Long Point Region Conservation Authority

Final Report

Dam Safety Review
Deer Creek Dam

Norfolk County, Ontario

D.M. Wills Project Number 24-5684



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2	Draft Report	March 5, 2025	Issued for Client Review
3	Final Report	April 23, 2025	Issued Final

This report/proposal has been formatted considering the requirements of the Accessibility for Ontarians with Disabilities Act.

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1.0 Project Initiation, Data Collection and Site Inspection

1.1 Introduction

D.M. Wills Associates Limited (Wills) was retained by the Long Point Region Conservation Authority (LPRCA) to undertake a Dam Safety Review (DSR) for the Deer Creek Dam on Deer Creek near the Deer Creek Conservation Area at 1929 Norfolk County Road 45, Town of Langton, Norfolk County, Ontario.

A full DSR has not been undertaken for the Deer Creek Dam, therefore the Hazard Potential Classification (HPC), Inflow Design Flood (IDF), adequacy of hydraulic capacity and stability factors of safety are not known. In order to address the limited dam safety information, the LPRCA and Wills prepared a detailed scope of work and work plan. Section 1.2 provides a general description of the scope of work for the project.

1.2 Scope of Work

Based on the Request for Proposal (RFP) issued by the LPRCA, dated August 12, 2024, and Wills' proposal, dated August 29, 2024, the scope of work for the Dam Safety Review includes the following activities:

- Background Data Collection and Review.
- Dam Inspection and Operating Staff Interviews.
- Hydrology Study and Hydraulic Model Development.
- Hydraulic Model Development and Detailed Dam Breach Assessment.
- Inundation Mapping and Incremental Loss Assessment.
- HPC and IDF Selection.
- Access Spillway Capacity and Freeboard for Inflow Design Flood.
- Structural (Gravity Dam) Stability Assessment.
- Geotechnical (Earth Embankment) Stability Assessment.
- Review of Dam Operation Practices.
- Review of Emergency Management Practices.
- Review of Public and Operator Safety.
- Develop and Prioritize Recommendations for Identified Deficiencies.
- Develop Cost Estimates for Recommended Works.

1.3 Site Location

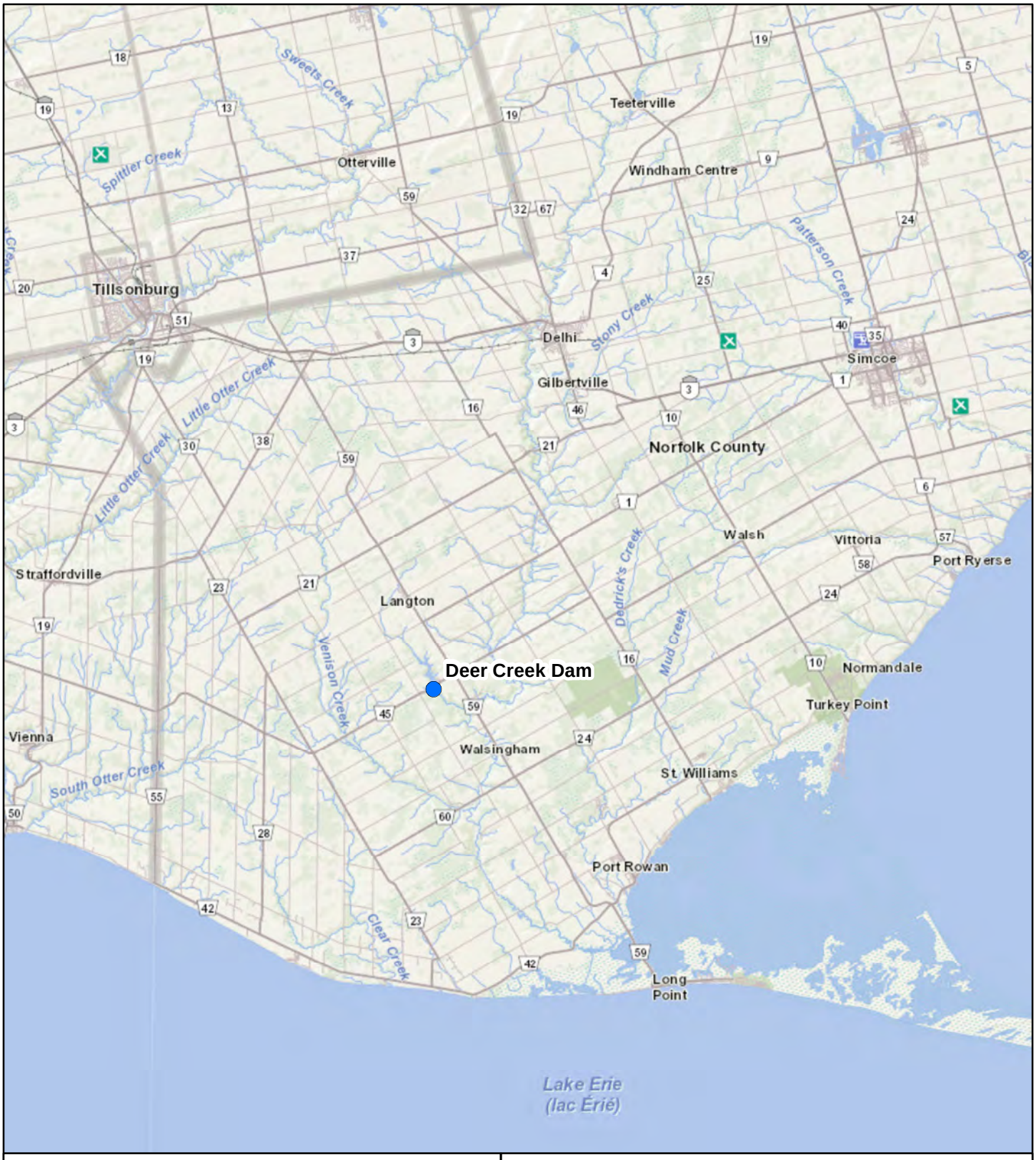
The Deer Creek Dam is located near 1929 Norfolk County Road 45, Town of Langton, Norfolk County, Ontario. The location plan is provided in Figure 1. The Deer Creek Dam is approximately 2 km north of Deer Creek's confluence with Big Creek and approximately 21 km upstream of Lake Erie.

1.4 Guidelines

In August 2011, the Ministry of Natural Resources (MNR), replaced the 1999 (Draft) Ontario Dam Safety Guidelines (ODSG) with a series of Dam Safety Technical Bulletins under Sections 14 and 16 under the Lakes and Rivers Improvement Act (LRIA). Following the release of the 2011 Technical Bulletins by the MNR, the bulletins became the standard for the review and analysis of dams in Ontario. Consequently, Wills has used the criteria within the August 2011 Technical Bulletins for this assignment. Where the information contained in the Technical Bulletins is not sufficient, Wills relied on the Canadian Dam Association (CDA) Dam Safety Guidelines and Technical Bulletins or other relevant documentation, as appropriate.

1.5 Vertical Datum

All elevations within this report are referenced to the CGVD2013 vertical datum. This vertical datum was used for the project since it is the vertical datum used by the LiDAR Digital Terrain Model (DTM) that was available from the Province of Ontario. Elevations on the as built drawings provided by the LPRCA were assumed to be referenced to the job datum which was 304.8 mm below CGVD28:78 vertical datum. Where required, transformations between the two datums were made by subtracting 0.451 m from the CGVD28:78 elevation to adjust to the CGVD2013 vertical datum. This is the difference in the published elevations at MNR COSINE Station 00819788110, which is located on Highway 59, just south of Norfolk County Road 45.



Legend

- Deer Creek Dam

N

NAD 1983 CSRS UTM Zone 17N

1:250,000

Created in ArcMap 10.7
Queens Printer for Ontario, 2024
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0 2,500 5,000 10,000 m

Figure 1 - Location Plan

Drawn By:	DG
Checked By:	DG
Map Date:	1/17/25
Project Number:	5684
Map File:	20250117-5684

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1.6 Site Reconnaissance

1.6.1 Introduction

Wills undertook a site inspection of the Deer Creek Dam on October 29, 2024. The purpose of this site visit was to review the dam and its operation with LPRCA staff, and to undertake a detailed inspection of the dam, including its various components, public safety measures and operator safety measures.

