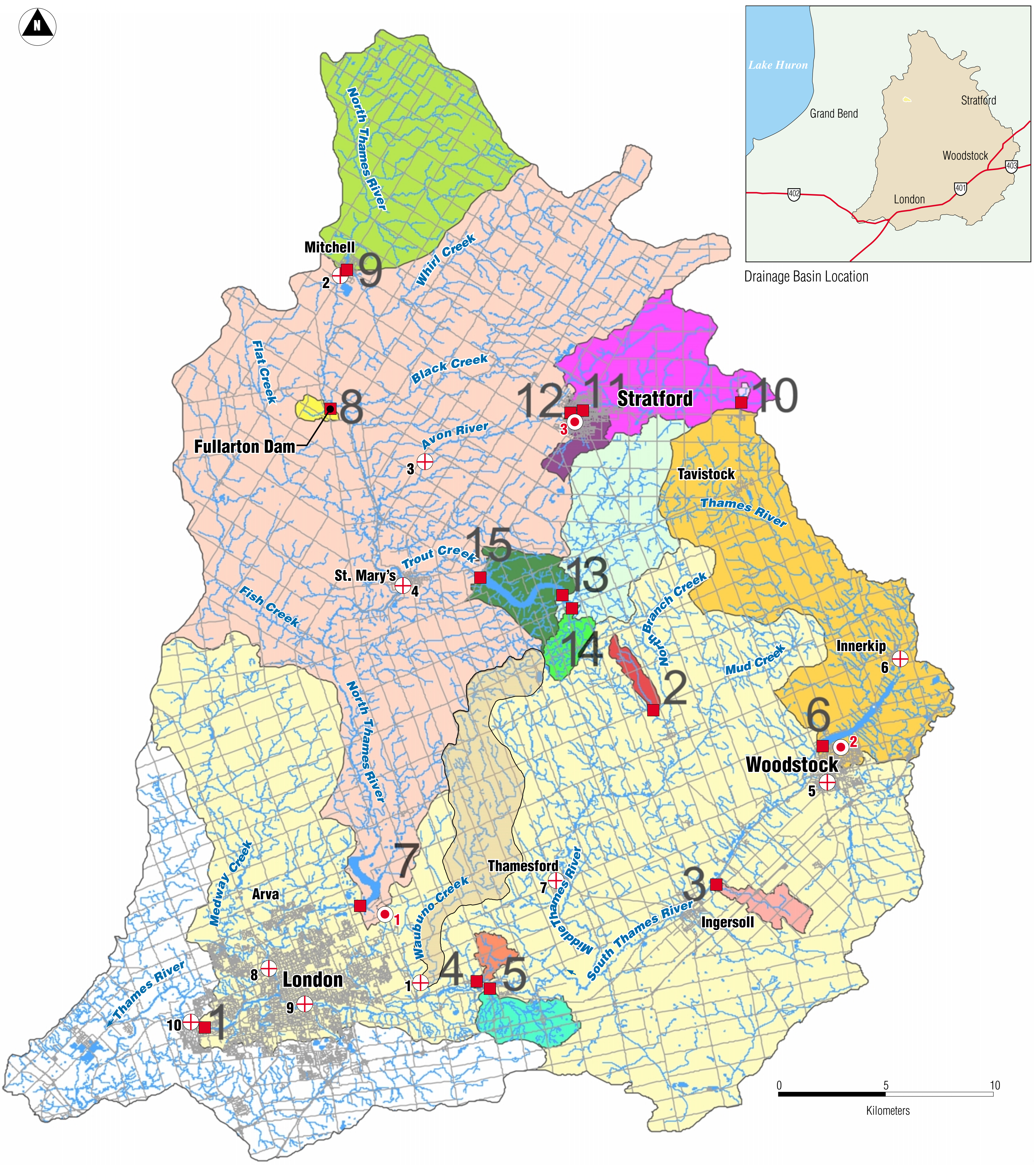
- **Precipitation:** Where more than one precipitation station data were used, the average precipitation over the basin was determined externally by applying Thiessen weighting coefficients and then input to the program. After the storm depth and duration have been established, a representative hyetograph 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

ACRES International

FIGURE 6.1 - BACK

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. The Thiessen polygon technique was applied to determine the applicable basin rainfall weighting factors, 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 Stratford was most representative of the storm events expected for the Fullarton basin. The data from the Stratford station were, therefore, 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 storm duration to be applied 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 of the basin. 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, 3-day and 5-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 profile of the flood hydrograph. The selected time interval of storm increments used in the study was 15 minutes.

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

Table 6.1

AES Rainfall Events for Stratford MOE, Station 6148105 (1966 to 2002)

Return Period (yrs)	Total Precipitation (mm)		
	6-Hr	12-Hr	24-Hr
2	40.6	46.7	53.1
5	62.9	70.5	77.5
10	77.7	86.3	93.6
25	96.4	106.1	114.0
50	110.2	120.9	129.1
100	124.0	135.5	144.1
250	142.2	154.9	164.1

Table 6.2

AES Rainfall Events for Stratford MOE, Station 6148105 (1959 to 2002) for Summer/Fall (May to November)

Return Period (yrs)	Cumulative Total Precipitation (mm)				
	1-Day	2-Day	3-Day	4-Day	5-Day
2	52.7	58.1	64.0	69.3	74.1
5	77.0	82.8	91.3	98.2	103.3
10	93.2	99.2	109.4	117.3	122.7
25	113.6	119.9	132.3	141.5	147.2
50	128.7	135.3	149.2	159.4	165.3
100	143.7	150.5	166.1	177.2	183.3
250	163.6	170.6	188.2	200.7	207.1

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. Because the hydraulic structures at damsites are to be evaluated under maximum flow, the storm distribution pattern must be selected to give the maximum hydrograph peak flows into the small reservoirs. Based on our past experience with dam safety analyses, the center-loaded (balanced storm), DDF-based hyetographs generate the highest peak flows. Appendix C provides additional background information pertaining to the use of balanced distributions.

Rainfall curves were derived from the DDF data for storms of various durations and return periods. The distributions of incremental rainfall were adjusted to fit a balanced storm pattern in each case. The total depth of rain in the storm was equal to the corresponding depth of rainfall for a given frequency and storm duration. The patterns were made dimensionless by dividing the total rainfall amounts by the cumulative incremental amounts and the total storm durations by the cumulative time amounts. The dimensionless data was required as the input format for the HEC-HMS model for Fullarton.

The balanced distributions for the 12-hr, 24-hr and 3-day storms are plotted in Figure 6.2 based on Stratford rainfall data. Figures 6.3 (a) and 6.3 (b) illustrate the 1:50-yr rainfall hyetographs over a 24-hr duration and a 72-hr duration, respectively. Appendix D summarizes the balanced distributions for the 6-hr, 12-hr, 24-hr, 2-day, 3-day and 5-day storms in Tables D1 to D6, respectively.

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, 1984). Since the size of the Fullarton basin drainage area is 4 km^2, it was not necessary to apply an areal reduction factor for watershed rainfall*.

Rain-On-Snowmelt Event

The DDF data of 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 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 A that covers the drainage area of the North Thames River basin below Mitchell was selected and used in the analysis of the Fullarton watershed.

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 A. Table D7 in Appendix D summarizes the 1-day, 3-day and 8-day rain-plus-snowmelt distributions. Because these would be longer duration storms (up to 8 days for the Fullarton Dam), they are expected to behave differently than the shorter duration storms given in Figure 6.2. The distribution of the rain-on-snowmelt storm extends over an 8-day period with the 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).

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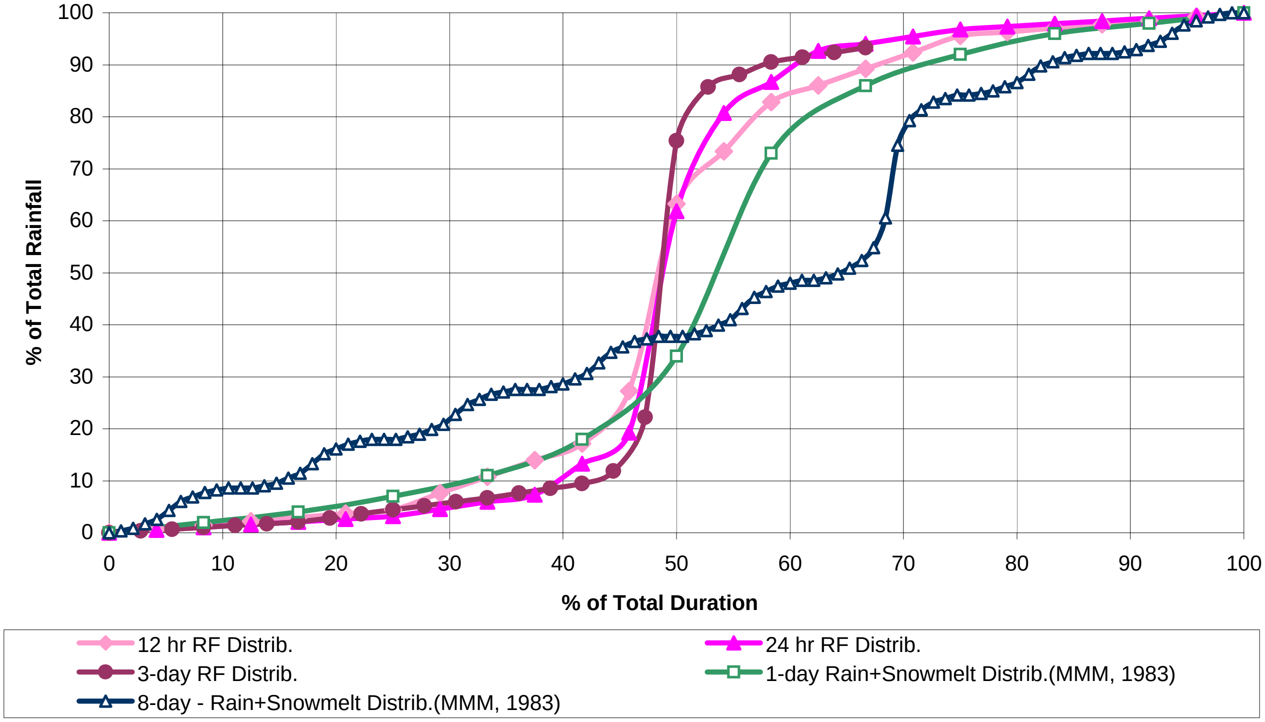
* Though no areal reduction was necessary, it should be noted that the >25-km^2 threshold is based on MNR guidelines and The World Meteorological Organization (WMO) curves (MNR, 1986; WMO, 1974) parameters not the US National Weather Service curves that are also presented in the MNR guidelines.

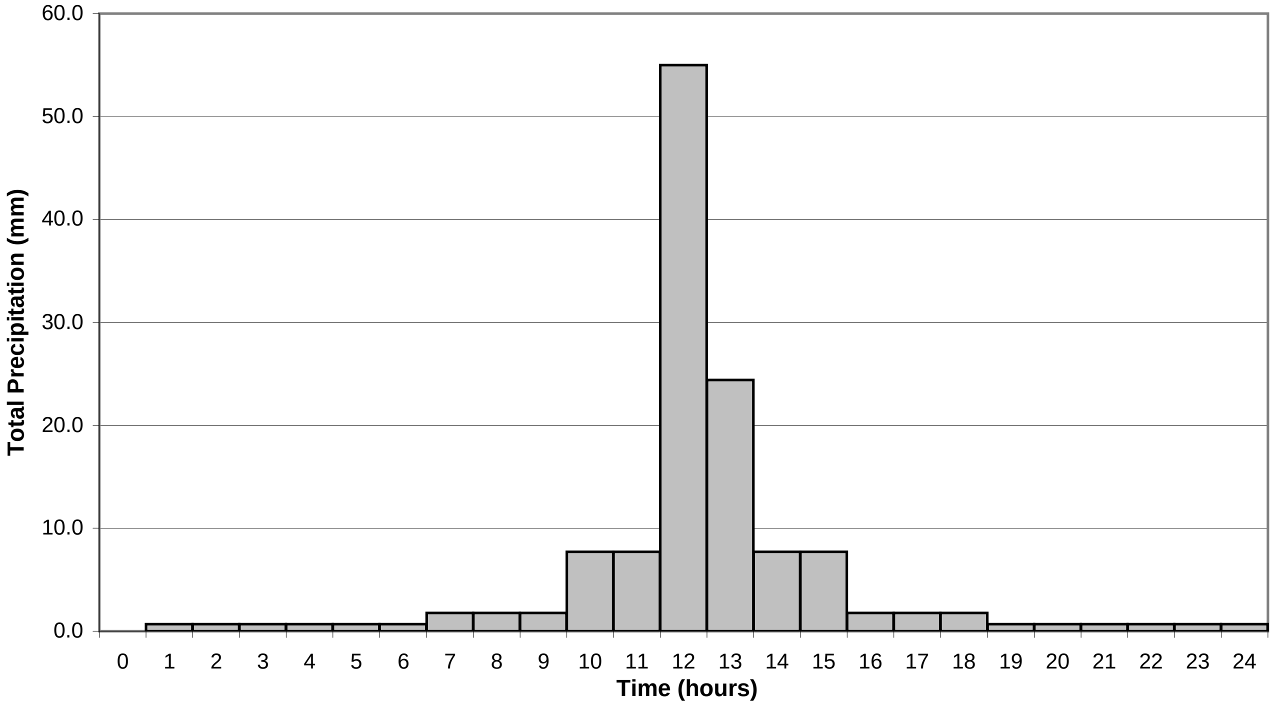
Figure 6.2: Dam Safety Assessment Report - Fullarton Dam
Comparison of Rainfall and Snowmelt Plus Rainfall Storm Distributions



Note: Rainfall and Rainfall-Plus-Snowmelt Depths based on the analysis results for Stratford MOE, Meterological Station (6148105); 1959 to 2002 (1 day, 2 days, etc) and 1966 to 2002 (6 hours, 12 hours, 24 hours).

BACK OF FIGURE 6.2

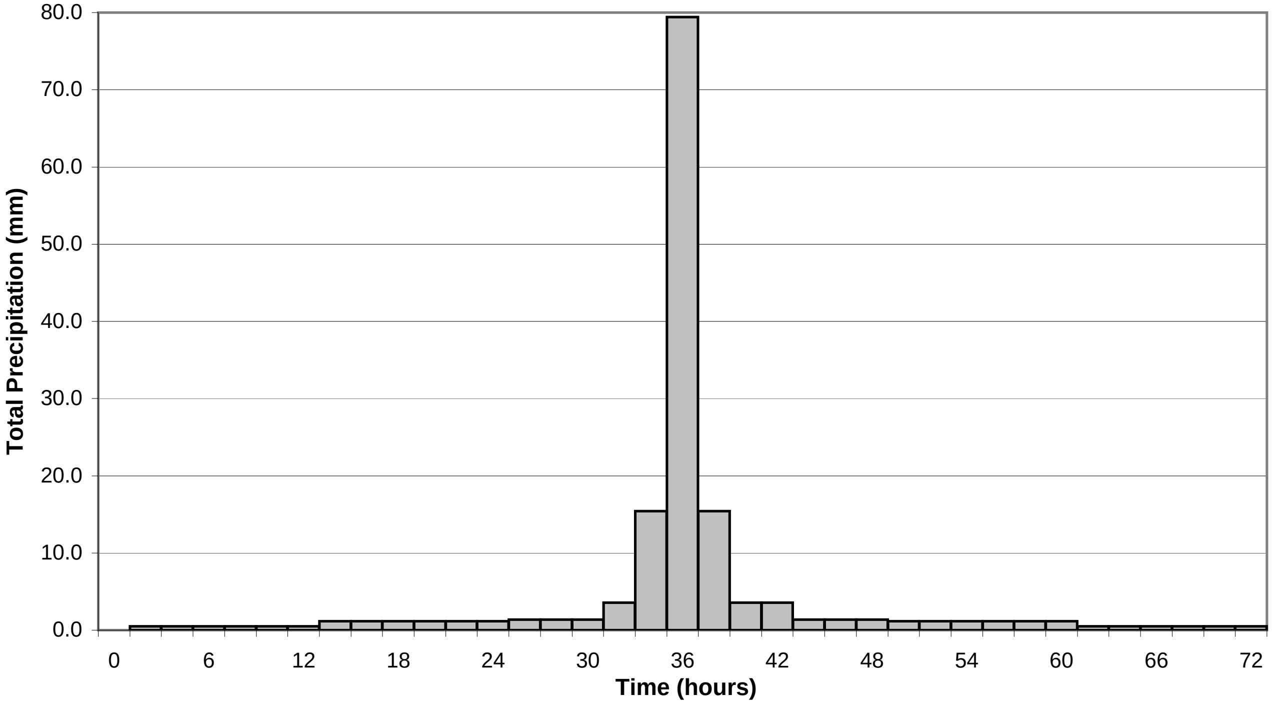
Figure 6.3 (a): Dam Safety Assessment Report - Fullarton Dam
50-Yr, 24-Hr Rainfall Distribution



Note: Rainfall hyetograph is based on Stratford RF distribution, as shown in Figure 6.2.