1.6.2 Dam General Description



The Deer Creek Dam was originally constructed in the late 1960's by the Big Creek Conservation Authority. The dam came under the ownership of the LPRCA in 1971 after the Big Creek Region and Otter Creek Conservation Authorities merged to form the LPRCA. The dam is located on Deer Creek, a tributary of Big Creek, in the former Township of North Walsingham, Norfolk County and is approximately 21 km upstream of Lake Erie. Surrounding the dam and its reservoir is the Deer Creek Conservation Area which is owned by the LPRCA. The dam currently serves as a feature to the Deer Creek Conservation Area and has historically provided low flow augmentation.

The Deer Creek Dam is approximately 160 m long and is comprised of two earth embankment sections that are bisected by a principal concrete spillway. According to the drawings provided, the earth embankment was constructed as a zoned embankment dam with an impervious core and sheet pile cut off wall that extends into native material. The impervious core is encased in a pervious shell with granular filters and 300 mm (12 in) toe drains on the downstream embankment. The spillway is a concrete gravity weir with a concrete chute that extends approximately 95 m (measured horizontally) to the toe of the embankments. The weir has a width of 15.2 m (50 ft) featuring an ogee inlet configuration with two stop log bays filled with logs to the ogee invert. The stop logs are not easily accessible and remain in place year-round. There is a 914 mm (36 in) diameter concrete low flow pipe that extends 122 m (400 ft) through the embankment from the reservoir to a secondary concrete stilling basin. The pipe is controlled by a valve that is housed in a concrete lift chamber on the upstream side of the eastern embankment and is used to drain the reservoir for maintenance or provide low flow augmentation.

Public safety measures have been installed at the site, including public safety signage at various locations around the stie and a buoy line on the upstream side of the concrete control structure.

1.6.3 Dam General Condition

The following is a summary of the general condition of the dam. Additional details are provided on MNRF Form B-2, Dam Inspection Report, which is included in Appendix A. Selected photographs are also included in Appendix A. The locations of specific dam features are referenced looking downstream. In general, the dam appeared to be in fair to good condition with some notable deficiencies described below.

1.6.3.1 Earth Embankments

Left Embankment – A large earth fill embankment exists on the left side of the spillway structure and bridge. The earth dam is in good condition with no noted locations of seepage or erosion. The upstream surface is protected with rock protection that was in generally good condition; however, some riprap is cracking/disintegrating. The sloughing/settlement that was noted during the previous inspection in 2014 does not appear to have gotten worse. Minor amounts of vegetation were found to be growing between the riprap on the downstream embankment. The downstream embankment was cleared of thick vegetation prior to the inspection and looked to be in good condition.

Right Embankment – A more natural embankment exists to the right of the spillway structure and bridge. The roadway embankment is in good condition and well vegetated.

1.6.3.2 Concrete Structure

Lift Control Chamber, Water Intake Tower, and Concrete Bypass Pipe – Limited inspection due to access. The majority of the concrete piping is permanently submerged and / or buried. A review of the GoPro video taken at the outlet reveals surface organics growth. The Lift Control Chamber appeared to be in good condition with recent concrete repairs.

Bypass Outlet Structure – The bypass outlet structure is generally in good to fair condition. However, there is a large area of spalling, delamination, and disintegration occurring at the interface with the angled face of the outlet wall. Severe map cracking with efflorescence is present on both side of the structure.

There is ±900 mm of sedimentation at the outlet structure. The baffle blocks / energy dissipators and toe drain outlets are completely buried.

Left Wingwall – The wingwall has been refaced as part of past rehabilitation. The exposed concrete is in good condition with no spalls or delaminations. A portion of the walls extending into upstream reservoir had limited inspection due to access limitations. The portion of the wall below the water level is covered in sheet piling.

Right Wingwall – The condition is similar to the left wingwall.

Left Bridge Abutment – The concrete is in good condition below bearing seat. This portion of spillway is assumed to be a County (Norfolk) asset. Erosion at the base of the wall is opening a gap between the abutment wall and spillway slab.

Right Bridge Abutment – The condition is similar to the left abutment.

Spillway Structure (Upper) – The walls on either side of the spillway slab have been partially refaced and patched. The newer concrete is in good condition, with the exception of a small delamination near the base of the right wall. Exposed original walls are in good condition with localized deterioration in the form of cracks with efflorescence and delamination. The culvert outlet on the right side of the spillway is delaminated around its perimeter as concrete appears to be starting to disintegrate.

The spillway slab is in good condition with light erosion where the original concrete is visible. The majority of the slab concrete is covered with an organic deposit that resembles disintegrated concrete with small aggregates.

Spillway Structure (Middle) – Similar to the upper portion of the spillway, the slab of the middle section of spillway is largely covered in a 20-50 mm thick layer of weak concrete-like material, assumed to be organic, and is superficial to the structural concrete of the underlying slab. Where exposed, the original concrete of the slab appears to be in good condition and is sound to hammer impact.

Transverse construction joints are present in the spillway slab and walls at ±7.47 m (24.5') intervals. Deterioration of the walls and slab of the spillway appear to be localized to the construction joint locations. Several wall joints are fully delaminated with some spalls and disintegration of underlying concrete. A PVC waterstop is exposed between slabs 6 and 7 near the interface with the left wall. In general, the construction joints in the slabs appear to be in good condition and functioning well.

A review of the embankment-side face of the wall at construction joint locations reveals that several disintegration patterns, and associated delaminations, appear to be reflected on both sides of the wall, full depth spalling is likely to occur at these locations in the future.

Lower Spillway Stilling Basin – The lowest portion of the spillway (stilling basin) is mostly covered in silt, vegetation, and debris (±500 thick) and could not be inspected due to limited exposure / visibility.

1.6.3.3 Wooden and Metal Structures

Chain-link Fencing – 5 foot tall galvanized fencing is present on both sides of the spillway channel, side mounted near the top of the wall. The fencing and posts are generally in good condition with no obvious forms of deterioration.

Gain Armouring – Limited inspection due to flow. Assumed fair to good condition.

Wing Wall Sheet Piling – Piling is in good condition with light uniform surface rust (patina) above water. Organic film covers the majority of the underwater steel face.

Embankment Sheet Piling – Sheet piles are illustrated on the original plans for the dam in the core of the left embankment. The presence and condition of these piles could not be reviewed during the site inspection.

Subdrain System – Inspection was limited to what was visible from the base of the spillway chute. The 8" perforated CSP subdrain on the left spillway chute wall had moderate corrosion with section loss at the bottom of the pipe. The 8" perforated CSP subdrain on the right spillway chute wall is in very poor condition and is disjointed/crushed with coarse grained material entering the pipe as well as moderate corrosion. Course grained material upstream of the disjointed area indicates this pipe may be damaged in more than one location.

1.6.3.4 Gates and/or Stop Logs

The top stoplogs are in poor condition with severe rot and decay. The same concrete like organic material that is coating the spillway is present on the downstream side of the stoplogs. Removing a small patch of this material showed the stop logs to be in fair condition, but inspection was limited. There is minor leakage between the stoplogs.

1.6.3.5 Valves

The valve system is operated from the lift control chamber using an Armco CPE-12 with removable hand crank. The valve system was operated by LPRCA staff during the inspection (25 rotations opened, 25 rotations closed) and appears to be in working condition with significant sediment discharge at the outlet when opened. The paint is peeling on the valve system and light corrosion is visible. The valve stem exits the lift chamber and appears to be in good condition. Inspection of the slide gate was not possible as it was submerged and potentially buried under sediment/organic material.

It is understood that LPRCA operates the valve system semi-annually to flush sediment and greases the valve system with a grease gun.

1.6.3.6 Water Level Gauges

There is no water level gauge at the Deer Creek Dam. However, it is understood that there is a staff gauge in the reservoir near the boat ramp. The staff gauge was not inspected.

1.6.3.7 Erosion and Seepage

Minor erosion was observed on the banks of the downstream bypass channel. There is minor leakage between the stoplogs.

1.6.3.8 Public Safety

There are medium public safety signs (1220 mm x 1220 mm) mounted on wooden posts facing upstream on the left and right embankments. The sign reads "DANGER, No Trespassing, Access Beyond This Point May Result in Drowning". The signs also have the name of the dam, and "In An Emergency call 519-842-4242 [LPRCA's office] or 911", as well as LPRCA's logo. The signs appear to be in good condition.

There is a medium public safety sign (1220 mm x 1220 mm) mounted to wooden posts on top of the concrete bypass structure tower facing the road. The sign reads

"DANGER, No Trespassing, Access Beyond This Point May Result in Drowning". The signs also have the name of the dam, and "In An Emergency call 519-842-4242 [LPRCA's office] or 911", as well as LPRCA's logo. The signs appear to be in good condition.

There are small public safety signs (600 mm x 600 mm) mounted to wooden posts on the left and right approaches to the dam. The signs read "DANGER, Dam Ahead – Keep Out". The signs also have the name of the dam, and "In An Emergency Call 911", as well as LPRCA's logo. The signs appear to be in good condition.

There is a small public safety sign (600 mm x 600 mm) mounted to the chainlink fencing facing the road on the far side of the east embankment. The sign reads "DANGER, No Trespassing". The sign appears to be in good condition but is becoming overgrown with vegetation.

There is a small public safety sign (600 mm x 600 mm) mounted to the chainlink fencing facing the road on the far side of the west embankment. The sign reads "DANGER, Keep Off Dam". The sign is in poor condition.

There is a small "No Trespassing" sign mounted to the chainlink fencing facing the road on the far side of the west embankment. The sign is in fair condition.

1.6.3.9 Operator Safety

No operator safety issues were identified.

1.7 Topographic and Bathymetric Surveys

Wills completed topographic surveys of the dam, and the bridges at Walsingham Townline Road, 6th Concession Road, and Norfolk County Road 60 between November 6, 2024 and November 8, 2024. The survey was limited to the key topographic features required slope stability modeling of the dam, and hydraulic properties. The survey was completed using a survey grade GNSS and Total Station. Hydraulic structure sketches and field photos have been included in Appendix A.

Callon Dietz completed a bathymetric survey of the reservoir between November 4, 2024 and November 6, 2024. The survey was completed by boat using a Trimble R12 GPS paired with a Sonarmite depth finder. The sounding was collected in requested locations around the Deer Creek reservoir with medium density for majority of the lake and high density close to the spillway. Cross sections were collected at 7 locations in shallower areas to the north using a combination of the Sonarmite and traditional GPS and survey pole methods. The metadata provided by Callon Dietz and the location of surveyed points in the reservoir has been included in Appendix A.

2.0 Hydrologic Assessment

2.1 Introduction

The purpose of the hydrology study is to determine the peak flows at key locations within the Deer Creek watershed and Big Creek watershed for a variety of flood

scenarios. The results of the hydrology study are the key inputs into the hydraulic model. The dam is within LPRCA's jurisdiction; therefore, the Regulatory event is the 100-year (1% AEP) flood.

The hydrology study involved the development of a new hydrologic model using HEC-HMS (Version 4.12). The hydrologic model development included the following tasks:

- Delineation of the Deer Creek watershed.
- Characterization of the Deer Creek watershed.
- Characterization of channel routing elements.
- Development rainfall hyetographs for the various storm events being considered.

The following sections describe the background information used in the analyses, present the results of the hydrology study, and identify the peak flows to be used in the hydraulic model.

A Single Station Frequency Analysis (SSFA) was performed on the Big Creek near Walsingham gauge to determine flood flows upstream and downstream of the confluence of Deer Creek and Big Creek. LPRCA's hydrology model was used to calculate the Probable Maximum Flood (PMF) for upstream of the confluence of Deer Creek and Big Creek.

2.2 Available Data

2.2.1 Precipitation Data and Design Storms

2.2.1.1 Annual Exceedance Probability Storm Event and Distribution

As per the Technical Guide - River and Stream Systems: Flood Hazard Limit (MNR, 2002), Figure B-1, Deer Creek and Big Creek are within Zone 1, meaning that the Regional storm event is Hurricane Hazel. The Regulatory event for LPRCA's jurisdiction is the 100-year flood (1% AEP).

The Annual Exceedance Probability (AEP) total rainfall volumes for the Deer Creek watershed are based on the Intensity-Duration-Frequency (IDF) parameters for Deer Creek Dam location in the Norfolk County and come from the MTO IDF Curve Lookup Tool. The IDF parameters and additional information regarding MTO IDF Curve Lookup Tool data sources and development are provided in Appendix B.

The total rainfall volumes were distributed based on various synthetic storm distributions for use within the hydrologic model. The most common synthetic storm distributions for the purposes of dam breach analysis are the 6-hour, 12-hour, and 24-hour SCS Type II storm distributions. The 24-hour SCS Type II storm was used in the hydrologic model to estimate the peak flows for the Deer Creek Dam and downstream drainage areas. The rationale for choosing the 24-hour duration is discussed in Section 2.3.10.

2.2.1.2 Probable Maximum Precipitation

Several methods for calculating probable maximum precipitation (PMP) volume have been developed for Ontario by the MNR; however, no method has been officially adopted or recommended. Based on direction from the MNR, Wills reviewed the PMP rainfall volumes from four methods to develop a range of possible values and can be found in Table 1.

Table 1 - PMP Rainfall Volumes

Storm Duration (hour)	OMNR 1977 (mm)	OMNR 2004 (mm)	OMNR 2006 (mm)	MNRF 2014 (mm)	Average (mm)
6	409	405	550	297	415
12	427	420	570	324	435
24	442	440	596	378	464

Wills selected the OMNR 2004 rainfall volume for this study as it aligns closely with the average volume of the methods reviewed. Wills used the 6-hour and the 12-hour duration PMP storm distributions which are recommended for watersheds less than 500 square miles (1295 km^2) (OMNR, 2004). The storm distributions used by Wills can be found in Appendix B.

2.2.2 Land Cover and Soils Data

Soils data was obtained from the Soil Survey Complex GIS Data available on Ontario GeoHub for southern Ontario. In 2015, the Ontario Ministry of Agriculture, Food and Rural Affairs and Agriculture (OMAFRA) and Agri-Food Canada, in cooperation with the MNR, compiled a geo-spatial soils database for southern Ontario. The database consolidated the existing digital soil data, mapped on a county basis, into a digitally stitched and standardized product. The GIS data indicate that the Deer Creek watershed consists of mostly fine sand, loamy sand, sandy loam, and loam. The soils map is included in Figure 2.

The data used to define the land cover within the Deer Creek watershed is the LPRCA's Land Cover data developed by 4DM in 2017 which was provided by LPRCA. The land cover data is a landscape level inventory of natural, rural, and urban areas within LPRCA's jurisdiction with a 20 m resolution. There are 15 landcover types in the land cover and these were consolidated by Wills to land cover types with similar hydrologic parameters for the purposes of this study. For example, coniferous treed, deciduous treed, and mixed treed were grouped as "forest". The Deer Creek Dam and downstream drainage areas are primarily comprised of mostly forest, and agricultural land. The land cover map is included in Figure 3.

2.2.3 Watercourse and Waterbody Data

Ontario Hydro Network (OHN) data were downloaded from the MNR's GeoHub database. The data used in this study included watercourses and waterbodies. LPRCA also provided a watercourse layer for use in this study.

2.2.4 LiDAR Digital Terrain Model and Catchment Delineation

The primary source of topographic data for the hydrology study was the LiDAR DTM that was provided through the MNR's GeoHub. The LiDAR for this project was collected as part of the 2016-2018 OMAFRA Lake Erie Project. The DTM represents the bare earth surface and was generated from the classified LiDAR point cloud data. Published vertical accuracy for the Eastern Ontario Project is 6.9 cm in non-vegetated conditions and 18.3 cm in vegetated conditions (MNR, 2025). The native resolution of the DTM was 50 cm. The 50 cm DTM was used for determining characteristics cross sections and slope data. The DTM was resampled to a 5 m resolution and used to delineate the Deer Creek Dam and downstream drainage areas, as well as the longest flow paths within HEC-HMS. The drainage area boundaries delineated by Wills were compared with drainage boundaries from the 2006 LPRCA hydrology study and the 2024 LPRCA hydrology study; as well as what Ontario Watershed Information Tool (OWIT). The drainage area to the dam estimated by Wills was 1.77 km^2 less then the drainage area used in previous hydrologic studies (11% difference). The largest differences were in the headwater areas. These differences were investigated using the 50 cm resolution LiDAR and show Wills delineation to be accurate. The drainage boundaries delineated by Wills generally conformed to those calculated using OWIT. A map of the subbasins can be found in Figure 4.

2.2.5 Recorded Hydrometric Data

There are no flow gauges located within the Deer Creek watershed; therefore, no discharge data was available for use for statistical analysis or model calibration for this watershed.

The Venison Creek near Walsingham gauge is located approximately 5.65 km from the Deer Creek Dam. The Venison Creek watershed neighbours the Deer Creek watershed to the west and has similar land cover and soil characteristics but has a significantly larger drainage area (68.4 km^2 vs 15.13 km^2). The results from a single station frequency analysis for this gauge was transferred to the Deer Creek Dam and were used as a point of comparison for the modeled return period flood flows for Deer Creek.