BACK OF FIGURE 6.3 (A)

Figure 6.3 (b): Dam Safety Assessment Report - Fullarton Dam
50-Yr, 3-Day Rainfall Distribution



Note: Rainfall hyetograph is based on Stratford RF distribution, as shown in Figure 6.2.

BACK OF FIGURE 6.3 (B)

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 Stratford is summarized in Table 6.3.

Table 6.3

AES Rainfall and Snowmelt Events for Stratford MOE, Station 6148105 (1959 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	31.2	44.1	53.9	62.3	69.6	77.2	83.2	88.6
5	41.2	55.7	68.8	79.0	88.9	99.5	108.9	117.0
10	47.8	63.4	78.7	90.1	101.7	114.3	125.9	135.9
25	56.1	73.2	91.1	104.1	117.8	133.0	147.5	159.7
50	62.3	80.4	100.3	114.5	129.8	146.8	163.4	177.3
100	68.4	87.6	109.5	124.8	141.7	160.6	179.3	194.8
250	76.6	97.1	121.4	138.2	157.2	178.8	200.0	218.0

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 Fullarton 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 directly to regulate a relatively small storage lake. 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 reservoir starting water level and discharge facilities setting, 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

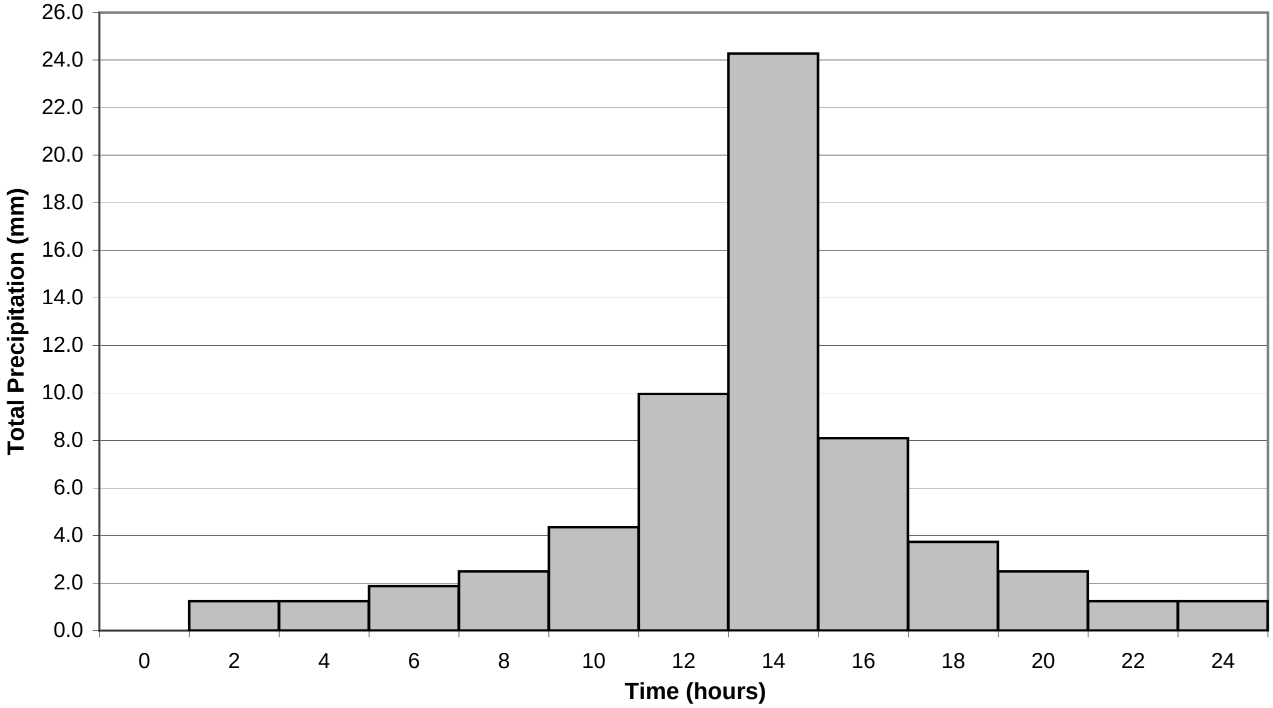
Hydrologic analysis of potential flood events at the damsite included the assessment of regional flood frequency characteristics along with deterministic rainfall-runoff modeling using the HEC-HMS simulation package, as described in Section 6.1.

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.

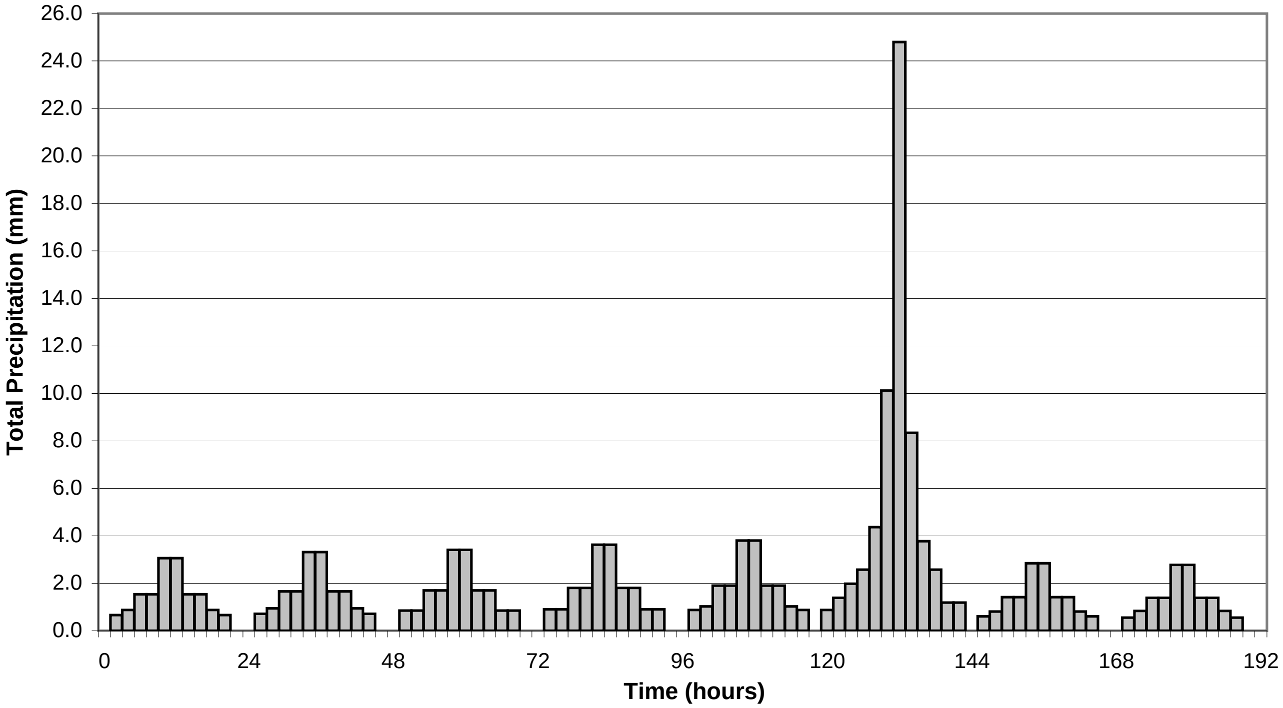
Figure 6.4 (a): Dam Safety Assessment Report - Fullarton Dam
50-Yr, 1-Day Rainfall Plus Snowmelt Distribution



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

FIGURE 6.4 (A) – BACK OF PAGE

Figure 6.4 (b): Dam Safety Assessment Report - Fullarton 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

(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 systems to optimize these parameters and match the model results with recorded data. Since no WSC streamflow stations or UTRCA meteorological monitoring stations were located directly on Fullarton Creek, a representative-gauged, unregulated river which had similar runoff characteristics to the Fullarton 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 south 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 to reduce the differences between the simulated and observed flows 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 used to assess their 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

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.

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.

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.

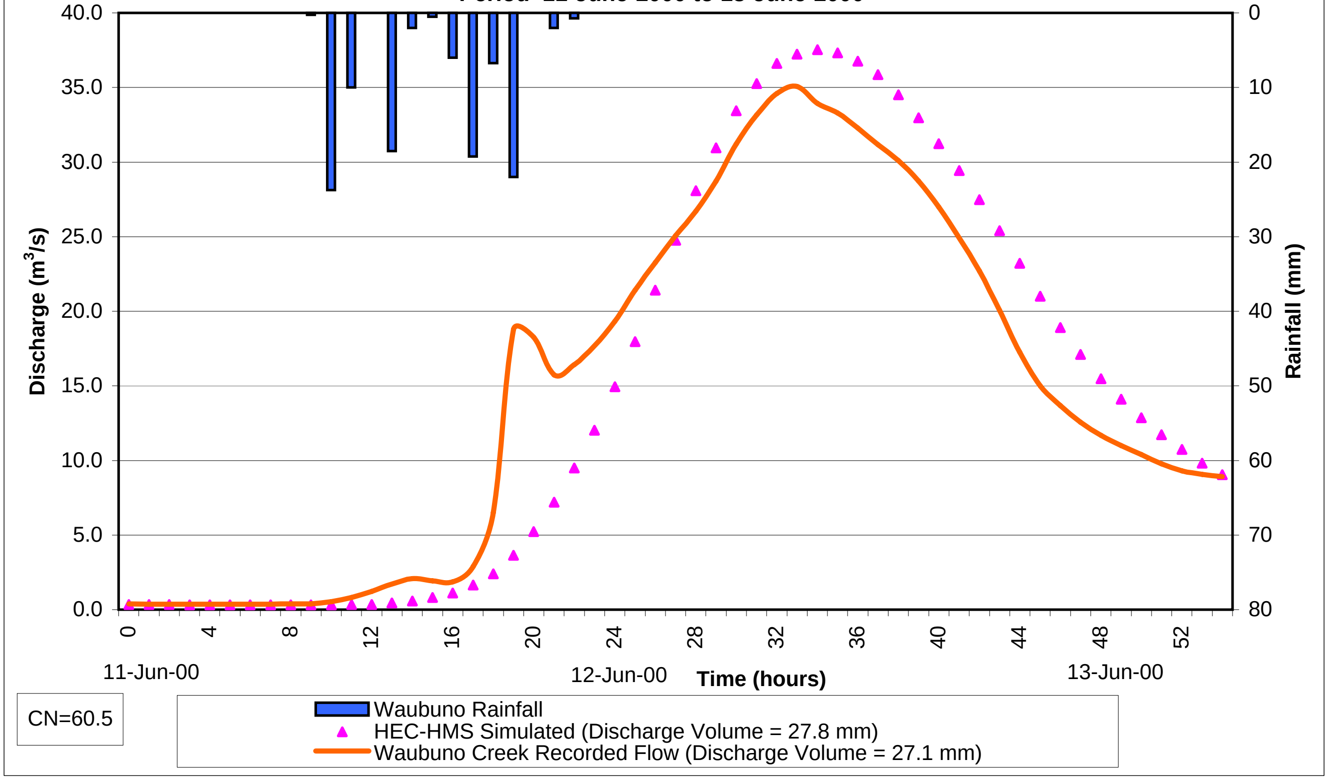
Results of the calibration are presented in Figures 6.5, 6.6 and 6.7. The HEC-HMS computed hourly outflow has been plotted beside the recorded flow for the storm events to provide a graphical comparison.

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 Fullarton basin.

(d) Storm Event Precipitation

Summer/fall storm rainfall amounts corresponding to the Stratford station for the shorter durations (6 hours, 12 hours and 24 hours) and the longer durations (2 days, 3 days and 5 days) were used in the HEC-HMS model. Summer/fall rainfall storm depths 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 Stratford 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.

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



BACK OF FIGURE 6.5

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

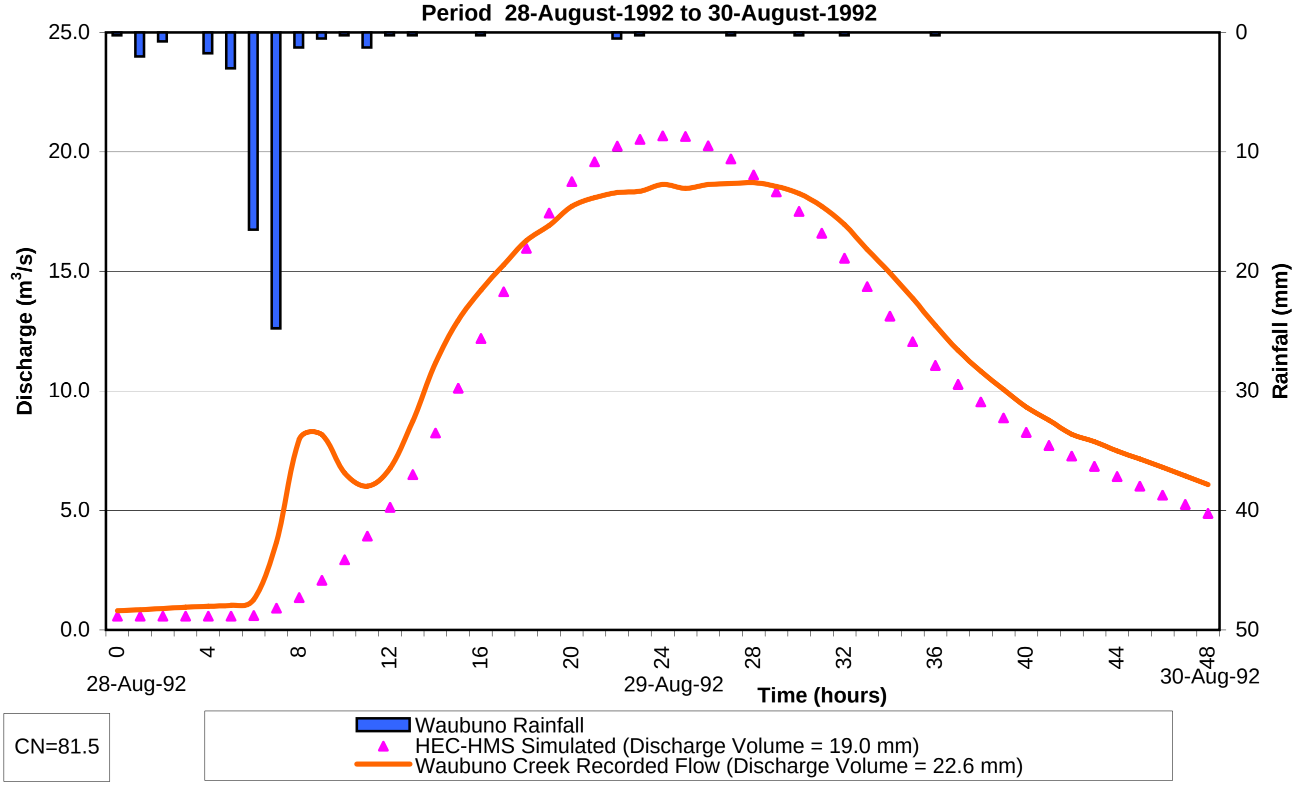


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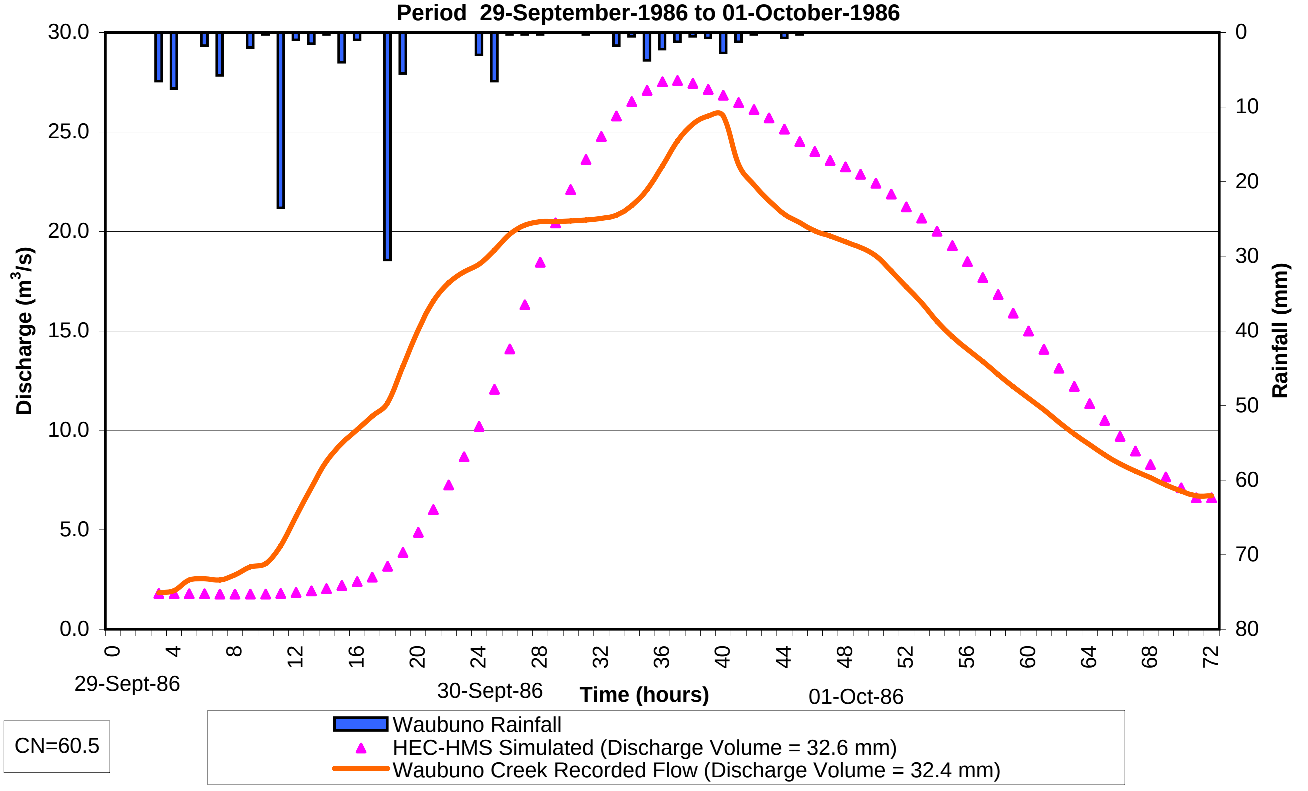
Figure 6.7: Dam Safety Assessment Report - Fullarton Dam
HEC-HMS Model Event Calibration Waubuno Creek
Period 29-September-1986 to 01-October-1986



BACK OF FIGURE 6.7

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 Fullarton 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 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 Fullarton 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 Fullarton Dam, which was part of the Dam Hazard Identification studies in July 2001. The elevations given on this drawing are to a local datum. Subsequently, a field survey was performed on the emergency spillway by C. Tasker of UTRCA on July 2, 2003 with the local benchmark (southeast corner of black anchor plate for gate at midpoint of dam) set at a reference of el 100.0 m.