The Big Creek near Walsingham flow gauge is located 1.4 km downstream of the confluence of Deer Creek and Big Creek. This gauge was used for the purposes of estimating flood flows in Big Creek for hydraulic modeling. The Co-ordinates, Subbasin Areas, and the Period of Record for the flow gauges are presented in Table 2.

Table 2 – Recorded Hydrometric Data

River	Station ID	Co-ordinates	Subbasin Area (km^2)	Period of Record (years)
Venison Creek near Walsingham	02GC021	42° 39' 12" N 80° 32' 54" W	68.4	49
Big Creek near Walsingham	02GC007	42° 41' 08" N 80° 32' 18" W	561	71

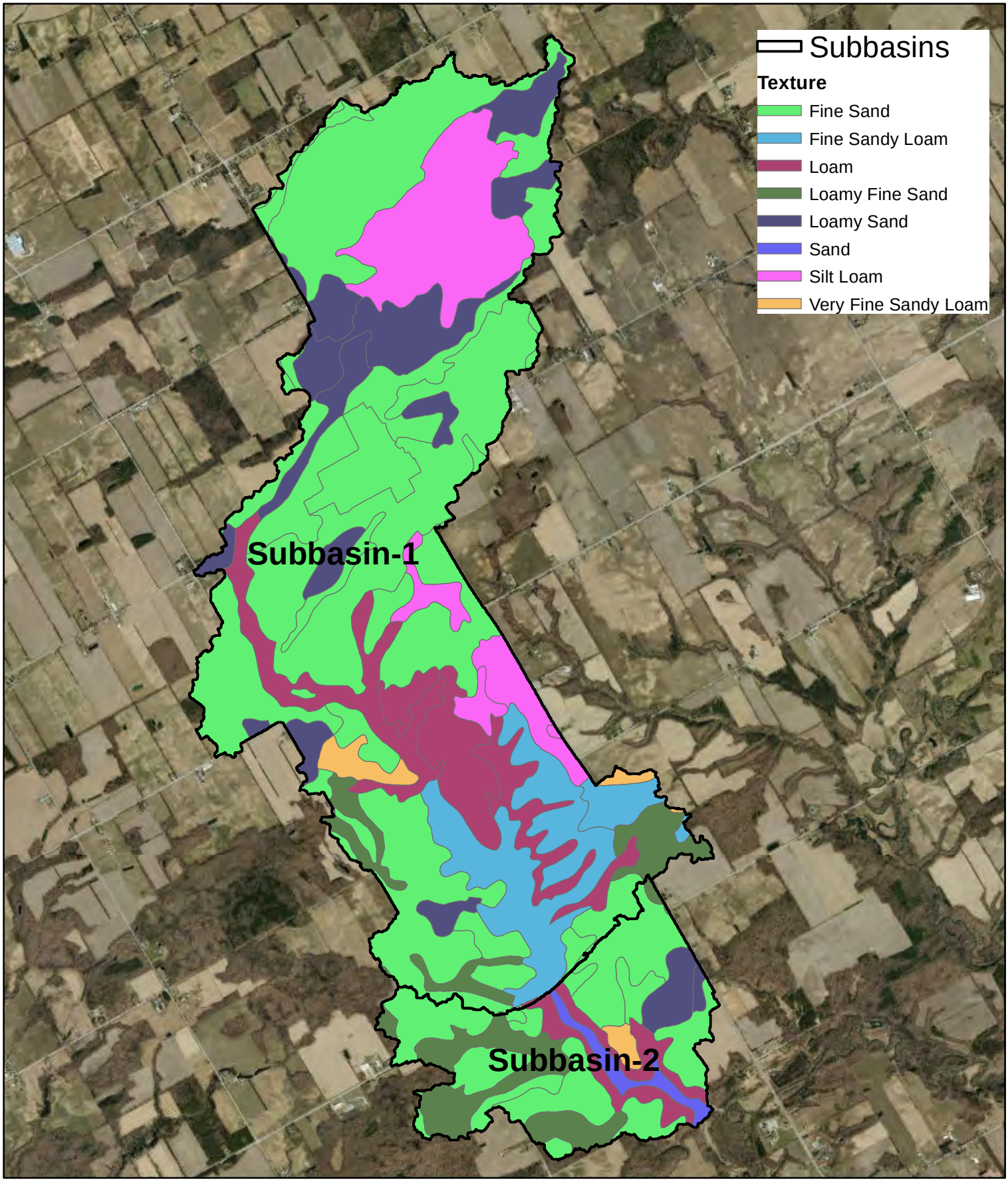


Figure 2
Soil Textures for
Deer Creek

Data Sources:
Soil Survey Complex, GEOHUB 2024
SWOOP 2010, GEOHUB 2024

NAD 1983 CSRS UTM Zone 17N
1:40,000
Meters
0 500 1,000

Drawn By:	MC
Checked By:	DG
Map Date:	01/28/25
Project Number:	5684
Map File Number	24-5684

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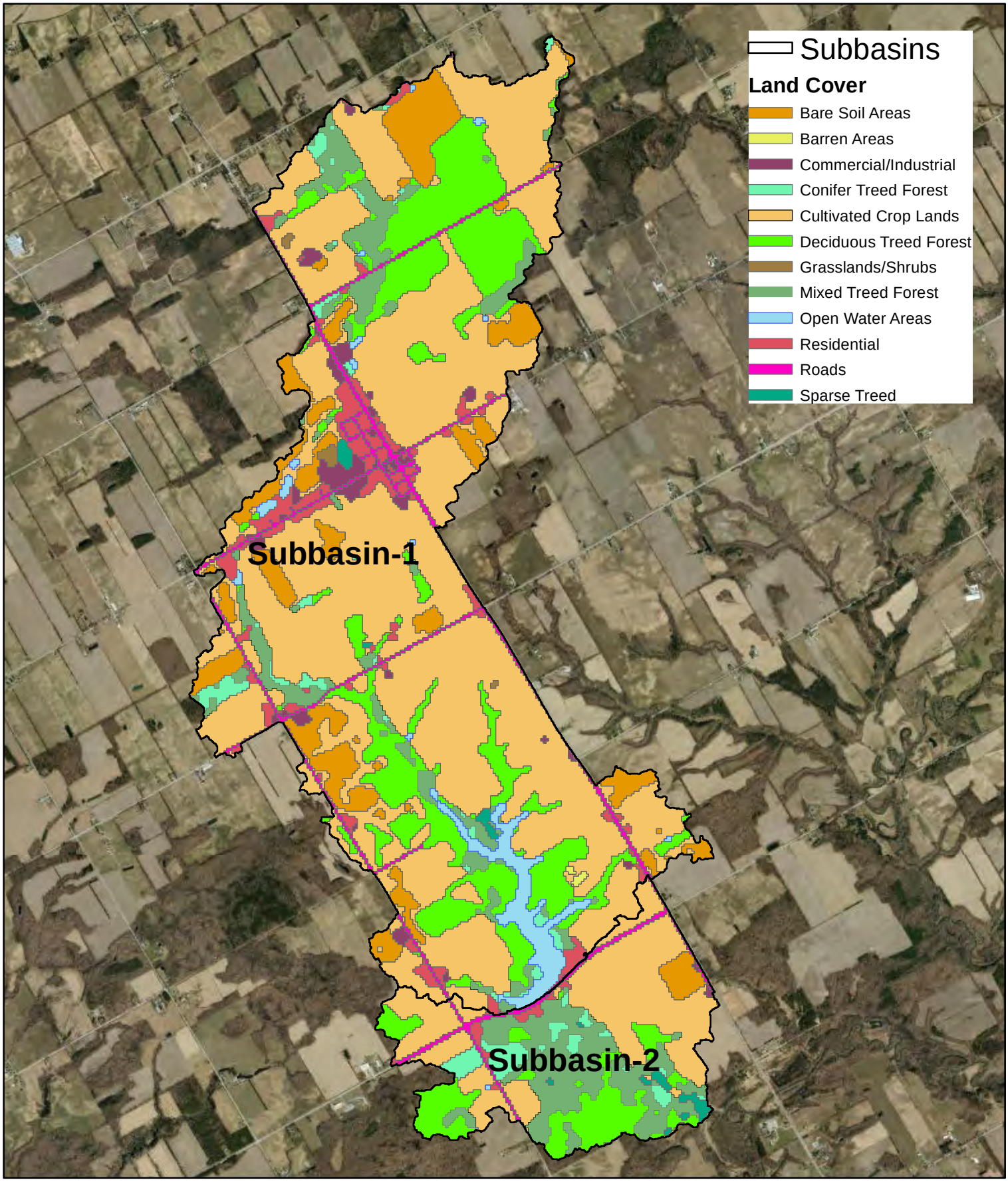