(g) Model Setup and Initial Conditions – Study Basin

The HEC-HMS model was set up for Fullarton 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 Fullarton Dam

Watershed	Local Drainage Area (km^2)	Total Drainage Area (km^2)	Pond Area (km^2)	Basin Lag[*] (hrs)	Curve Numbers (CN)		Stream Length (km)	Average Slope (m/m)	Storm Event	Base Flow (m^3/s)
					II	III[**]				
Unnamed Tributary of the North Thames River	4.0	4.0	0.016	2.6	79	91	2.8	0.0039	Spring Fall	0.12 0.01

---

[*] 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 Perth 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 Fullarton are summarized in Table 6.6.

The elevation-volume relationship for the Fullarton 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 reflecting all stop logs in at the drop inlet structure. It was assumed that the stop logs are not manipulated at the Fullarton Dam; therefore, the same log settings were adopted for both spring and summer/fall storm events. 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	Spring		Fall	
	Level (m)	Stop Log Settings*	Level (m)	Stop Log Settings
Fullarton	99.40	all logs in	99.34	all logs in

Note: All elevations referred to a local datum.

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* Top of logs in drop inlet structure el 99.33 m.

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

Table 6.8 (a)

HEC-HMS Simulation Results for Unnamed Tributary of the North Thames River Subbasin

Event Duration	Event Timing	Storm Return Period (yrs)	Fullarton Dam			
			Total Precipitation (mm)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak Water Level (m)
1-Day (Rain-on-Snowmelt-AMC III)	Spring	2	31.2	2.4	2.3	99.62
		5	41.2	3.8	3.7	99.68
		10	47.8	4.7	4.7	99.72
		25	56.1	6.0	5.9	99.76
		50	62.3	6.9	6.8	99.79
		100	68.4	7.8	7.8	99.82
		250	76.6	9.1	9.0	99.85
3-Day (Rain-on-Snowmelt-AMC III)	Spring	2	53.9	4.1	4.1	99.70
		5	68.8	5.6	5.6	99.75
		10	78.7	6.6	6.6	99.78
		25	91.1	7.8	7.8	99.82
		50	100.3	8.7	8.7	99.84
		100	109.5	9.6	9.6	99.87
		250	121.4	9.6	9.6	99.87
8-Day (Rain-on-Snowmelt-AMC III)	Spring	2	88.6	4.3	4.3	99.70
		5	117.0	5.9	5.8	99.76
		10	135.9	6.9	6.9	99.79
		25	159.7	8.2	8.1	99.83
		50	177.3	9.2	9.1	99.86
		100	194.8	10.1	10.1	99.88
		250	218.0	11.3	11.3	99.91

Notes:
All elevations referred to a local datum.

Lowest crest elevation of earth-fill embankment is taken at approximately 100.00 m, based on field survey of steel marker at dam surface.

Table 6.8 (b)

HEC-HMS Simulation Results for Unnamed Tributary of the North Thames River Subbasin

Event Duration	Event Timing	Storm Return Period (yrs)	Fullarton Dam			
			Total Precipitation (mm)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak Water Level (m)
6-Hr Rainfall	Summer	2	40.6	2.0	1.9	99.59
(AMC II)		5	62.9	5.3	5.3	99.74
		10	77.7	8.0	7.9	99.82
		25	96.4	11.7	11.7	99.92
		50	110.2	14.7	14.6	99.99
		100	124.0	17.7	17.6	100.05
		250	142.2	21.8	21.8	100.11
12-Hr Rainfall	Summer	2	46.7	2.4	2.4	99.62
(AMC II)		5	70.5	6.0	5.9	99.76
		10	86.3	8.8	8.7	99.85
		25	106.1	12.5	12.5	99.94
		50	120.9	15.5	15.5	100.01
		100	135.5	18.5	18.5	100.06
		250	154.9	22.7	22.6	100.12
24-Hr Rainfall	Summer	2	53.1	3.0	3.0	99.65
(AMC II)		5	77.5	6.7	6.6	99.78
		10	93.6	9.4	9.3	99.86
		25	114.0	13.1	13.1	99.95
		50	129.1	16.0	15.9	100.02
		100	144.1	18.9	18.9	100.07
		250	164.1	22.9	22.9	100.12
(AMC III)		Hazel	211.1	32.5	32.5	100.21
2-Day Rainfall	Summer	2	58.1	3.6	3.5	99.67
(AMC II)	(May to	5	82.8	7.3	7.2	99.80
	November)	10	99.2	10.0	10.0	99.88
		25	119.9	13.7	13.7	99.97
		50	135.3	16.6	16.5	100.03
		100	150.5	19.5	19.4	100.08
		250	170.6	23.3	23.3	100.13
3-Day Rainfall	Summer	2	64.0	4.0	3.9	99.69
(AMC II)	(May to	5	91.3	8.0	7.9	99.82
	November)	10	109.4	10.9	10.8	99.90
		25	132.3	14.8	14.7	99.99
		50	149.2	17.7	17.7	100.05
		100	166.1	20.7	20.7	100.10
		250	188.2	24.7	24.7	100.14
5-Day Rainfall	Summer	2	74.1	4.1	4.0	99.69
(AMC II)	(May to	5	103.3	7.2	7.1	99.80
	November)	10	122.7	9.4	9.3	99.86
		25	147.2	12.2	12.2	99.94
		50	165.3	14.4	14.3	99.98
		100	183.3	16.5	16.5	100.03
		250	207.1	19.4	19.3	100.08

Notes:
All elevations referred to a local datum.
Lowest crest elevation of earth-fill embankment is taken at approximately 100.00 m, based on field survey of steel marker at dam surface.
IDF water level

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 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 mean annual 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 Fullarton 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)
		Fullarton Subbasin
2	1.00	2.3
5	1.32	3.0
10	1.57	3.6
20	1.80	4.1
50	2.13	4.9
100	2.37	5.4
200	2.60	5.9
500	2.92	6.7
Drainage Area (km^2)		4.0
Regional Q2y		2.3
Unit Runoff for the 1:2-yr flood ($m^3/s/km^2$)		0.5703

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[*] MNR Technical Guidelines.

The results are compared in Figure 6.8 and show that computed floods from the 24-hr and 3-day summer rainfall storms are higher than the regional peak flood estimates for outflow floods with return periods equal to and greater than the 2 years. In all cases, these are significantly higher than the regional estimates. The 3-day summer rainfall storm yields the most severe flood conditions at the dam in terms of water level rise and outflows. Due to the inherent variation in drainage basin morphology and degree of both natural and regulated storage, deviations about the regional estimates are expected. This is very pronounced in the case of Fullarton and may be due in part to the small basin size (4 km^2).

The index flood curve for Region 4 was developed using fairly large gauged catchments (i.e., average basin size is 721 km^2), but also contained catchments that were quite small. The estimated floods for Station No. 02GA032 near Guelph, with a drainage area of 2.5 km^2, in the data set are examined and compared to those generated by HEC-HMS for Fullarton. The prorated index flood curve is then plotted against the HEC-HMS floods for corresponding return periods. The resulting index curve based on Station No. 02GA032 gives higher values for comparable return periods than the overall Region 4 index flood curve. This plotted curve agrees better with the HEC-HMS curves. For example, the 1:50-yr flood from the index curve for Station No. 02GA032 shows a close agreement with the 1:50-yr HEC-HMS, 3-day rainfall flood (i.e., 13.9 m^3/s compared to 17.7 m^3/s, respectively). The difference between the model and index flood is about 21% and is considered reasonable for this type of study. The favorable comparison provides a good rationale for selecting the deterministic flood model results since these are of the same order of magnitude as those obtained by the Regional Index Flood method for small catchments.

6.2.2 Hydraulic Analysis

6.2.2.1 Discharge Capabilities

A hydraulic analysis of the Fullarton damsite was performed to evaluate its existing spillway capacity and check on tailwater levels. Information obtained during the site visits, existing data and reports were reviewed. The present spillway capacity at the site was reviewed using recent structure surveys completed in 2001 as well as additional field survey completed by C. Tasker of UTRCA on July 2, 2003. The impacts of any upstream or downstream hydraulic conveyance constraints were also evaluated.

The details of the pond impounded behind the Fullarton 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.

Figure 6.8: Dam Safety Assessment Report - Fullarton Dam
Flood Estimate Using Regional Index Flood Versus Deterministic Modeling

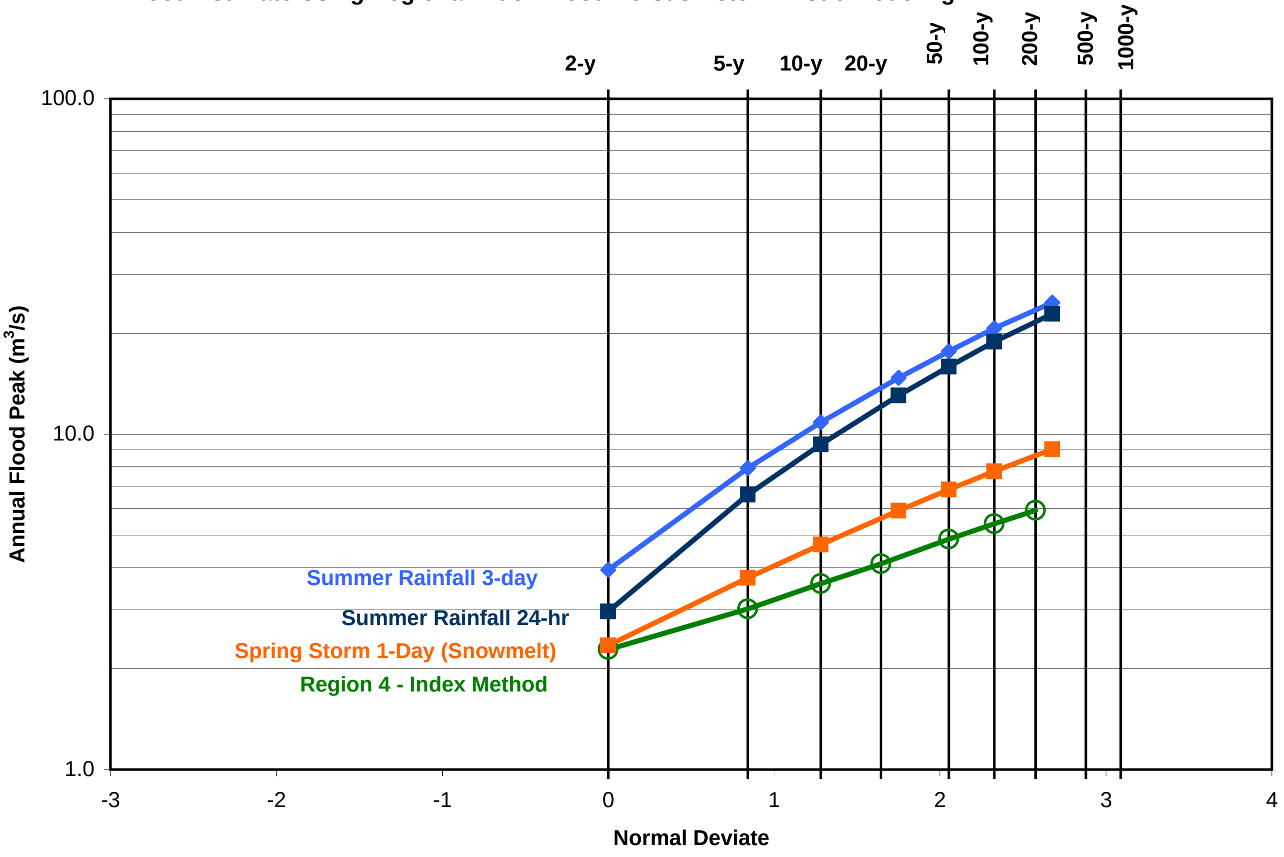

FIGURE 6.8 – BACK

The spillway capacities for the drop 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 Fullarton Dam is plotted in Figure 6.9.

6.3 Assessment and Confirmation of the IHP and IDF

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.

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 Fullarton Dam – Preliminary IHP and IDF

The Fullarton Dam is located approximately 2 km south of the town of Fullarton on an unnamed tributary of the North Thames River. Fullarton pond has a surface area of 0.02 km^2 and controls a total drainage area of 4 km^2. The primary discharge structure at the dam consists of a drop inlet to a concrete discharge pipe through the dam. Flow releases discharge into the creek below the dam and continue to flow for approximately 0.45 km before entering the main stem of the North Thames River. There is an emergency

Table 6.10

Fullarton Dam –
Spillway Capacity and Storage Relationship

Drop Inlet Structure (1)		Emergency Spillway (2)		Combined Discharge		Headpond Storage	
Elevation (m)	Discharge (m^3/s)	Elevation (m)	Discharge (m^3/s)	Elevation (m)	Discharge (m^3/s)	Elevation (m)	Storage (m^3x10^6)
99.33	0.00	99.33	0.00	99.33	0.00	97.30	0.000
99.38	0.08	99.38	0.00	99.38	0.08	99.30	0.017
99.43	0.24	99.43	0.00	99.43	0.24	101.30	0.068
99.48	0.44	99.48	0.06	99.48	0.50		
99.53	0.68	99.53	0.37	99.53	1.04		
99.58	0.94	99.58	0.81	99.58	1.76		
99.63	1.24	99.63	1.36	99.63	2.60		
99.68	1.57	99.68	2.08	99.68	3.64		
99.73	1.91	99.73	3.09	99.73	5.00		
99.78	2.28	99.78	4.33	99.78	6.61		
99.83	2.38	99.83	5.81	99.83	8.19		
99.88	2.41	99.88	7.54	99.88	9.95		
99.93	2.44	99.93	9.54	99.93	11.97		
99.98	2.47	99.98	11.81	99.98	14.28		
100.03	2.49	100.03	13.99	100.03	16.59 [*]		
100.08	2.52	100.08	16.13	100.08	19.45 [*]		
100.13	2.55	100.13	18.40	100.13	23.41 [*]		

Notes:

(1) Assumes all logs in drop inlet structure

(2) Average crest elevation of emergency spillway is 99.46 m.