Figure 3
Land Cover for
Deer Creek

Data Sources:
LPRCA Land Cover, 4DM/LPRCA
SWOOP 2010, GEOHUB 2024

NAD 1983 CSRS UTM Zone 17N
1:40,000
Meters
0 500 1,000

Drawn By:	MC
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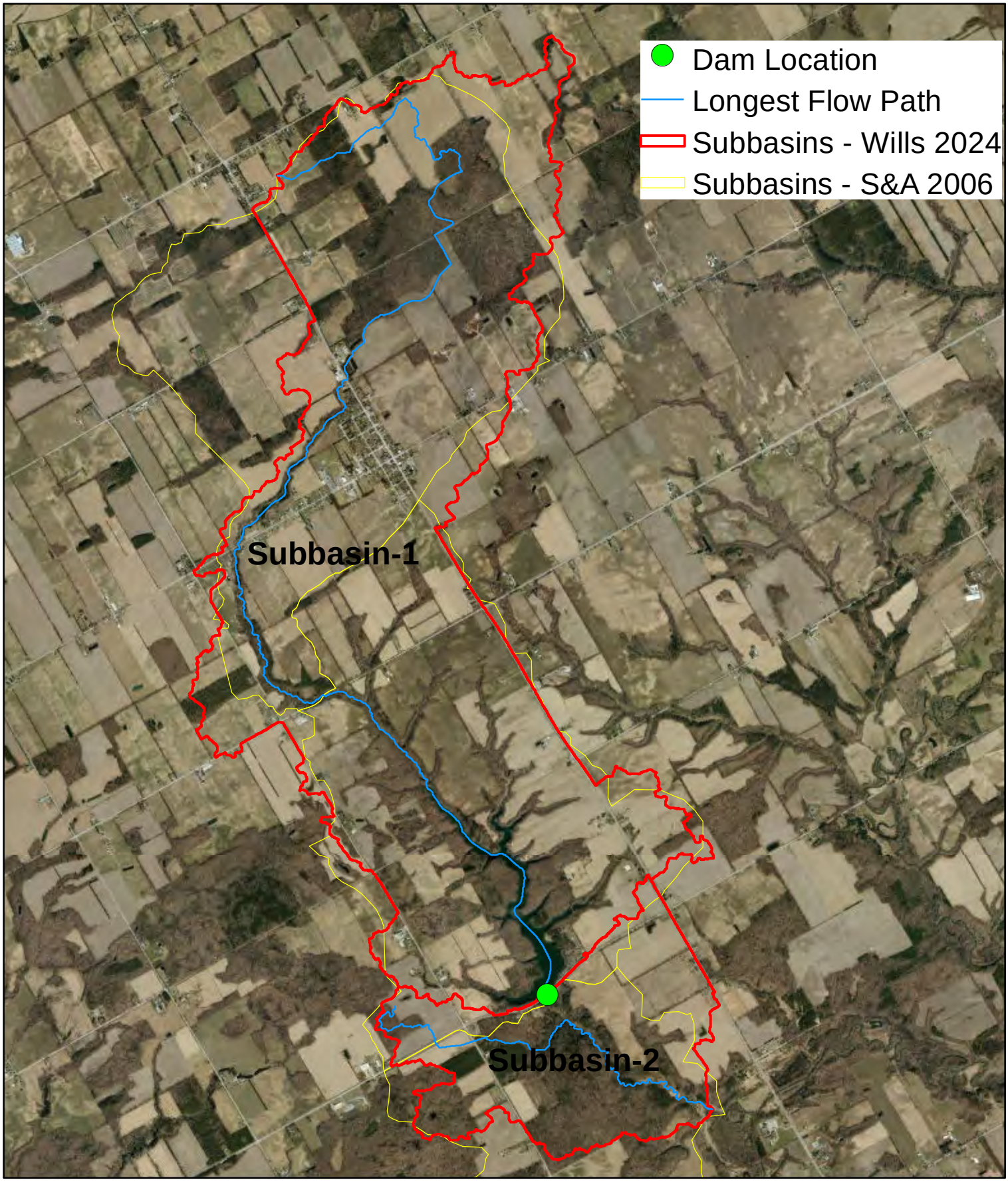


Figure 4
Longest Flow Path and Subbasin for Deer Creek

Data Sources:
SWOOP 2010, GEOHUB 2024
LPRCA Hydrology Study, Matrix Solutions, 2024

NAD 1983 CSRS UTM Zone 17N
1:40,000
Meters
0 500 1,000

Drawn By:	MC
Checked By:	DG
Map Date:	01/28/25
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2.2.6 Existing Hydrologic Model

The LPRCA provided a detailed HEC-HMS model that was produced by Matrix Solutions in 2024 as part of the LPRCA Riverine Flood Hydrology Study. This model covers the entirety of the LPRCA jurisdiction including the Deer Creek and Big Creek study areas. Calibration and verification of the model was completed by Matrix Solutions for Big Creek using the five flow gauges in the watershed. These calibration events included rainfall only, and rain on snow events. Wills used this model to calculate the PMF for Big Creek as well as extract parameters for linear reservoir baseflow for use in Wills' hydrologic model for Deer Creek.

2.3 Hydrologic Model Development

2.3.1 Model Selection

The HEC-HMS (Version 4.12) hydrologic model was selected as the preferred hydrologic model to be used for this project. HEC-HMS is a free hydrologic modeling software developed and maintained by the U.S. Army Corps of Engineers' (USACE) Hydrologic Engineering Centre (HEC) with a long history of use in Canada and internationally. The software can simulate the complete hydrologic process of watersheds including rainfall, snowmelt, evapotranspiration, and soil moisture accounting in lumped, semi-lumped or gridded models. HEC-HMS is capable of single event or continuous modeling simulations and allows the user significant control of modeling approaches for each hydrologic process. Analysis tools built into HEC-HMS include GIS, model optimization, forecasting streamflow, assessing model uncertainty, erosion and sediment transport, and water quality. HEC-HMS is well integrated with other HEC software products. The software is suitable for many applications including watershed studies, flood hazard mapping, dam safety reviews, hydraulic structure design, and flood forecasting exercises.

The following information is required to calculate the input parameters for HEC-HMS to compute hydrographs, peak flows, and routing information:

- Physical characteristics of subbasins to compute infiltration and runoff, which includes topographic information, soil drainage features, and land cover.
- Physical characteristics of the watercourses for reach routing, which includes slope, length, geometry, and reach roughness.
- Meteorological information such as rainfall, and when modeling snowmelt, temperature and snow water equivalent, to calculate hydrographs and peak flows at points of interests.

A single event, semi lumped modeling approach was chosen for computing peak flows and hydrographs for this study based on the available data and modeling objectives.

2.3.2 Subbasin Delineation and Hydrologic Elements

The Deer Creek Dam and the downstream drainage area were subdivided into 2 sub-basins, 1 routing reach, and 1 junction based on the various tributaries and the

anticipated flow change locations in the hydraulic model. The subbasins were delineated using the 5 m resolution LiDAR DTM and enforcing hydrology in HEC-HMS using the watercourse layer. The sub-catchment boundaries can be found in Figure 4. Subbasin drainage areas ranged from 3.20 km^2 to 15.13 km^2 and the routing reach length was 2.37 km. The model layout and schematic can be found in Figure 5.