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

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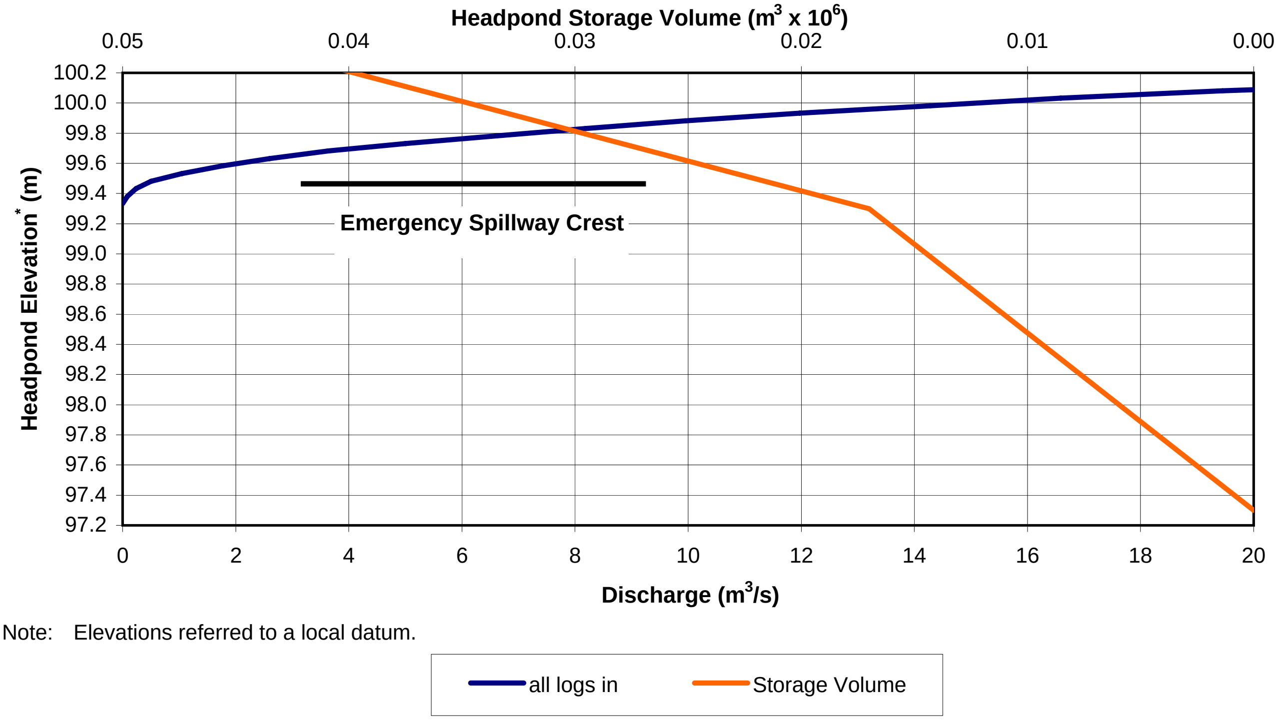
[*] Includes flow over top of embankment dam; lowest crest elevation is taken at approximately 100.00 m, based on field survey of steel marker at dam surface.

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



Note: Elevations referred to a local datum.

FIGURE 6.9 – BACK OF PAGE

spillway located on the right or east bank of the reservoir. This consists of a lower section at the end of the embankment dam which is armored with a form of interlocking concrete blocks across its width and part of the crest of the spillway. The embankment dam is approximately 3.4 m high and impounds a total estimated storage volume of 0.02 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.

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. Deterministic rainfall/runoff modeling results have established that the 50-yr, 3-day summer storm event is the governing flood for this site. This flood event has been used to assess the adequacy of the existing discharge facilities at the damsite to meet dam safety requirements. 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.

Table 6.11

Preliminary IHP and IDF Classifications for Fullarton 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^3x10^6)	Spillway Facilities	By Height	By Storage	Loss of Life	Economic, Social & Environmental	IHP	IDF
Unnamed Tributary of the North Thames River	4.0	4.0	0.016	3.4	0.02	- drop inlet structure - emergency spillway	SMALL	SMALL	None expected	Minor flood damages downstream	VERY LOW	25-yr flood to 50-yr flood

6.3.3 Fullarton 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, 3-day summer storm IDF event, approximately 84.2% of the discharge would be conveyed through the emergency overflow spillway with the remainder going through the drop inlet and over the embankment section. The inflow flood for this frequency was estimated to be 17.7 m^3/s while the peak outflow was also 17.7 m^3/s due to negligible attenuation by the pond. The dam discharge facilities would be unable to pass this flood without slightly overtopping the main embankment dam by only 0.05 m at an upstream water level of 100.05 m. Therefore, the dam does not have adequate spillway capacity to pass the IDF, on the basis of all logs left in the drop inlet structure. Presently, the Fullarton Dam is confirmed as a VERY LOW hazard structure, and the corresponding IDF is the 50-yr, 3-day summer storm event. The final IHP and IDF classifications are presented in Table 6.12.

Table 6.12

Final IHP and IDF Assessments for Fullarton 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**							
Unnamed Tributary of the North Thames River	VERY LOW	1:50-yr	Summer (rainfall) 3-day	99.34	17.7	17.7	100.05	0.71	97.8

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). To obtain conservative estimates of freeboard requirements, the effective fetch in the reservoir was calculated with the primary wind direction aligned with the longest fetch length or radial in the vicinity of the dam structure. Since the reservoir is relatively small, no corrections were made from overland to overwater wind speeds.

A Gumbel extreme value extrapolation of the wind frequency data (NRC-CNRC, 1995) for the station at Mitchell 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 107 km/h (100 years) or 130 km/h (1000 years) would both be long enough to establish steady-state wind/wave conditions in the headpond.

The computed effective fetch length for the Fullarton pond is 0.09 km. The effective fetch at the dam center was 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 both cases. The significant wave height was calculated as a function of effective fetch and wind speed. The design wave was taken as the average of the highest 10% of waves (H_{10}), and was determined from the significant wave height from the SPM ($H_{10} \approx 1.27$ Hs). The resulting wave heights and wave run-ups are summarized in Table 6.13 for the 100-yr and 1000-yr wind speeds.

Table 6.13

Freeboard Requirements for Fullarton Dam

											Available Freeboard		
				1:1000 Wind				1:100 Wind			Normal Freeboard	Minimum Freeboard	
Type	Abutment Conditions	Crest Elevation (m)	Normal Water Level (m)	Design Wave Height (m)	Wind Setup (m)	Wave Run-Up (m)	IDF Water Level (4) (m)	Design Wave Height (m)	Wind Setup (m)	Wave Run-Up (m)	Crest (1) Normal (m)	Crest (2) IDF (m)	Remarks
Earth fill	Earth fill	100 (5)	99.33	0.35	0.02	0.27 (3)	100.05	0.31	0.02	0.24 (3)	0.38	-0.31	Embankment dam is overtopped during IDF

Notes:
(1) Crest elevation - (NWL + 1:1000-yr wind setup + 1:1000-yr wave run-up).
(2) Crest elevation - (IDF + 1:100-yr wind setup + 1:100-yr wave run-up).
(3) Conservatively estimated as the design wave height; waves expected to break before reaching the structure.
(4) Water level based on all logs in drop inlet structure.
(5) Lowest crest elevation of earth-fill embankment is taken at approximately 100.00 m, based on field survey of steel marker at dam surface.

All elevations referred to a local datum.

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 are given in Table 6.13.
- Under peak IDF water level conditions, minimum freeboard requirements at the damsite have been conservatively established for specified 100-yr wind conditions. Minimum freeboard requirements are given in Table 6.13.

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

7 Civil/Structural Assessment

7 Civil/Structural Assessment

The Fullarton Dam is essentially an earth embankment with a drop inlet structure located in the pond connected to a concrete pipe passing through the dam and an emergency spillway structure whose entrance is protected by cable-connected concrete blocks. These structures do not lend themselves to structural 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 Fullarton Dam is located on a small, unnamed eastward-flowing stream, which is a tributary of the southward-flowing North Thames River. The latter is the main drainage course in the area.

The upland terrain is rolling and locally hummocky. Relief is about 35 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, 1976), the area is characterized by thick deposits of sediments. These were deposited during the Wisconsin glaciation which occurred in the Pleistocene era.

The damsite is on the south side of and is close to the North Thames River, which is situated along a major geological contact in terms of glacial deposits. On the west side of the North Thames River, the predominant material is a silty and clayey silt, calcareous till known as the Rannoch Till. On the east side of the river, from north to south, is a silt till known as the Elma Till, a thin strip of a clay till known as the Wartburg Till (Milverton Moraine) and a sandy silt till known as the Stratford Till. Irregular, local deposits of glaciolacustrine silt and clay overlie these various till deposits. Recent alluvium occurs in river and stream courses. Outwash gravel deposits are common. Numerous eskers and moraines exist. The Mitchell Moraine forms the west bank of the North Thames River. Other moraines exist further west.

Horizontally bedded sedimentary bedrock underlies the sediments, but is not exposed.

8.1.2 Site Geology

The site is in a rolling area of cultivated land. Overburden consisting of clay, silt, sand and gravel forms the ground surface, and probably underlies the dam and reservoir. Depth to the top of the 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.

8.2.2 Foundation Preparation and Characteristics

There are no records of dam construction and of the foundation preparation. Based on the log of the boreholes, 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 low plasticity clay. Based on information presented in Section 5 and on an empirical correlation, for clay-rich materials, between effective angle of friction and plasticity index (Holtz and Kovacs, 1981), an angle of friction of 32° 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 mostly sandy material with fines and gravel. Based on information presented in Section 5 and on an empirical correlation for granular materials between effective angle of friction and the ‘N’ values which relate to the in situ density, an angle of friction of 32° was selected for the upper foundation material and 38° for the lower foundation material.

8.4 Bearing Capacity

The allowable bearing capacity of the foundation is estimated to be in the order of 200 kPa (Canadian Geotechnical Society, 1992). The embankment exerts a maximum total pressure of approximately 75 kPa and, hence, the foundation has more than adequate bearing capacity.

8.5 Settlement

Fullarton 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 are not considered to be susceptible to liquefaction due to their substantial clay and silt content (Arumoli et al., 1999). The soils that comprise the foundation contain substantial fines and are well-graded overall. They are also considered not susceptible to liquefaction. The low seismicity potential in the site area also reduces the risk of liquefaction to a negligible level.

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, although locally the slope was soft (mushy) on the right side downstream. This suggests the water table was higher than average at this location.

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 3.4 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

Table 8.1 describes the properties for the various materials used in the stability analyses.

8.9.4 Phreatic Surface

The phreatic surface was established using the piezometer readings and a point just above the toe. Figure 8.1 shows the phreatic surface.

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 Fullarton 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 0.020 for the 1:100-yr return period.

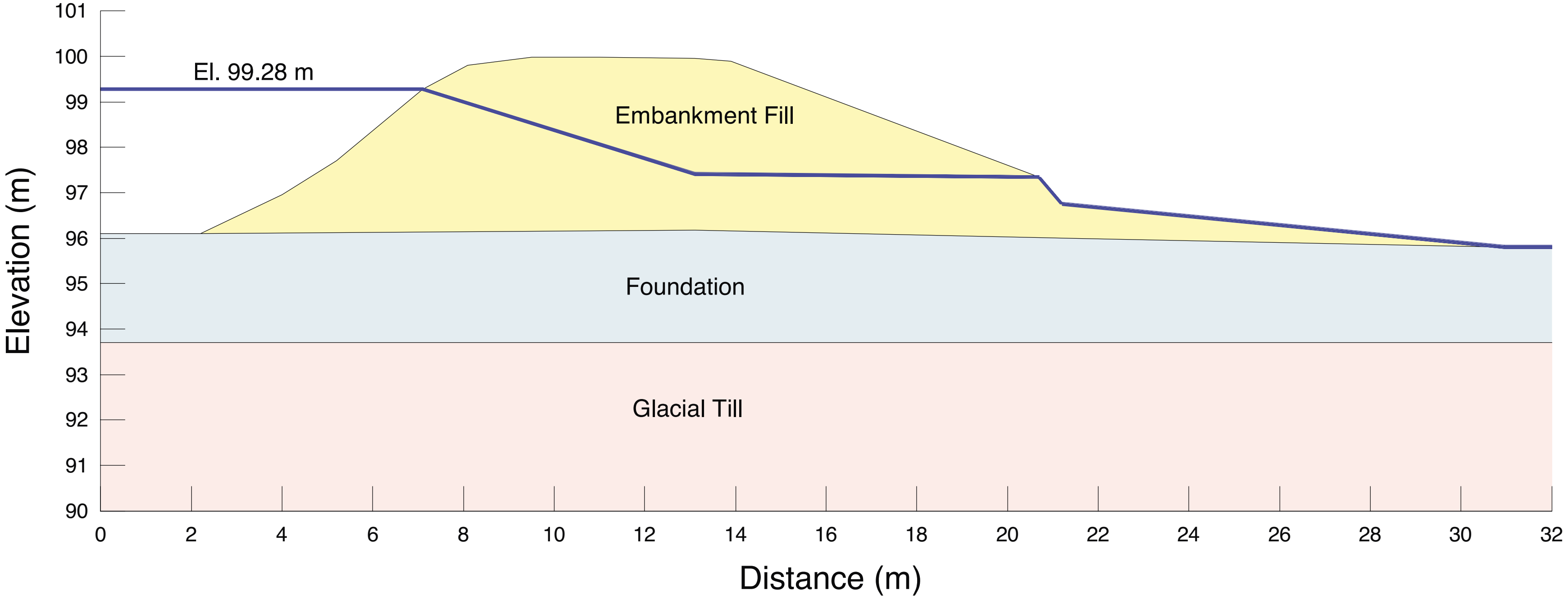


Figure 8.1
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Fullarton Dam
Stability Section

FIGURE 8.1 – BACK OF PAGE

Table 8.1

Stability Analysis of Earth Embankments

Item	Criteria	Calculated	Comments
General			
IHP		Very Low	
Flood Conditions			
IDF		50-yr flood	
Materials			
Embankment			
- embankment fill (CL)			
cohesion (kPa)		0	
ϕ (deg)		32	
moist unit weight (kN/m^3)		17.8	
saturated unit weight (kN/m^3)		19.0	
Foundation			
- SP - SM			
cohesion (kPa)		0	
ϕ (deg)		32	
moist unit weight (kN/m^3)		18.5	
saturated unit weight (kN/m^3)		21.0	
- 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)		99.28	
IDF water level		100.05	
Seismic, horizontal (S_h) PGA (*g*)		0.020*	* 2/3, i.e., 0.013*g*, was used in pseudostatic analyses
Load Combinations			
Upstream Slope			
Normal (NWL)	1.50	1.32	Does not meet the criteria
Extreme (NWL, S_h)	1.10	1.26	
Extreme (IDF)	1.30	1.35	
Rapid Drawdown	1.20	N/A	
Downstream Slope			
Normal (NWL)	1.50	1.41	Does not meet the criteria
Extreme (NWL, S_h)	1.10	1.36	
Extreme (IDF)	1.30	1.41	
Rapid Drawdown	N/A	N/A	

Table 8.2

Probabilistic Earthquake Parameters

Probability of Exceedance per Year	**0.010**	**0.005**	**0.0021**	**0.001**
Peak horizontal ground acceleration (*g*)	0.020	0.028	0.038	0.050

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 0.013*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. 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 8.2 to 8.7 graphically depict the cross sections analyzed and the minimum factors of safety established for both the upstream and downstream sections.

The upstream slope and the downstream slope fail to meet acceptance criteria for the normal water level condition.

A parametric study indicates that a small increase in the angle of friction for the embankment fill would bring the dam stability into compliance. It may be that the actual angle of friction of the fill is greater than that assumed, and perhaps sufficiently greater to meet compliance requirements. It is recommended that further shear strength testing be done followed by reanalysis. It is noted that sufficient sample material is available from the recently completed boreholes.