2.3.3 Catchment Characterization

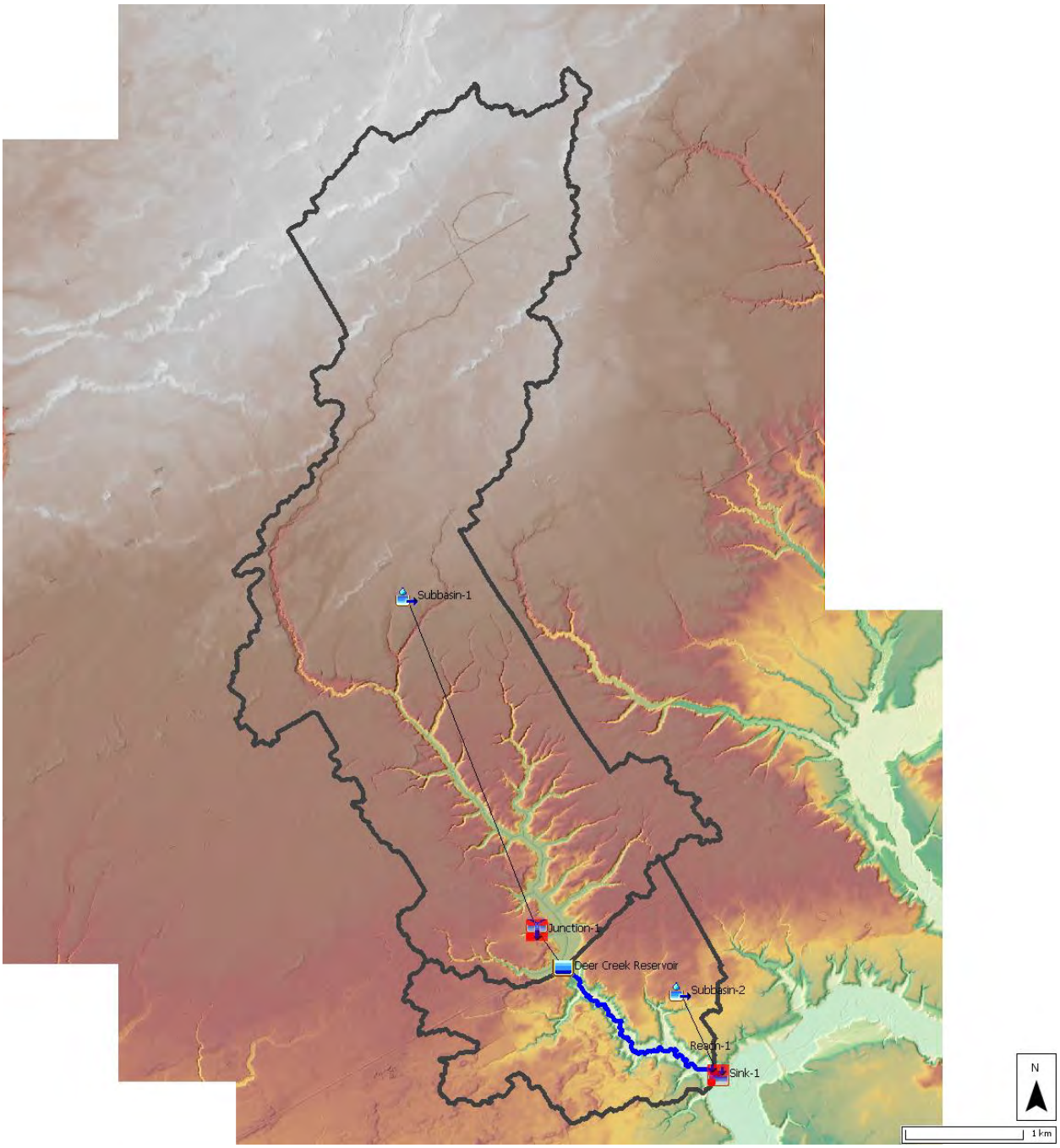
Deer Creek is a tributary of Big Creek and generally flows in a southern direction. The headwaters originate approximately 7.6 km north of the Deer Creek Dam. The landcover is mostly agricultural land and forest. The Deer Creek watershed was delineated using the LiDAR DTM and HEC-HMS. The drainage area to the dam is 15.13 km^2 and the drainage area to the confluence of Deer Creek with Big Creek is 18.33 km^2. The longest flow path to the dam is 12.77 km and the longest flow path from the most upstream point in the watershed to the confluence with Big Creek is 15.15 km. Deer Creek's watershed topography can be described as gently rolling with very deep and defined valleys downstream of the community of Langton. The total basin relief to the dam is 41.3 m with a basin slope of 5.6%. The longest flow path slope to the dam is 0.3%. The elongation ratio is an indicator of shape of the basin reported by HEC-HMS. The subbasins have elongation ratios of less than 0.7, which indicates that they are elongated rather than circular in shape.

The hydrologic characterization parameters for each sub-catchment are provided in Table 3. These hydrologic parameters were used as the initial parameters or used to calculate other parameters in the model.

Table 3 – Subbasin Characteristics

Catchment ID	Area (km2)	Basin Slope (m/m)	Basin Relief (m)	Longest Flow Path (km)	Longest Flow Path Slope (m/m)	10-85 Flow Path Length (km)	10-85 Flow Path Slope (m/m)	Elongation Ratio
Subbasin1	15.13	0.056	41.3	12.77	0.003	9.58	0.003	0.344
Subbasin2	3.20	0.088	35.1	4.44	0.007	3.33	0.009	0.455

Figure 5 – HEC-HMS Model Schematic



2.3.4 Initial Abstraction

At the initial stage of rainfall, a certain amount of water is intercepted by vegetation before it can reach the ground and infiltrate. When water does it make through the vegetation it must fill depressions in the landscape before it can produce runoff. These collectively are called initial abstractions. HEC-HMS requires the user to define initial canopy storage, canopy storage, initial depression storage, and depression storage to account for initial abstractions. Initial abstractions were calculated for each catchment based on an area weighted average of typical values for land cover types in each catchment using GIS in accordance with the Technical Guidelines for Flood Hazard Mapping (Environmental Water Resources Group Ltd. (EWRG), March 2017). It was assumed that all canopy and depression storage in the subbasins were available for each event modeled. The input parameters for initial abstractions can be found in Table 4. The standard parameters and calculations are provided in Appendix B.

Table 4 – Canopy and Depression Storage

Catchment ID	Initial Canopy Storage Used	Max Canopy Storage (mm)	Initial Depression Storage Used	Max Depression Storage (mm)	Total Abstraction (mm)
Subbasin1	0	2.1	0	7.8	9.9
Subbasin2	0	2.4	0	8.7	11.1

2.3.5 Infiltration Loss Method

The Green and Ampt method was chosen as the loss method in the model to account for infiltration and compute rainfall excess. The Green and Ampt Model was chosen as it has a long history of use in Southern Ontario, is physically based, and parameter estimation can be done with soil texture information or measured in the field. The Green and Ampt is based on Darcy's Law and is a simplification of the Richard's equation. As water content at the soil surface increases, it is pulled through the soil column by suction at the wetting front in a piston like displacement. The parameters required in HEC-HMS for the Green and Ampt model are saturated hydraulic conductivity, initial water content, porosity, suction head at the wetting front, and percent impervious.

Saturated hydraulic conductivity, field capacity, wilting point, porosity, and suction head at the wetting front were calculated for each catchment based on an area weighted average of typical values from soil textures in each catchment using GIS. Typical values were taken from Rawl et al. (1983), EWRG (2017), and Schroeter & Associates (2006). The initial water content for each modeling scenario was assumed to be at field capacity for all events modeled in this study. The percent impervious was calculated from land cover for each catchment. It was assumed that land cover designated as roads, and open water are 100% impervious. Medium and commercial/industrial land cover was assumed to be 50% impervious, and 70% impervious, respectfully. The Green and Ampt parameters used in the model can found in Table 5. The standard parameters and calculations are provided in Appendix B.

Table 5 – Loss Method - Green and Ampt

Catchment ID	Effective Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Suction at the Wetting Front (Average) (mm)	Saturated Hydraulic Conductivity (mm/h)	% Impervious
Subbasin1	0.42	0.14	0.06	140.7	64.3	8.5
Subbasin2	0.42	0.10	0.04	129.2	82.2	4.4

2.3.6 Runoff Transform – Unit Hydrograph

The SCS Unit Hydrograph method was used in the model to convert excess rainfall to runoff. The SCS Unit Hydrograph method is based upon an average of unit hydrographs derived from gauged rainfall and runoff for a large number of small agricultural watersheds in the United States (USACE, 2023). This method uses a dimensionless, curvilinear unit hydrograph to route excess rainfall to the subbasin outlet. HEC-HMS requires the peaking rate factor and lag time as input parameters for each subbasin.

The peaking rate factor controls the volume of water on the rising and recession limbs (NOAA, 2005). Choosing a peaking rate factor is based on land cover and topography. The hydrograph peaking rate factors and their associated limb ratios can be found in Table 6.

Table 6 – Hydrograph Peaking Factors & Recession Limb Ratios (Wanielista, et al 1997)

Description of Land Cover and Topography	Peaking Factor	Limb Ratio (Recession to Rising)
Urban areas; steep slopes	575	1.25
Typical SCS	484	1.67
Mixed urban/rural	400	2.25
Rural, rolling hills	300	3.33
Rural, slight slopes	200	5.50
Rural, very flat	100	12.00

The peaking rate factors for the subbasins in the model were chosen based on topography, land cover, and engineering judgement.