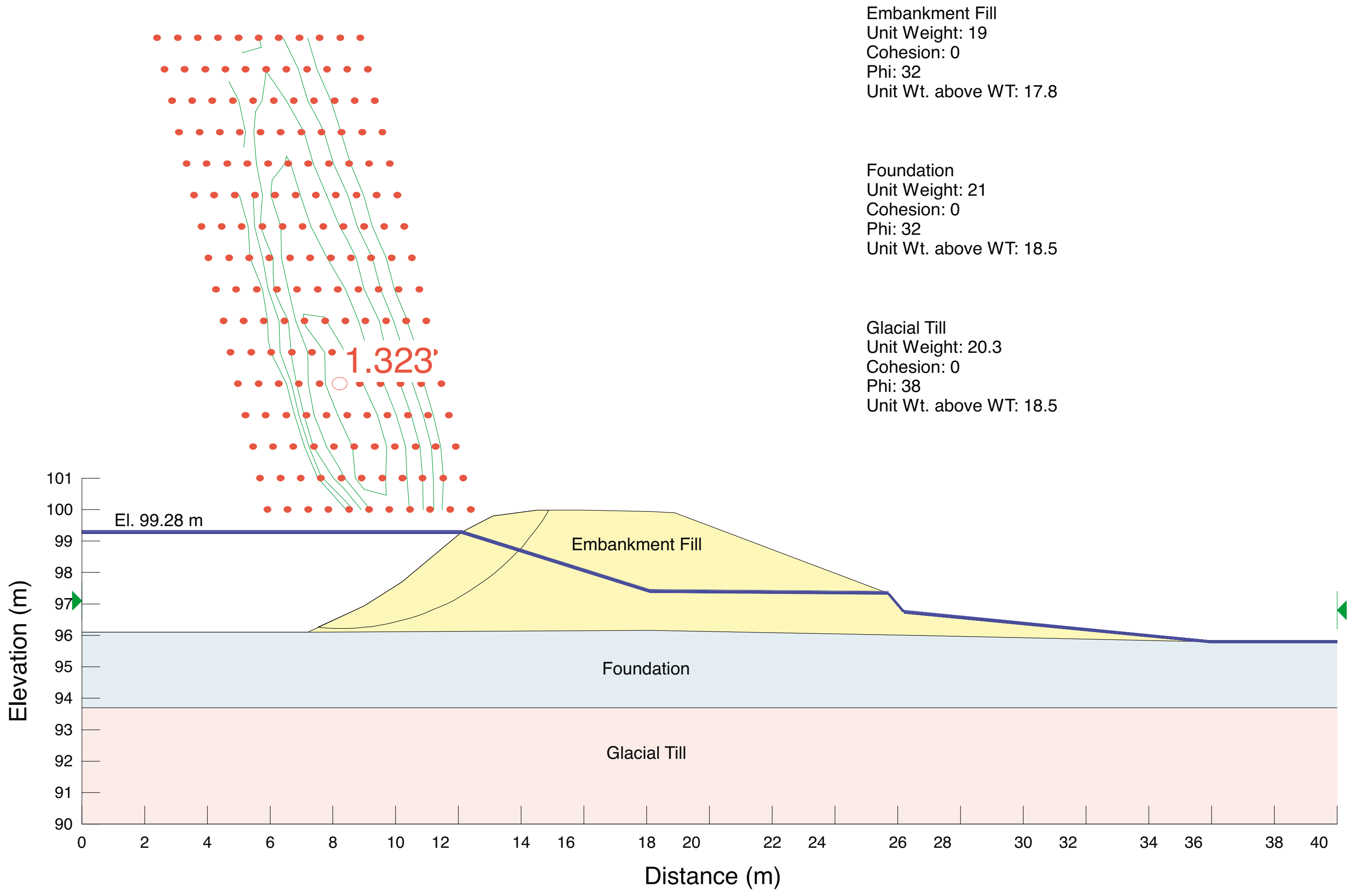


Figure 8.2
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Fullarton Dam

Upstream Slope Stability, Normal Load Condition

ACRES International

FIGURE 8.2 – BACK OF PAGE

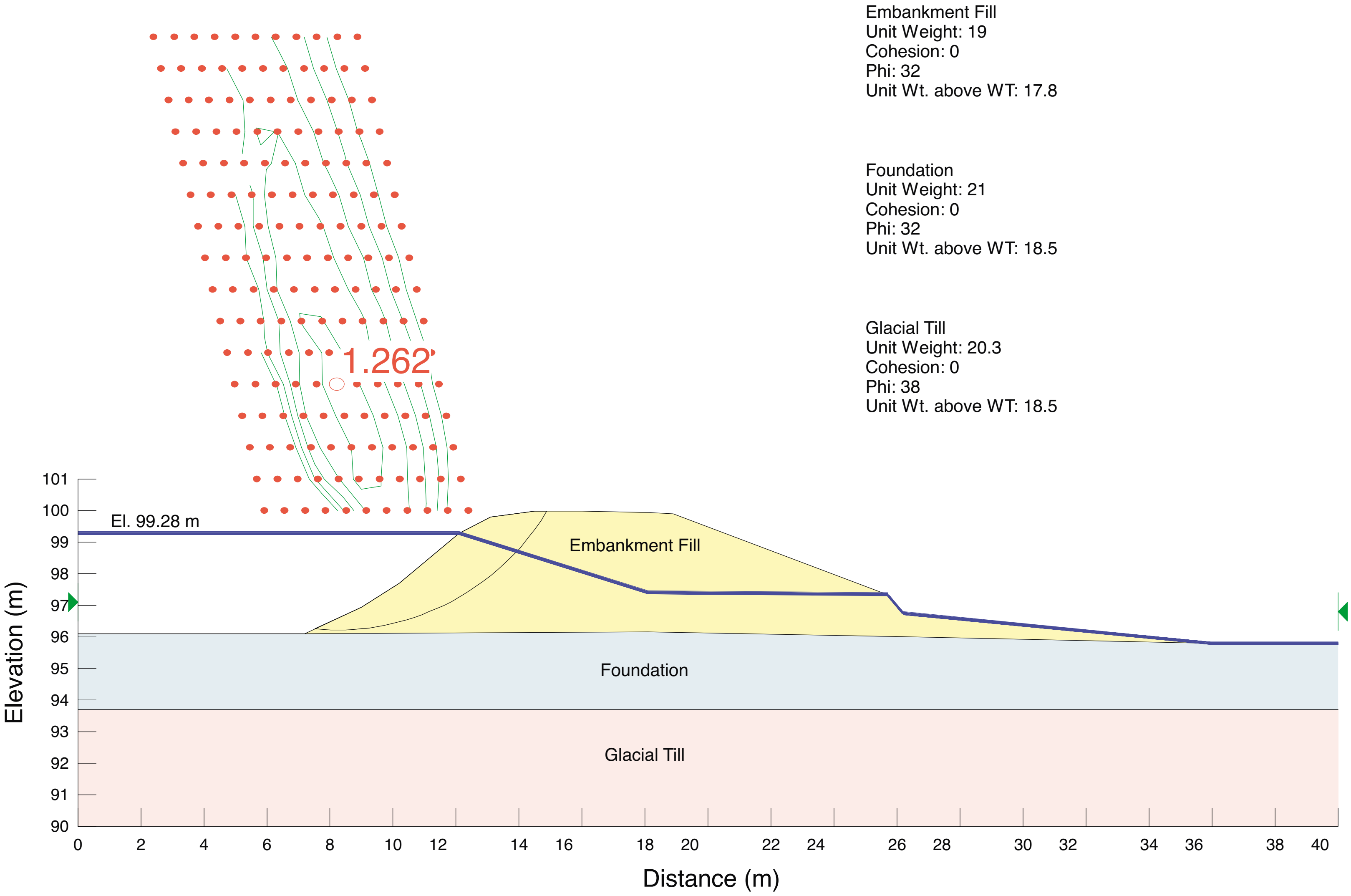
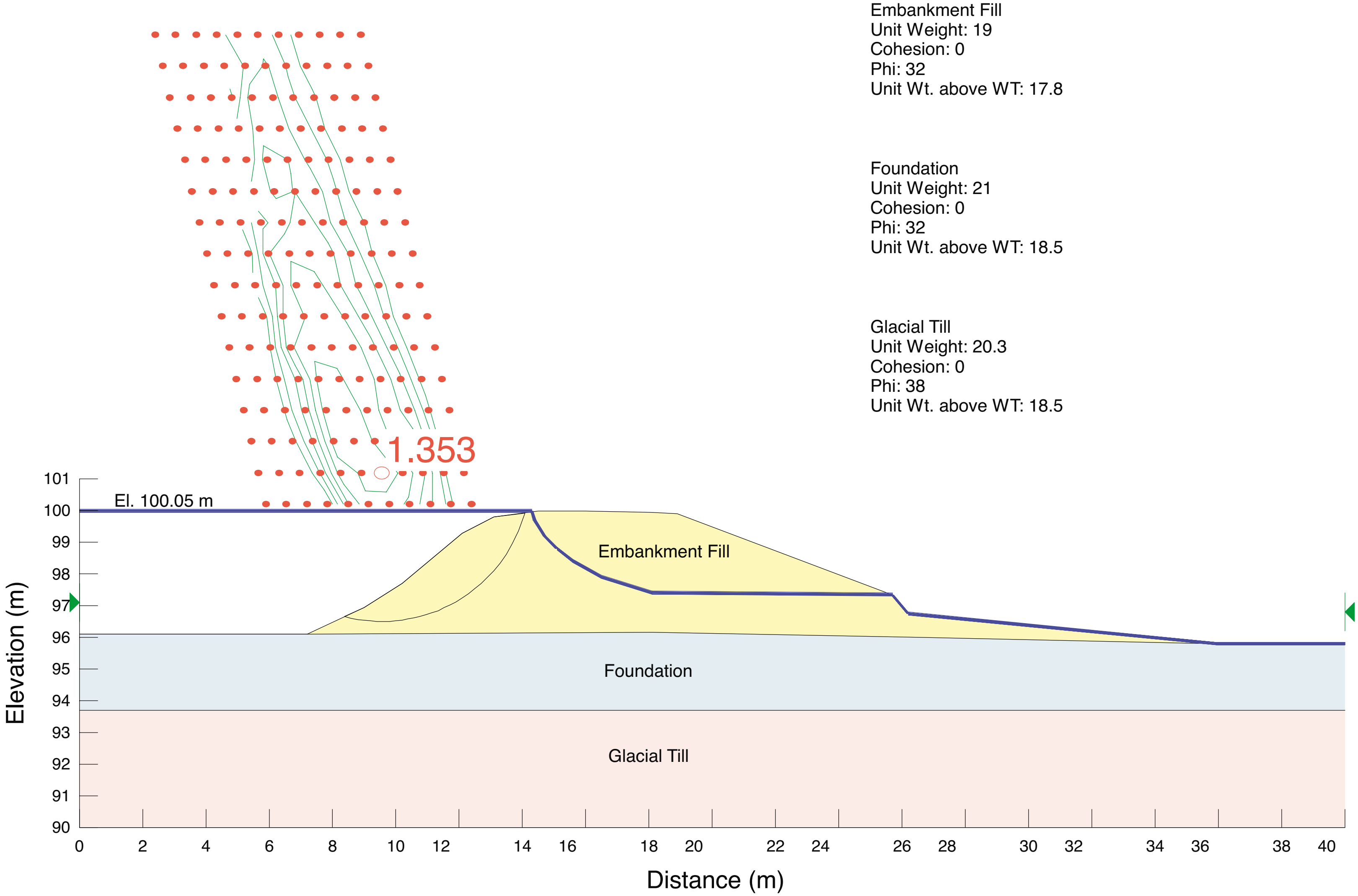


Figure 8.3
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Fullarton Dam

Upstream Slope Stability, Extreme Load Condition (NWL, S_h)

FIGURE 8.3 – BACK



Embankment Fill
Unit Weight: 19
Cohesion: 0
Phi: 32
Unit Wt. above WT: 17.8

Foundation
Unit Weight: 21
Cohesion: 0
Phi: 32
Unit Wt. above WT: 18.5

Glacial Till
Unit Weight: 20.3
Cohesion: 0
Phi: 38
Unit Wt. above WT: 18.5

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

Upstream Slope Stability, Extreme Load Condition (IDF)

ACRES
International

FIGURE 8.4 – BACK

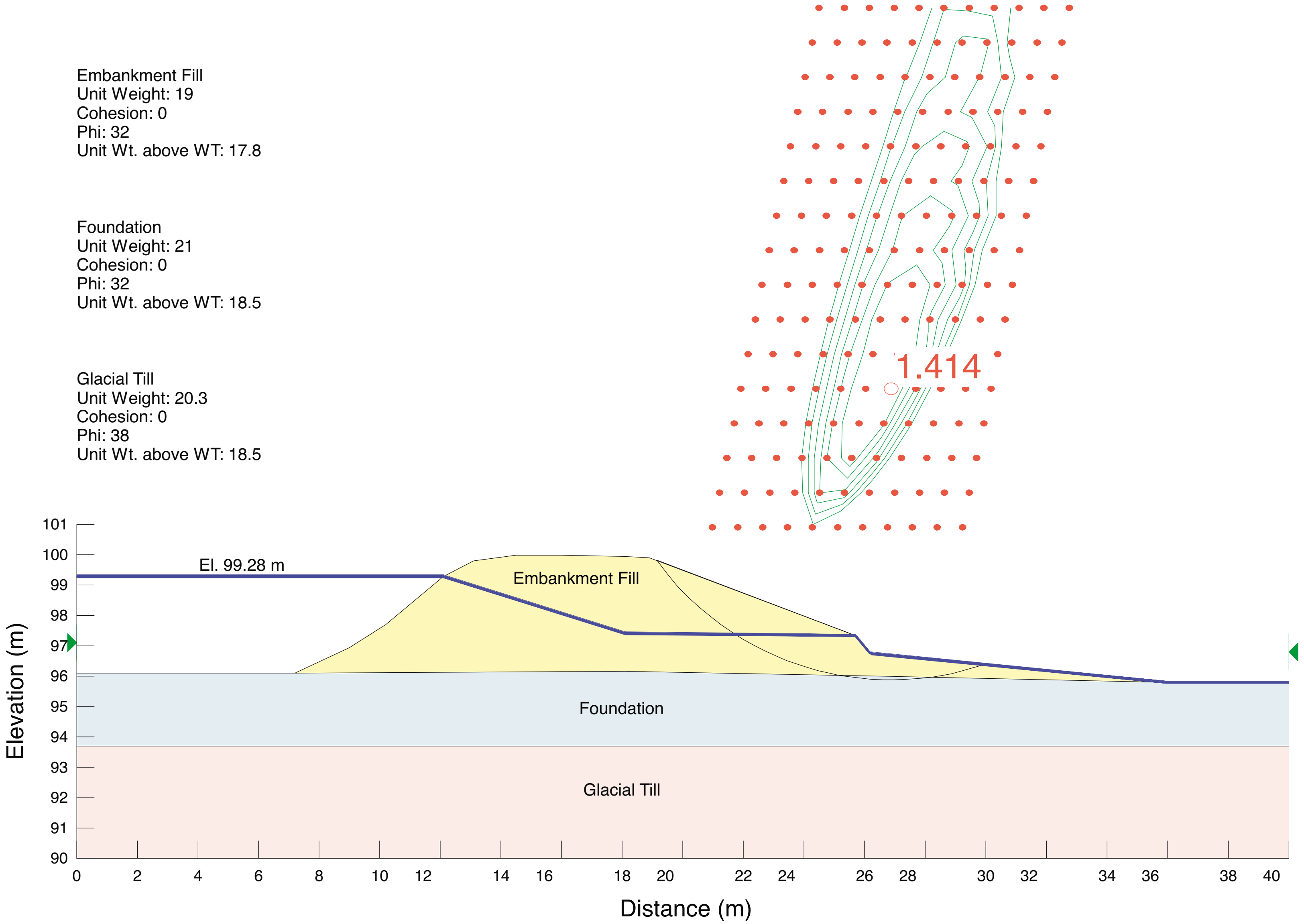


Figure 8.5
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Fullarton Dam

Downstream Slope Stability, Normal Load Condition

FIGURE 8.5 –BACK

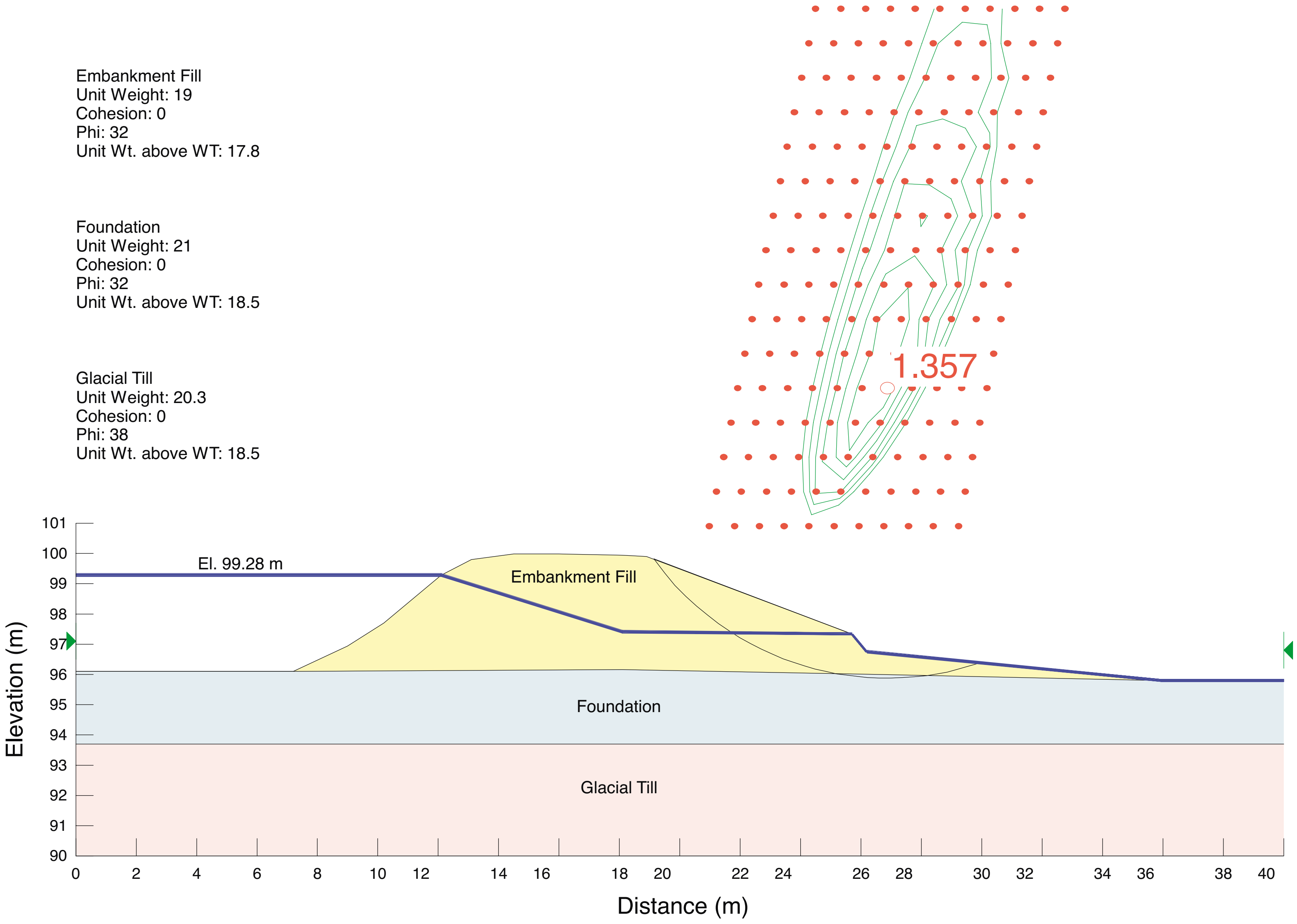


Figure 8.6
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Fullarton Dam

Downstream Slope Stability, Extreme Load Condition (NWL, S_h)