Lag is the time from the centre of mass of excess rainfall to the time to peak of a unit hydrograph as described in the Natural Engineering Handbook Part 630 Chapter 16 – Hydrographs (National Resources Conservation Service (NRCS), 2007). Lag Time can be related to most watersheds with time of concentration, T_c, using the following equation (NRCS, 2007):

$$t_p = 0.6 * T_C$$

The time of concentration is used to estimate the peak discharge from a watershed, and it depends on the slope, watershed characteristics, and flow path length. Time of concentration is the longest time required for runoff from the most distant point in the watershed to travel to the outlet. Many empirical equations are available to estimate the time of concentration. In this study, the TR-55 method has been used to calculate the time of concentration. The TR-55 Method computes travel times for sheet flow, shallow concentrated flow, and channel flow along the longest flow path to calculate the time of concentration. The equation can be found below.

$$T_c = t_{sheet} + t_{shallow\ contrated} + t_{channel}$$

The detailed time of concentration calculations and equations for each component can be found in Appendix B. The resulting peaking rate factor, time of concentration, and lag time for each catchment are presented in Table 7.

Table 7 – Runoff Transform – Unit Hydrograph

Catchment ID	Peaking Rate Factor	Time of Concentration (hrs)	Lag Time (Hrs)	Time of Concentration (mins)	Lag Time (min)
Subbasin1	300	6.43	3.86	385.9	231.6
Subbasin1	300	3.99	2.40	239.6	143.8

2.3.7 Baseflow and Interflow

In large rural watersheds interflow generally makes up a large proportion of the total volume on the receding limb of a hydrograph. Wills used the Linear Reservoir baseflow method within the HEC-HMS model to account for this phenomenon. Wills used a two-layer linear reservoir for baseflow to represent interflow and shallow groundwater processes. Water from infiltration at the surface is routed through the upper layer with quicker interflow, while the lower layer attempts to model the slower shallow groundwater flow. Water enters the upper layer, and its outflow becomes the inflow for the lower layer, mimicking the movement of water from surface layers to deeper subsurface layers. This method captures the delayed response of watersheds to precipitation events, providing a more accurate simulation of baseflow contributions to streamflow over time. Additionally, it is the only method in HEC-HMS that conserves volume. HEC-HMS requires initial baseflow, baseflow fraction, and groundwater coefficients for each linear reservoir layer as inputs.

The groundwater coefficients are related to the Clark Watershed Storage Coefficients (CSC). The HEC-HMS manual recommends starting groundwater coefficients 3 times larger than the CSC for the first layer and 10 times larger for the second layer. Wills used the following equation in order to estimate the CSC (Sabol, 1988).

$$R = \frac{T_c}{1.46 - 0.0867 * \frac{L^2}{A}}$$

Where R is the CSC in hours, T_c is the time of concertation, L is the longest flow path and A is the subbasin area. Wills calculated the groundwater coefficients for both layers for each subbasin using this method and they can be found in Table 7.

Table 8 – Linear Reservoir Groundwater Coefficient Calculations

Subbasin	Time of Concertation (Hours)	Area (km^2)	Longest Flow Path (km)	Clark Storage Coefficient (Hours)	3*Clark Storage Coefficient (Hours)	10*Clark Storage Coefficient (Hours)
Subbasin 1	6.43	15.13	12.77	12.2	36.7	122.3
Subbasin 2	3.99	3.20	4.44	4.3	12.9	43.1

The groundwater fraction between each linear reservoir layer was initially 0.22 for the first layer, and zero for the second layer. This is the same as the LPRCA's existing HEC-HMS model. Wills used the average of the base flow field measurements 0.16 m^3/s or unit base flow of 0.01 m^3/s per km^2 of drainage area from the original dam design report.

2.3.8 Reach Routing

The Muskingum-Cunge method was chosen as the channel routing method in the model. The Muskingum-Cunge method is based on the solution of the continuity and momentum equation. This method was chosen as it has a long history of use in flood mapping projects in Southern Ontario and has measurable physically based parameters. The Muskingum-Cunge method in HEC-HMS requires reach length, slope of the energy grade line (estimated as channel bed slope), wave celerity, characteristic cross section, and Manning's roughness.

The reach length, slope, cross section, and invert were calculated using the LiDAR DTM in HEC-RAS and HEC-HMS. The Manning's roughness in the channel and left and right overbanks were chosen based on typical values of the land cover at the characteristic cross section. Where the land cover was determined to be forested or wetland, a Manning's roughness value of 0.08 was used. A main channel Manning's roughness of 0.035 was used, which is typical for Ontario (EWRG, 2017). Wave Celerity was estimated to be 1.5 m/s, which is adequate for most applications (USACE, 2023). The reach routing parameters used in the model can be found in Table 9.

Table 9 – Reach Routing Parameters

Reach	Length (m)	Slope (m/m)	**Manning's** n	Index Celerity (m/s)	Shape	Left Overbank Mannings Roughness	Right Overbank Mannings Roughness	Invert
Reach1	2.37	0.007	0.035	1.5	8 Point	0.08	0.08	183.79

2.3.9 Design Storm Distribution and Duration

The primary objective of the hydrology model to determine the return period (AEP) flood events for the Deer Creek Dam and the downstream drainage area. There are no flow gauges in the Deer Creek watershed, and therefore the return period (AEP) flood events were calculated using synthetic storms.

Typically, synthetic design storms for floodplain mapping in Ontario are represented by rain on snow (spring) events or intense thunderstorms (summer). Choosing a design storm approach is dependent on watershed characteristics such as land cover, time of concentration, watershed size, and seasonality of when most annual maximum peak flow rates occur (EWRG, 2017). Given the relatively small size of the watershed, and the time of concentration, it was assumed that summer rainfall events will be sufficient for the purposes of this study.

2.3.9.1 Summer Rainfall

Wills developed synthetic storm events with durations of 6, 12, and 24 hours using the SCS Type 2 distribution. The 24-hour duration resulted in the highest volume and peak flows, and therefore, only this data is provided. The MTO IDF curves do not have the 1000-year rainfall volume; therefore, it had to be estimated by Wills. The 1000-year rainfall volume was estimated by plotting the 2-year to through 100-year rainfall volumes and creating a logarithmic trend line. The logarithmic equation was then used to calculate the 1000-year event and can be found in Figure 6. The rainfall volumes for summer return periods can be found in Table 10.

Figure 6 – 1000-Year Return Period Estimation from 24 Hour IDF Volumes

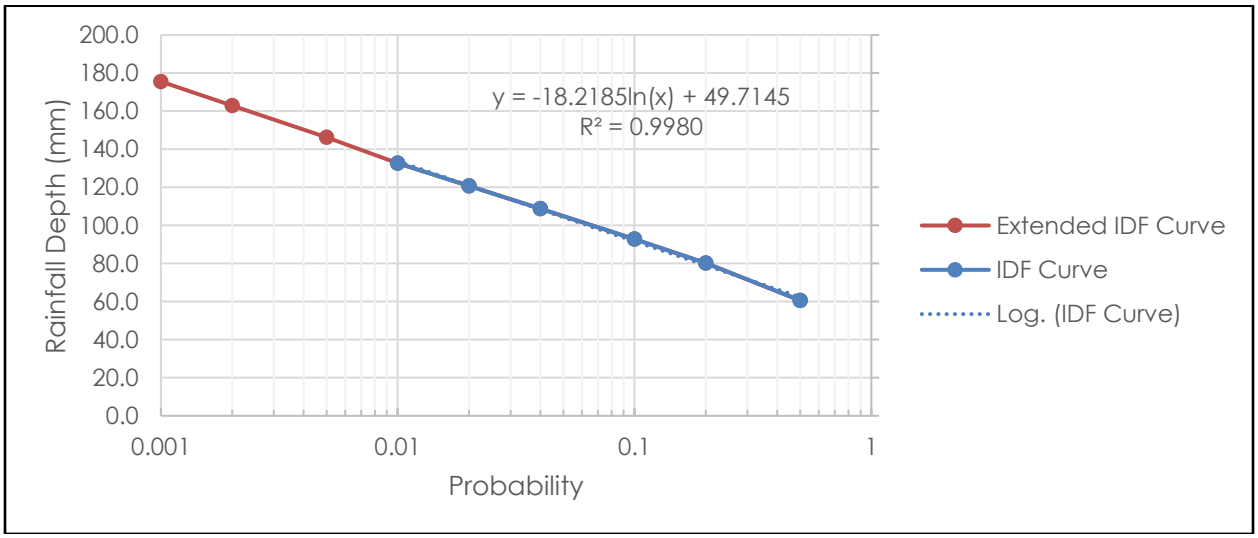


Table 10 – 24 Hour Summer Rainfall Volumes

Probability	Return Period	24 Hour Rainfall (mm)
0.5	2-year	60.6
0.2	5-year	80.2
0.1	10-year	92.9
0.04	25-year	108.8
0.02	50-year	120.8
0.01	100-year	132.7
0.001	1000-year	175.6

The 24-hour SCS design storm for all return period flows and the Regional storm run in the model have been provided in Appendix B. The 100-year (1% AEP) 24-hour SCS total rainfall volume for Deer Creek is 132.7 mm with 35.6 mm falling during the most intense 15 minutes of the event.