FIGURE 8.6 – BACK

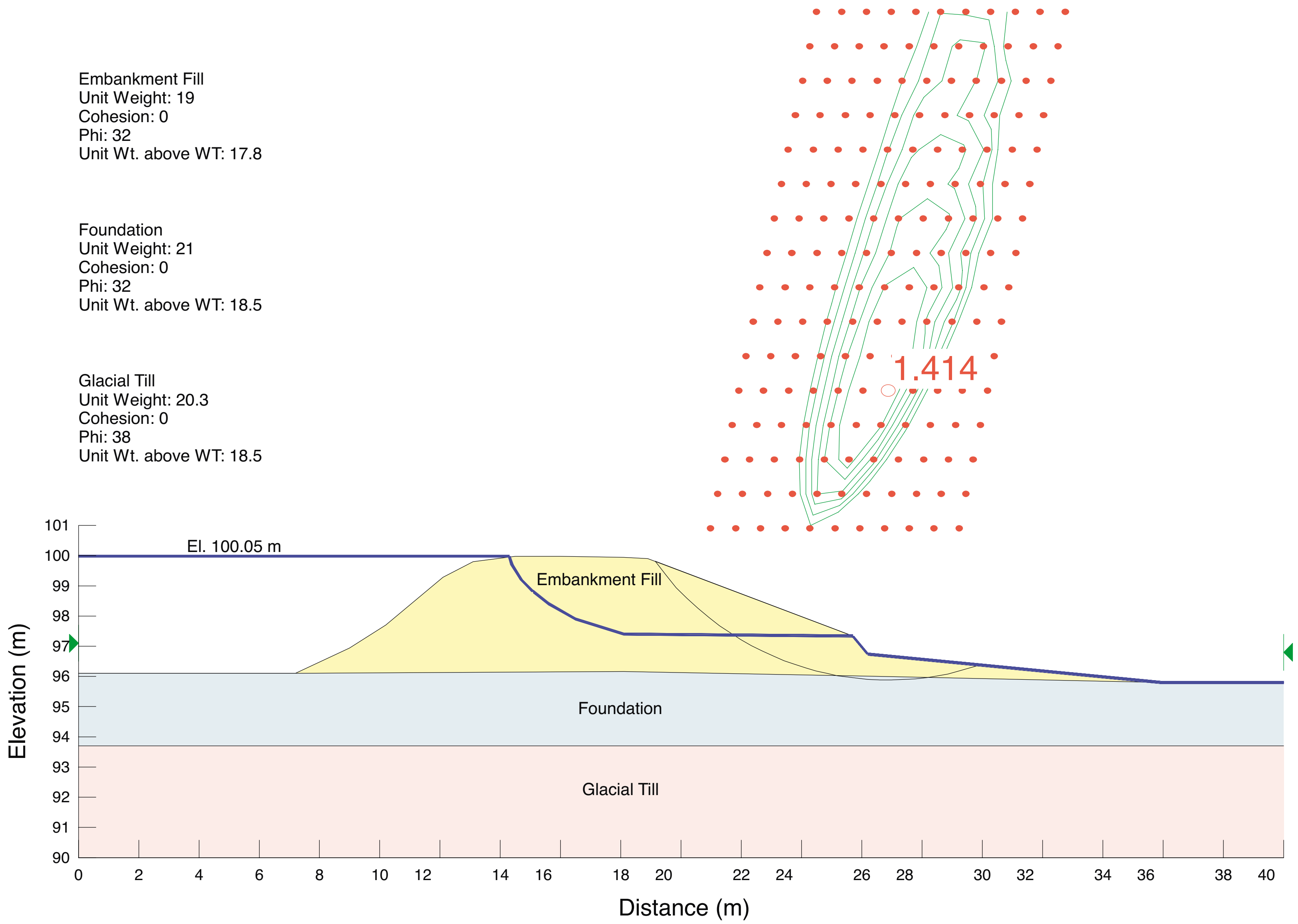


Figure 8.7
Upper Thames River Conservation Authority
Dam Safety Assessment Report - Fullarton Dam

Downstream Slope Stability, Extreme Load Condition (IDF)

FIGURE 8.7 BACK

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 or displacement in the dam or in the abutments. Some shallow sinkholes or pits appear to exist on the right-side downstream slope; these should be filled and monitored. Some further rehabilitation is required; for example, there is no riprap protection on the upstream slope and some wave-induced erosion has occurred locally.

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

9 Operations, Maintenance and Safety

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

9.1 Operation

The drop inlet structure and concrete outlet pipe combined with the emergency spillway structure at the Fullarton Dam are inadequate to ensure the safe passage of the IDF. Inadequate freeboard occurs during passage of the IDF.

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 E.

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

9.3 Fall Protection

Because the Fullarton 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.

10 Emergency Preparedness Plan

10 Emergency Preparedness Plan

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

Fullarton 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 raising the crest of the dam. 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

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 is shown in Table 11.2. Details of the recommended action and associated costs for the Fullarton Dam are summarized in Table 11.3. An overall cost summary of the remedial repairs, excluding allowances for engineering, permitting and environmental costs, is provided in Table 11.4.

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

Estimated Remedial Repair Costs – Fullarton 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	Perform shear strength tests on existing samples. Rerun slope stability analyses.	-	-	5,000	1	Survey by UTRCA to verify inclination of upstream slope.
2	Embankment	Upstream slope	Some erosion occurring	Install riprap	-	150 m^2	3,750	2	1.5 m x 100 m = 150 m^2
3	Embankment	Downstream slope	Sinkholes occurring	Regrade downstream slope and fill in sinkholes	-	10 m^3	500	2	
4	Embankment	Crest	Too low	Raise height of crest	-	150 m^3	4,500	2	Crest overtopped during IDF and inadequate freeboard.
5	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.
6	Conduit outlet pipe	Pipe sections	Check alignment of pipe sections	First check visually	-	-	-	2	Perform during time of low flow. By UTRCA.
7	Conduit outlet	Outlet	Partially blocked	Remove blockage	-	-	-	2	By UTRCA.
8	Conduit outlet	End of pipe	Damaged	Monitor and repair as required	-	-	-	5	By UTRCA.
9	Inlet structure	Concrete	Erosion occurring	Monitor and repair as required	4.1	-	-	5	By UTRCA.
							15,250		

Table 11.4

Budget Estimate Summary for Construction Costs for Maintenance Repairs for the Fullarton Dam

Item No.	Description	Unit	Quantity	Unit Price ($)	Amount ($)
1	Mobilization and demobilization	LS	1	1,500	1,500
2	Repairs to dam and structures	LS	1	15,250	15,250
3	**Subtotal (Construction Costs)**				**16,750**
4	Contingency on Construction Costs (25%)				4,250
5	**TOTAL ESTIMATED CONSTRUCTION COSTS**				**21,000**

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Appendix A

Appendix A

Photographs

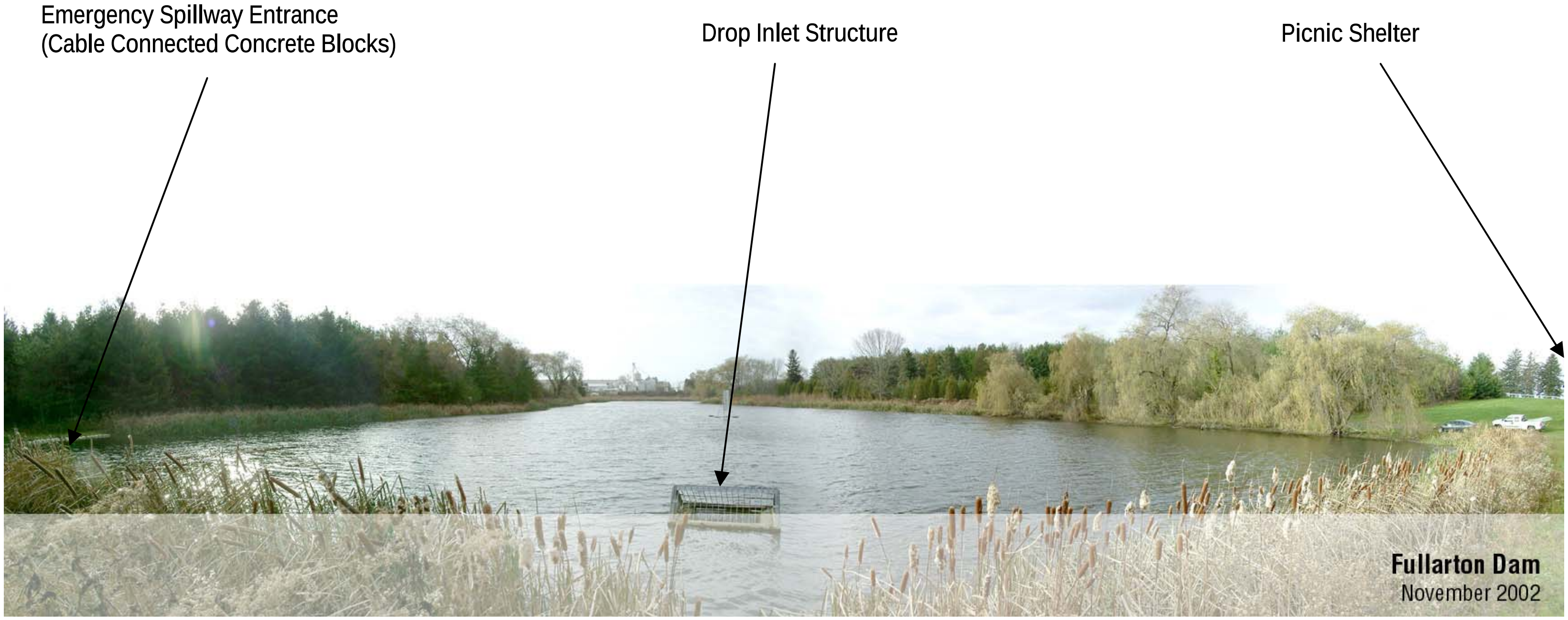

Photo 1 – Upstream View of Reservoir

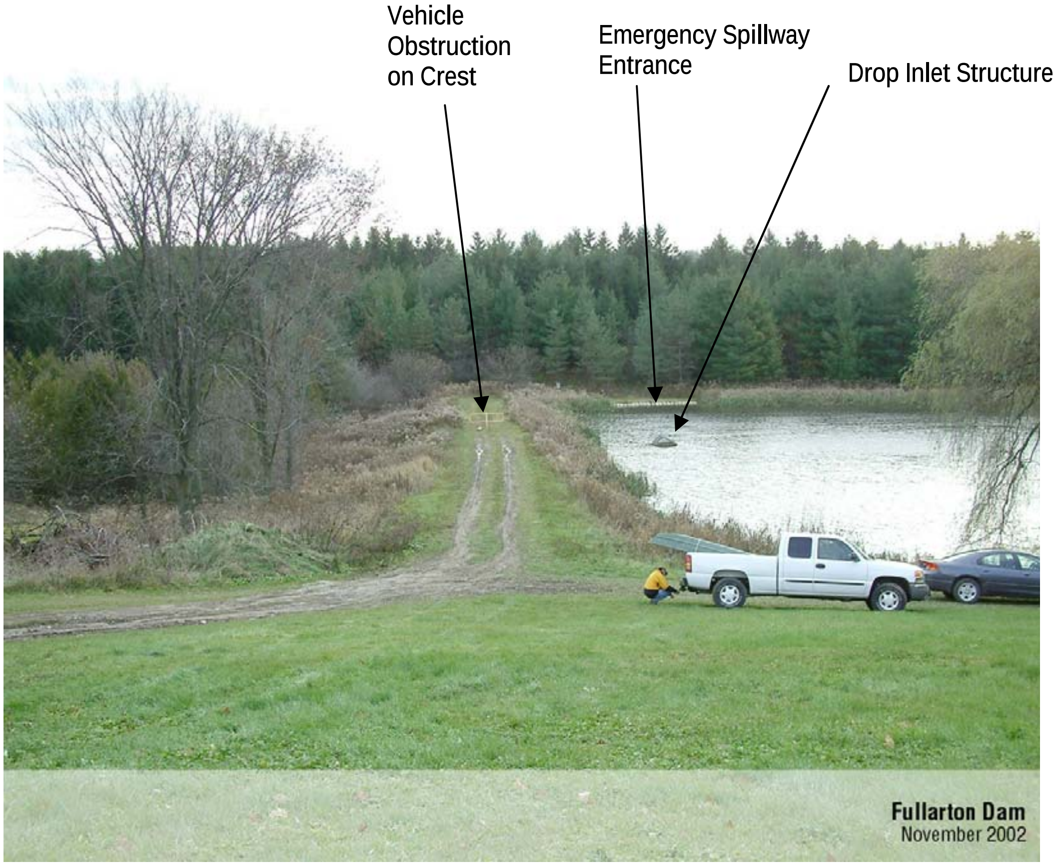

Photo 2 – View of Dam from Left Bank

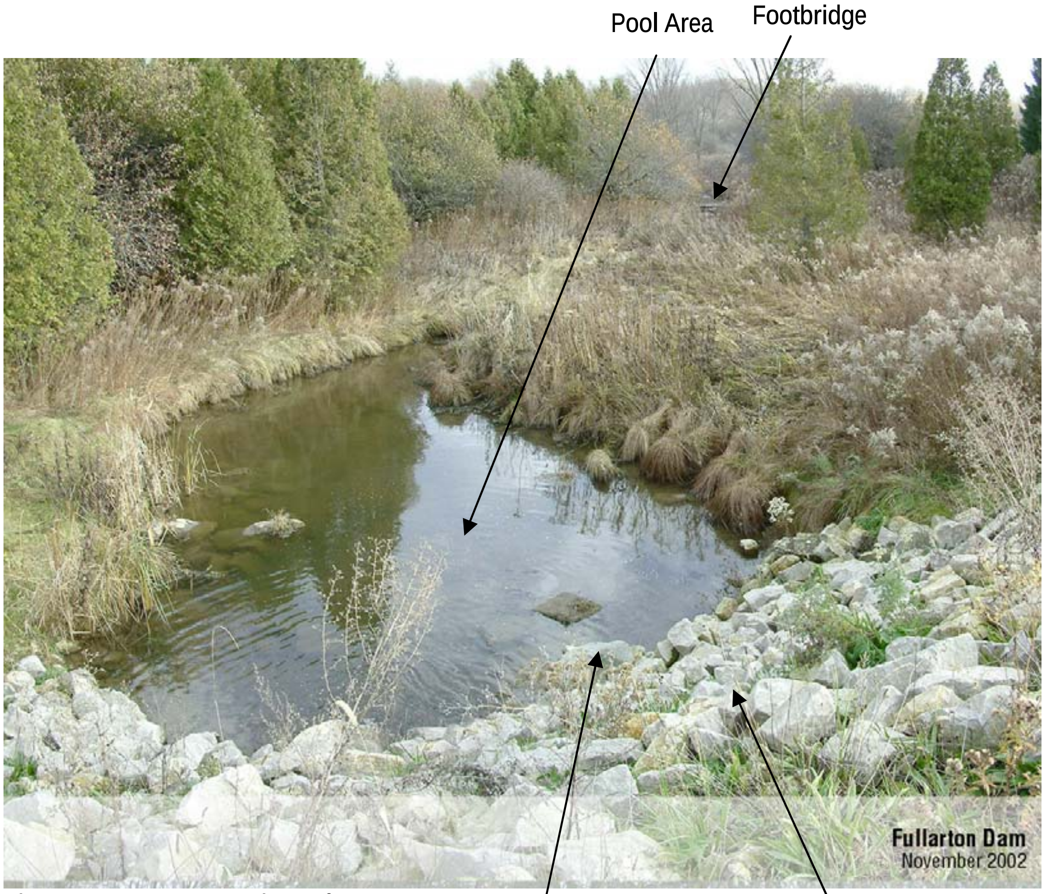

Photo 3 – Downstream Channel

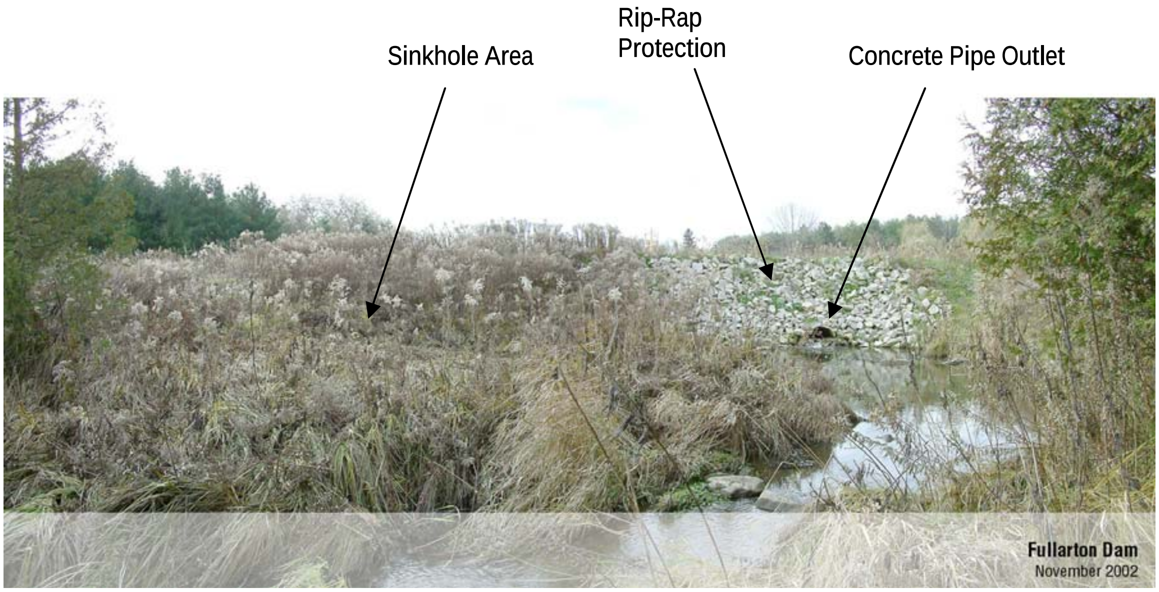

Photo 4 – Downstream View of Dam

Photo 5 – View of Emergency Spillway Entrance

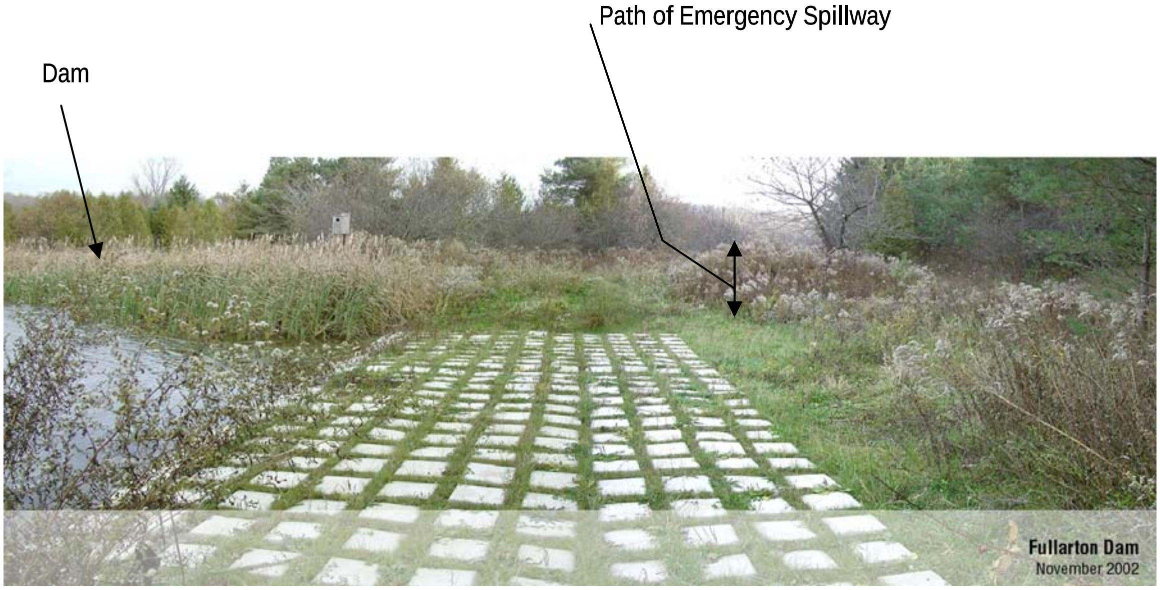

Photo 6 – Close-up of Cable-Connected Concrete Blocks at Emergency Spillway Entrance

Photo 7 – Upper Reach of Emergency Spillway Channel

Photo 8 – Wood Footbridge Across Downstream Channel

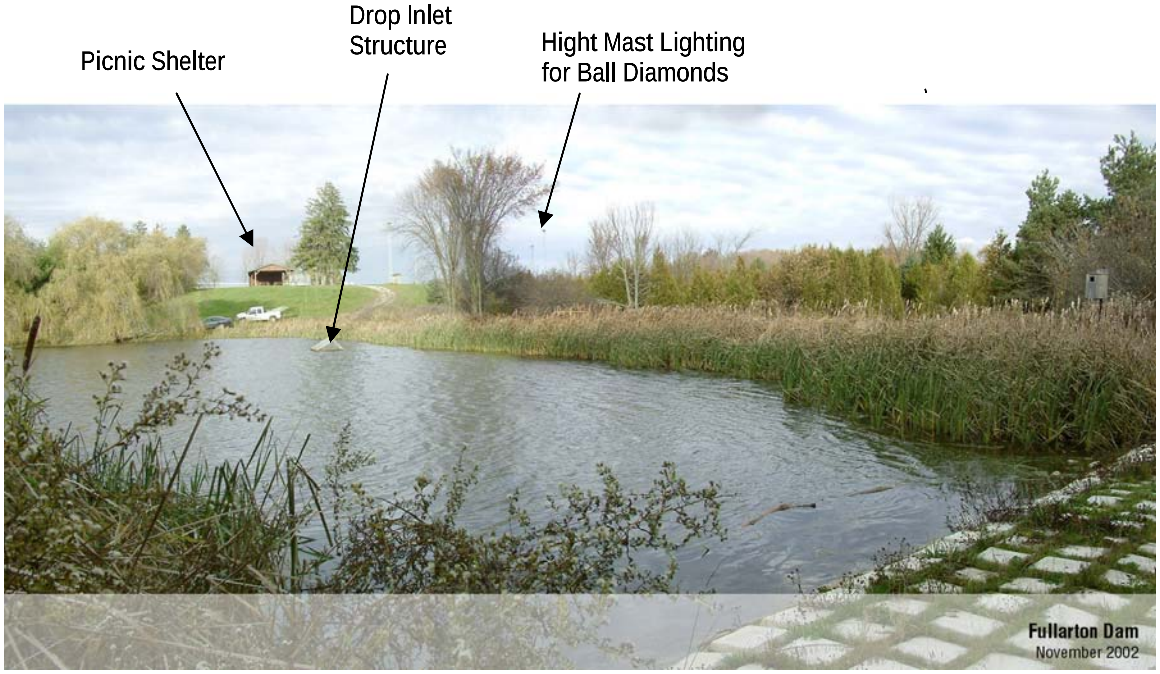

Photo 9 – Upstream Face of Dam

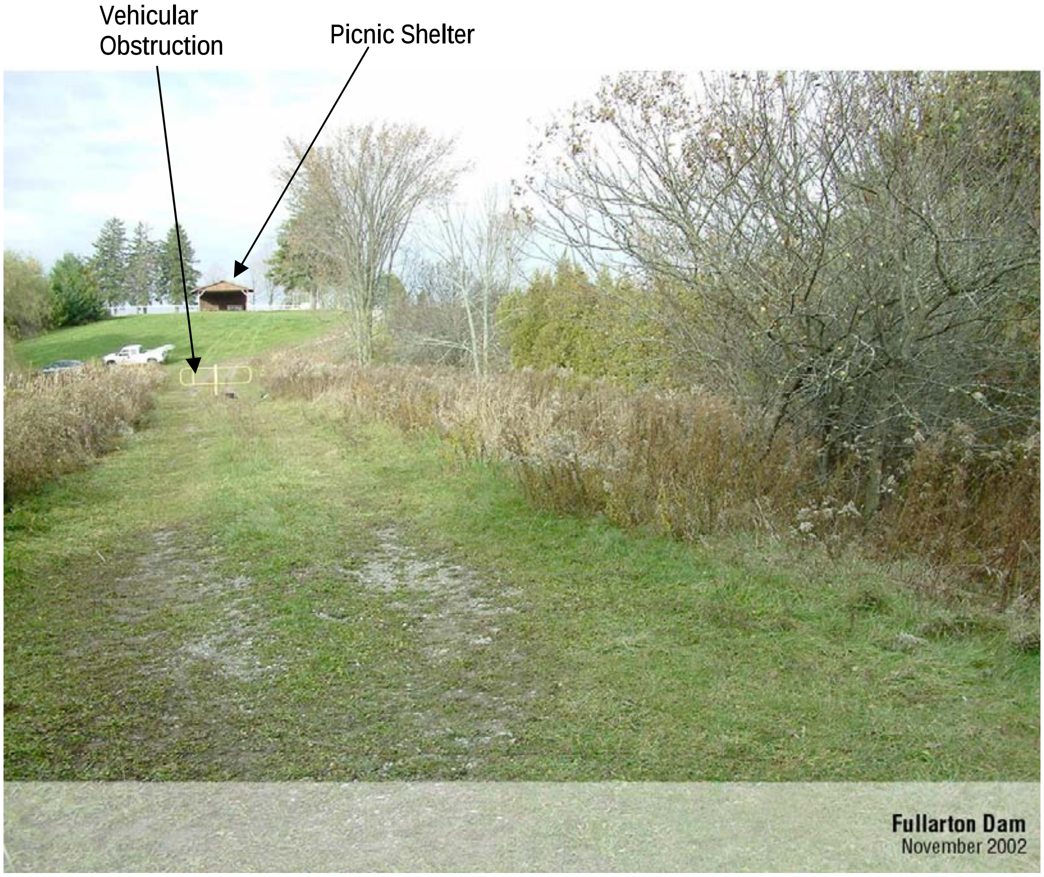

Photo 10 – View of Crest from Right Bank

Photo 11 – Close-Up of Outlet Pipe

Photo 12 – Drop Inlet Structure

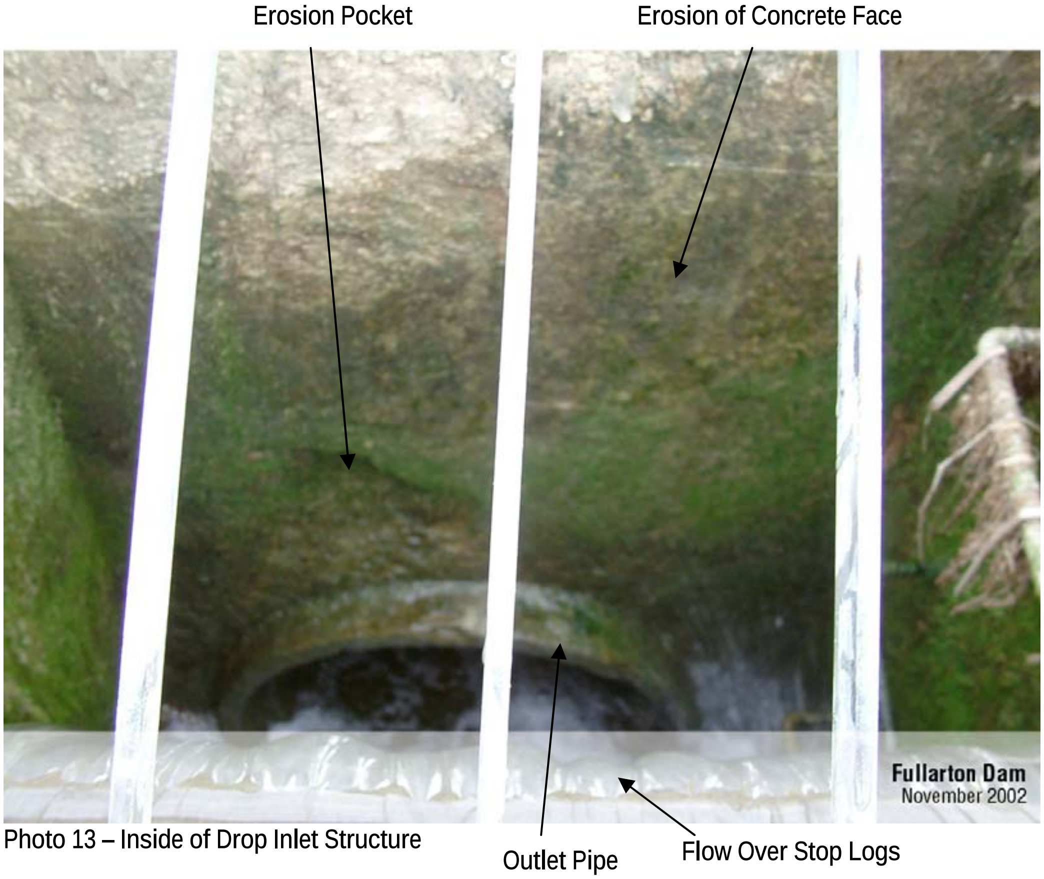

Photo 13 – Inside of Drop Inlet Structure

Appendix B

Appendix B

Forms B1 and B2

Form B1

Pre-Inspection Background Information

Prepared By: Acres International Limited

Name of Dam: Fullarton

Latest Construction:

2000	Lower and widen emergency spillway. Install cable-crete along emergency spillway approach. Repair last section of outlet pipe and install riprap above pipe section.
?	Stop logs replaced.
Fall 1985	Removed excess weed growth on emergency spillway.

Inspection Dates:

July 2001	UTRCA
July 2000	UTRCA
August 1985	UTRCA
July 1982	UTRCA

Access: Approximately 2 km south of the town of Fullarton, turn off into Fullarton Conservation Area from County Road 163

Lake Controlled: Fullarton Pond

Lake Area: 0.02 km^2

Watershed: Unnamed tributary of the North Thames River, North Thames River Watershed

Drainage Area: 4 km^2

Gauge Info: None at the dam

Rule Curves: Not available

List of Drawings: UTRCA:

#? Dam Hazard Identification, Fullarton 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 closest regional streamflow gauging station are

- North Thames River near Mitchell (Station 02GD014 drainage area = 306 km^2)
- Fish Creek near Prospect Hill (Station 02GD010; drainage area = 150 km^2); discontinued in 1995.

While these gauging stations are in the same watershed area, their drainage areas are much larger and are probably of limited value.

Topographic Maps: 40 P/6 St. Marys (1:50 000-scale)

Soil and Land-Use Maps: Soil Map of Perth County, Ontario (digitized, UTRCA) and The Upper Thames River Watershed Report Cards 2001

Dam Height: Approximately 3.4 m

Dam Length: Approximately 100 m

No. of Sluiceways: One overflow drop inlet structure

No. of Stop Logs per Bay: Unknown

Hydrologic Flows: Nothing available in files

Hydraulic Analysis: Nothing available in files

Dam Operation: Dam is not operated

Soils Reports: See Appendix A, Table 4. Soil Type in Upper Thames River Watershed, Report Cards 2001

Underwater Inspections: None available

Property Ownership: UTRCA

CA Maintenance: Township of West Perth

Dam Maintenance: UTRCA

Divestment Opportunities: West Perth will consider dam decommissioning if maintenance costs rise

Known Problems: Dam was overtopped in summer 2000. Erosion has occurred around outlet pipe. Inspection report identifies that crest elevation is lowest over outlet with exception to emergency overflow. Consider raising crest to prevent overtopping in this area.

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

Form B2

Dam Inspection Report

Date:	November 14 and 20, 2002
Structure:	Fullarton Dam
Municipality:	West Perth
Location:	2999 Road 163A, Lot 15, Conc. East of the Michell Road, Township of West Perth
GPS Coordinates:	UTM, NAD83: 17 482 909 E, 4 802 383 N Lat/Long: 43° 22’ 27” N, 81° 12’ 40” W
Inspected By:	B. Craig, T. Hartung, P. Last, M. Ragwen and B. Sinclair of Acres International Limited
Weather:	Partially sunny with cloudy conditions, air temperature approximately 8°C

---

1. Earth Embankment

For details, see the photographs in Appendix A.

- No sloughing, displacement, settlement or cracking seen along embankment.
- Localized erosion due to wave action along upstream shore of embankment.
- Few sinkholes located on downstream slope to the right of outlet, causes unknown.
- Washout around outlet pipe from dam overtopping has been repaired with riprap.
- Emergency overflow not well-defined at downstream end due to vegetation.

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

For details, see the photographs in Appendix A.

- Delamination near stop log gains on intake shaft.
- Hydraulic erosion inside intake shaft above outlet pipe fixture.
- Chipping of concrete edge at outlet pipe.
- Outlet pipe partially blocked by riprap.

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

For details, see the photographs in Appendix A.

Galvanized steel trashrack around drop shaft in good condition and well-anchored.

---

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

For details, see the photographs in Appendix A.

Stop logs located on upstream side of intake shaft. Not inspected due to high water overflowing structure. Logs are installed year-round and not manipulated.

---

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.

No winches are required on-site. Logs removed manually with aid of a punt boat and pick.

---

7. **Valves** (type and number)

For details, see the photographs in Appendix A.

None at this site.

---

8. **Boom** (driftwood, chains, anchors)

For details, see the photographs in Appendix A.

No safety boom present or required at this site.

---

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

Benching due to erosion has occurred locally on upstream side of embankment as indicated in Item 1.

---

10. **Seepage or Leaks**

For details, see the photographs in Appendix A.

No seepage was noticed through the embankments.

---

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

Vehicular access possible from left side of dam. UTRCA maintenance has keys to unlock access gate permitting vehicles to reach dam site. Dam is within walking distance from adjacent parking lot.

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

- No warning signs for public using the dam.

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13. **Signage**

For details, see the photographs in Appendix A.

- Only sign is one warning of unstable ice in winter.

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

West Perth will consider dam decommissioning if maintenance costs rise.

---

15. **General Remarks**

The dam is generally in good condition and well-maintained.

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16. **Recommendations**

- Clear downstream end of emergency overflow to ensure proper channel is provided.
- Regrade crest of dam over outlet pipe as required to prevent overtopping in this area.

Appendix C

Appendix C

Discussion on the Balanced Distribution

Appendix C

Discussion on the Balanced Distribution

In the case of dam safety studies, since the variation of rainfall depths in a storm is essentially random then the objective is to come up with a critical storm pattern, produced by re-arranging the rainfall excess pattern into the most critical sequence. There are two ways of approaching this selection. It can be based on either (a) the worst possible storm pattern or (b) an analysis of recorded storm distribution patterns (e.g., the F. A. Huff study in 1990). Acres conducted a brief review that examined the existing reports on the statistics of the frequency of hyetograph shapes that would be applicable to the Upper Thames watershed and its small dams. Acres examined and compared the Huff storm distributions (Huff, 1990), the AES 30% distribution, 6-hr Becker design storm, the 12-hr southern Ontario design storm and the SCS Type II storm distribution. Our proposal did not entail an in-depth study of historical storm patterns to arrive at a storm pattern applicable to dam safety studies. It should be remembered that in December 2002 when Acres received an e-mail containing the UTRCA VO2 model of the Upper Thames watershed there were problems with the VO2 model storm files that precluded their (and the model's) use at the time. As an alternative, it was agreed that the HEC-HMS rainfall-runoff model would be used for the smaller damsites (Dorchester Mill Pond, Dorchester CA, Centreville, Embro, Harrington and Shakespeare dams).

In Acres experience with dam safety studies, one can use a historical storm pattern from a set with a certain frequency of occurrence but there are disadvantages to this method. Consideration has to be given of joint probability and the fact that historical storms rarely repeat themselves. Also, the selection may not turn out to be the critical pattern. In addition, the consequences of climate change are causing significant changes in rainfall patterns (more intense and frequent storms) that would reduce the efficacy of the selection historical storm patterns for peak flows for dam safety assessment.

Because of our long experience with dam safety work, our practice is to use an IDF-generated hyetograph for the critical design storm for the key dam structures. Conservative considerations are made for the time-wise distribution of rainfall depths which must vary from zero at the beginning and end of the storm and rise to a maximum at some intermediate time increment. The smallest increment of rainfall depends upon the time of concentration of the river basin. Acres has found that although the storms are either front-loaded, back-loaded or center-loaded, it is the center-loaded IDF hyetographs that generate the highest peak flows. Because of this, in synthesizing the design hyetograph, the increment of maximum rainfall intensity is generally placed somewhere near the midpoint of the storm and the hyetograph developed using the alternating block method (Chow et al, 1988).

In other studies, Acres analyzed three patterns of hyetographs

(a) center-loaded (derived by the alternating block method)
(b) SCS Type II, and
(c) SCS Type II but with the highest rainfall intensity occurring at the end of the first quartile of the storm duration.

The center-loaded storm pattern produced the highest peak outflow although this was only 0.3% larger than outflow from the SCS Type II storm pattern. For all practical purposes, the center-loaded storm pattern gives the same outflow as the SCS Type II storm. In our view, the use of the SCS Type II storm pattern would have resulted in very similar outflows to the center-loaded storms used in our analysis. The original SCS Type II distribution itself was developed using the ‘balanced’ or ‘alternative block’ method and then made dimensionless by the United States Department of Agriculture. The SCS front end-loaded storm which is similar to the AES storm pattern, produced a peak outflow that was 6% less than the center-loaded distribution.

Acres did examine the 30% AES storm distribution data that UTRCA submitted in 2003. The basic criterion is that the aggregated incremental rainfall in the distribution must be equal to the corresponding value in the AES DDF data. In both the 12-hr and 24-hr 1:50-yr storms using UTRCA percentages, we found that these exceeded the amounts that were in the DDF data and were, therefore, outside of the DDF results. These were amended and the percentages adjusted to conform to the DDF data and the distribution changed accordingly. They were still front end-loaded storms and the resulting flood peaks were lower than the center-loaded storms. The result was expected since the SCS loss function is also front end-loaded. These were not considered appropriate for dam safety assessment of the dams’ discharge capacities.

Appendix D

Appendix D

Balanced Distribution Curves

(Tables D1 to D7)

Table D1

Balanced Distribution
6-Hr Rainstorm Based on Stratford Rainfall Data

Duration (hrs)	Cumulative Percentage of Storm Depth
0.0	0.00
0.3	1.75
0.5	3.49
0.8	5.24
1.0	6.99
1.3	8.73
1.5	10.48
1.8	12.23
2.0	13.97
2.3	19.51
2.5	25.05
2.8	30.76
3.0	57.08
3.3	70.24
3.5	75.77
3.8	81.31
4.0	86.03
4.3	87.77
4.5	89.52
4.8	91.27
5.0	93.01
5.3	94.76
5.5	96.51
5.8	98.25
6.0	100.00

Table D2

Balanced Distribution 12-Hr Rainstorm Based on Stratford Rainfall Data

Duration (hrs)	Cumulative Percentage of Storm Depth
0.0	0.00
0.5	0.74
1.0	1.48
1.5	2.21
2.0	2.95
2.5	3.69
3.0	4.43
3.5	7.61
4.0	10.79
4.5	13.98
5.0	17.16
5.5	27.25
6.0	63.23
6.5	73.33
7.0	82.84
7.5	86.02
8.0	89.21
8.5	92.39
9.0	95.57
9.5	96.31
10.0	97.05
10.5	97.79
11.0	98.52
11.5	99.26
12.0	100.00

Table D3

Balanced Distribution 24-Hr Rainstorm Based on Stratford Rainfall Data

Duration (hrs)	Cumulative Percentage of Storm Depth
0.0	0.00
1.0	0.53
2.0	1.06
3.0	1.59
4.0	2.12
5.0	2.65
6.0	3.18
7.0	4.56
8.0	5.94
9.0	7.32
10.0	13.28
11.0	19.25
12.0	61.85
13.0	80.75
14.0	86.72
15.0	92.68
16.0	94.06
17.0	95.44
18.0	96.82
19.0	97.35
20.0	97.88
21.0	98.41
22.0	98.94
23.0	99.47
24.0	100.00

Table D4

Balanced Distribution 2-Day Rainstorm Based on Stratford Rainfall Data

Duration (hrs)	Cumulative Percentage of Storm Depth
0.0	0.00
2.0	0.38
4.0	0.76
6.0	1.15
8.0	1.53
10.0	1.91
12.0	2.29
14.0	3.30
16.0	4.31
18.0	5.32
20.0	7.96
22.0	19.34
24.0	78.02
26.0	89.41
28.0	92.04
30.0	94.68
32.0	95.69
34.0	96.70
36.0	97.71
38.0	98.09
40.0	98.47
42.0	98.85
44.0	99.24
46.0	99.62
48.0	100.00

Table D5

Balanced Distribution 3-Day Rainstorm Based on Stratford Rainfall Data

Duration (hrs)	Cumulative Percentage of Storm Depth
0.0	0.00
2.0	0.35
4.0	0.69
6.0	1.04
8.0	1.39
10.0	1.73
12.0	2.08
14.0	2.85
16.0	3.63
18.0	4.41
20.0	5.18
22.0	5.96
24.0	6.74
26.0	7.65
28.0	8.57
30.0	9.48
32.0	11.87
34.0	22.20
36.0	75.41
38.0	85.74
40.0	88.13
42.0	90.52
44.0	91.43
46.0	92.35
48.0	93.26
50.0	94.04
52.0	94.82
54.0	95.59
56.0	96.37
58.0	97.15
60.0	97.92
62.0	98.27
64.0	98.61
66.0	98.96
68.0	99.31
70.0	99.65
72.0	100.00

Table D6

Balanced Distribution 5-Day Rainstorm Based on Stratford Rainfall Data

Duration (hrs)	Cumulative Percentage of Storm Depth
0.0	0.00
6.0	0.93
12.0	1.85
18.0	2.79
24.0	3.73
30.0	5.24
36.0	6.75
42.0	8.85
48.0	10.95
54.0	13.43
60.0	80.10
66.0	86.57
72.0	89.05
78.0	91.15
84.0	93.25
90.0	94.76
96.0	96.27
102.0	97.21
108.0	98.15
114.0	99.07
120.0	100.00

Table D7

Rain-on-Snowmelt Distribution Pattern for Gauge A 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	0.000	0.0	0.000
1.0	1.00	2.0	0.866	2.0	0.397
2.0	2.00	4.0	1.732	4.0	0.794
3.0	3.00	6.0	3.609	6.0	1.686
4.0	4.00	8.0	5.486	8.0	2.678
5.0	5.50	10.0	9.095	10.0	4.364
6.0	7.00	12.0	12.704	12.0	6.051
7.0	9.00	14.0	14.581	14.0	6.943
8.0	11.00	16.0	16.313	16.0	7.836
9.0	14.50	18.0	17.180	18.0	8.233
10.0	18.00	20.0	18.046	20.0	8.629
11.0	26.00	22.0	18.046	22.0	8.629
12.0	34.00	24.0	18.046	24.0	8.629
13.0	53.50	26.0	18.985	26.0	9.055
14.0	73.00	28.0	19.924	28.0	9.480
15.0	79.50	30.0	21.803	30.0	10.437
16.0	86.00	32.0	23.798	32.0	11.500
17.0	89.00	34.0	27.673	34.0	13.308
18.0	92.00	36.0	31.547	36.0	15.116
19.0	94.00	38.0	33.425	38.0	16.179
20.0	96.00	40.0	35.304	40.0	17.136
21.0	97.00	42.0	36.243	42.0	17.561
22.0	98.00	44.0	37.182	44.0	17.986
23.0	99.00	46.0	37.182	46.0	17.986
24.0	100.00	48.0	37.886	48.0	17.986
		50.0	39.271	50.0	18.466
		52.0	41.249	52.0	18.946
		54.0	43.821	54.0	19.906
		56.0	48.173	56.0	20.866
		58.0	58.261	58.0	22.786
		60.0	82.988	60.0	24.706
		62.0	91.296	62.0	25.666
		64.0	95.055	64.0	26.626
		66.0	97.626	66.0	27.106
		68.0	98.813	68.0	27.586
		70.0	100.000	70.0	27.586
		72.0	100.000	72.0	27.586
				74.0	28.076
				76.0	28.566

Table D7

Rain-on-Snowmelt Distribution Pattern for Gauge A 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	29.628
				80.0	30.690
				82.0	32.731
				84.0	34.773
				86.0	35.835
				88.0	36.815
				90.0	37.305
				92.0	37.795
				94.0	37.795
				96.0	37.795
				98.0	38.326
				100.0	38.857
				102.0	39.920
				104.0	41.049
				106.0	43.241
				108.0	45.432
				110.0	46.495
				112.0	47.557
				114.0	48.089
				116.0	48.620
				118.0	48.620
				120.0	49.019
				122.0	49.802
				124.0	50.921
				126.0	52.376
				128.0	54.838
				130.0	60.545
				132.0	74.533
				134.0	79.233
				136.0	81.359
				138.0	82.814
				140.0	83.485
				142.0	84.157
				144.0	84.157
				146.0	84.525
				148.0	84.893
				150.0	85.722
				152.0	86.642
				154.0	88.207
				156.0	89.772

Table D7

Rain-on-Snowmelt Distribution Pattern for Gauge A 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	90.601
				160.0	91.429
				162.0	91.798
				164.0	92.166
				166.0	92.166
				168.0	92.166
				170.0	92.545
				172.0	92.924
				174.0	93.682
				176.0	94.440
				178.0	96.083
				180.0	97.726
				182.0	98.484
				184.0	99.242
				186.0	99.621
				188.0	100.000
				190.0	100.000
				192.0	100.000

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 E

Appendix E

MNR Dam Safety Bulletins

DRAFT

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.

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 F

Appendix F

Dam Operator Questionnaire

FULLARTON

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: GLENGOWAN Site: FULLARTON
Prepared by: CHRIS TASKER Date: Feb 14/2003
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**	**Number**
Sluices –gate NA	
Sluices –log WOODEN 2x6	unknown
Sluices –valve (Manufacturer, size, type, etc.)	
Debris boom TRASH RACK	
Non-overflow walls NA	
Spillways/overflow walls ~~NA~~ emerg.	1
Upstream retaining walls NA	
Downstream retaining walls NA	
Other – Drop inlet	
2. Elevation Datum (Canadian Geodetic Datum (CGD) or other - specify)	top of logs

Part II - General Operational Information

3. Please list any major repairs/maintenance since construction that you know of.	- lower, widen emergency s/w - rip-rap above pipe - reposition last pipe - installation of cable-crete on emergy s/w approach - stop logs replaced
4. (a) Who operates this site? N/A	☐ Contractor ☐ Other Contact person Legal Agreement in place?
(b) How many staff are normally available to operate the site?	N/A
(c) Is this person/team responsible for operating other sites? (d) If yes, where?	☐Yes ☐No N/A
(e) If answer to (c) is yes, is there sufficient staff to operate these sites simultaneously?	☐Yes ☐No N/A
(f) If answer to (e) is no, is other assistance available? (g) If yes, who and from where?	☐Yes ☐No N/A
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 (b) Summer/Fall (c) Winter If yes, please describe (e.g. will the access road or a bridge be accessible if there is a major flood?)	☐ Yes ☑ No ☐ Yes ☑ No ☐ Yes ☑ No
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 Log removal under high flows not possible
10. How many staff members are required to achieve maximum spill capacity for the above time estimate?	N/A
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
13. (a) Is there a water level gauge at this site? (b) If no, is there a gauge at a dock nearby? (c) What is the location of the gauge (if applicable)? (d) To what is this gauge referenced? (e) Is the gauge metric or imperial?	☐ Yes ☑ No ☐ Yes ☑ No ☐ CGD ☐ Local structure datum ☐ Other datum ☐ 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 D/S + on pond
15. (a) What other agencies are involved with flow regulation along the river? (b) Who are the contact persons?	N/A
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 Ice unstable
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 Trash rack ☐ Chained ☐ Cable strung ☐ Permanent ☐ Seasonal ☐ Yes ☐ No ☐ Permanent ☐ Seasonal
22. What structural aspects of the dam do you inspect during operational visits?	-No operational visits - Monthly ~~[illegible]~~ visits during summer - Embankment + Areas D/S - Emerg SW etc

23. Log Settings	Gauge CGD local
(a) What is the normal regulated water level	
(b) How many logs are usually in for the normal summer setting?	
(c) How many logs are normally removed for the winter drawdown condition?	None
(d) How many logs can actually be removed in an emergency?	None - once WL raised log can't be removed
(e) Is the bottom log fixed in place and not removed?	~~☐Yes ☐No~~ NA

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?	☐Yes ☑No ☐Not applicable
(b) If yes, under what headwater elevation and when?	
(c) If no, what is the constraint?	Sediment release
B Operating Equipment	
27. Type of equipment used to operate the discharge facilities:	punt + pick
(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?	☐ Yes ☑ No ☐ Not applicable
(b) Is auxiliary power available?	☐ Yes ☑ No ☐ Not applicable
(c) If yes, specify source.	
29. (a) Is the discharge facility operating equipment located at the site (keys, winch handles, chain falls, etc.)?	☐ Yes ~~☑~~ No ☑ Not applicable
(b) If no, where are they located?	
(c) Is there more than one set?	☐ 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?	☐ Yes ☐ No ☑ Not applicable
(c) If yes, is the backup located on site? (d) If no, where is backup located?	☐ 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?	☐ Yes ☐ No ☑ Not applicable
(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
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 once WL is elevated

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 rubble & 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 clean trash rack when debris builds up
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 None
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.)	- overtopping caused washout at pipe outlet - have lowered emerg SW & increased capacity
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 ☐ 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).	drop inlet top

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	
emerg SW											
drop inlet											

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

Drawings

Plot Scale PLOTSCALE
Aug 28 , 2007 , 1:42pm Login name: Park110733
Drawing Name: C:\Work\14504_Upper Thames\Fullarton\14504-FT-001.dwg

0 60 120 180 240 300 Metres

Scale 1:3000 (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.
2004	A	ISSUED WITH REPORT	B.C.	P.L.	M.M.

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

FULLARTON DAM
KEY PLAN

SCALE

ACRES PROJECT NO.
P14504.02

DRAWING NO.
14504–FT–001

REVISION
B

CIVIL ELECTRICAL MECHANICAL GEOTECHNICAL HYDROTECHNICAL ARCHITECTURAL OTHER

B

DRAWING No. 14504–FT–001

Scale 1:750 (plotted as A1 size sheet)

LEGEND

● BH1 IDENTIFICATION NUMBER 2003 YEAR OF INVESTIGATION	BOREHOLE	

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

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

FULLARTON DAM
SITE PLAN

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

CIVIL ELECTRICAL MECHANICAL GEOTECHNICAL HYDROTECHNICAL ARCHITECTURAL OTHER

Plot Scale PLOTSCALE
Aug 28 , 2007 , 1:42pm Login name: Park110733
Drawing Name: C:\Work\14504_Upper Thames\Fullarton\14504–FT–002.dwg

B

DRAWING No. 14504–FT–002