2.3.10 Rainfall and Computation Time Step

The computation time step and rainfall time step were considered based on watershed characteristics. The rainfall time step for modeling should be less than the minimum time of concentration of any catchment (EWRG, 2017). Subbasin 2 has shortest time of concentration of 3.99 hours within the catchments and therefore 15 min rainfall time steps were considered appropriate for the purposes of this study. A computation time step of 15 minutes was used for the purposes of this study.

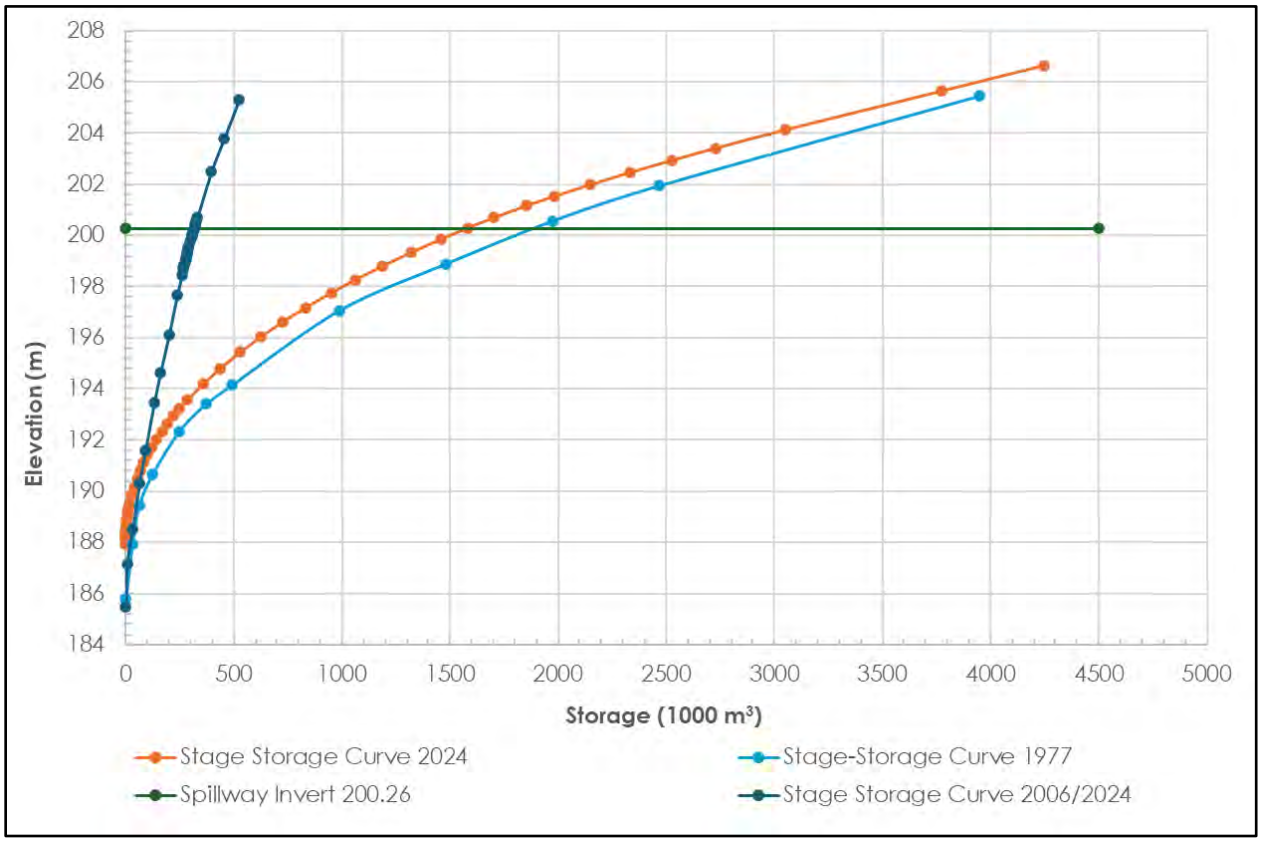
2.3.11 Areal Reduction Factor

The equivalent circle diameter method was used to assess the need for application of areal reduction factors to the design storms. The point of interest for hydraulic modeling, is the Deer Creek Dam and is approximately 7.6 km as the "crow flies" from the most remote portion of the upstream watershed. This results in an equivalent circular area of 45.7 km^2. The aerial reduction factor was determined using the areal reduction factor table for Hurricane Hazel (MNR, 2002). An area of 45.7 km^2 resulted in an aerial reduction factor of approximately 0.992, which was applied for all return period events using the 24-hour duration storm.

2.3.12 Reservoir Routing

Wills developed a stage-storage curve for the Deer Creek Reservoir using the LiDAR DTM and the Bathymetric Survey Data with GIS and HEC-RAS. The stage storage curves from 1977 design report, LPRCA's existing HEC-HMS model, and the curve developed by Wills can be found in Figure 7.

Figure 7 – Stage-Storage Curve for Deer Creek Dam



The 2024 stage-storage curve (orange), developed by Wills, indicates that for the same amount of storage, the elevation is generally higher than the 1977 curve (blue). This suggests that the reservoir may have changed over time, possibly due to sediment accumulation. The general shape of these curves are similar.

The 2006/2024 curve used in the LPRCA hydrology studies is significantly different than the original design curve and that calculated by Wills. The curve shows the reservoir to have significantly less storage than the surveyed data.

2.4 Hydrologic Model Results

The HEC-HMS model was run for the 2-year (50% AEP), 5-year (20% AEP), 10-year (10% AEP), 25-year (4% AEP), 50-year (2% AEP), 100-year (1% AEP), and 1000-year (0.1% AEP) SCS 24-hour storms; as well as the Hurricane Hazel, and PMP events using the existing condition parameter set as described in Section 2.3.3. The 6-hour and 12-hour storm events were also run but resulted in lower peak flows and volumes. As a result, the 24-hour storm event was selected as the basis for further analysis. The peak flows for the 25-year (4% AEP), 100-year (1% AEP), and 1000-year (0.1% AEP), PMP, and Hurricane Hazel for each hydrologic element in the Deer Creek Dam and downstream drainage areas are provided in Table 11 for the summer storm events. The results for all return period (AEP) storms can be found in Appendix B.

Table 11 – Summer Peak Flows

Hydrologic Element	Drainage Area (km^2)	Peak Discharge (m^3/s)					
		25-Year	100-Year	1000-year (Modeled)	PMF 6-Hour	PMF 12-Hour	Hurricane Hazel
Subbasin1	15.13	5.3	7.1	14.9	70.7	73.9	11.8
Deer Creek Reservoir	15.13	4.05	5.50	11.65	59.31	62.57	9.96
Reach-1	15.13	4.05	5.49	11.64	59.21	62.49	9.95
Subbasin2	3.20	1.51	1.84	3.50	19.59	20.35	3.31
Sink-1	18.33	4.85	6.53	13.31	67.81	71.79	11.88

2.5 Hydrologic Model Validation

2.5.1 Regional Index Flows

Index Flood Method 1 – Matrix Solutions 2024

Matrix Solutions (2024) developed regional flood flow estimates for small ungauged sandy watersheds using linear regression based on statistical analysis of data from several gauges in the region, focusing on areas with similar watershed and meteorological characteristics. The linear equation for flood flow calculations for all annual exceedance probabilities in the small sand plain/western watersheds is expressed as:

$$Y = A(Drainage\ Area)$$

where Y is the index flood (in m^3/s), the drainage area is in km^2, A is the coefficient that corresponds to the return period flood below.

Index	2-year	5-year	10-year	25-year	50-year	100-year	1000-year
A	0.1614	0.2461	0.3057	0.3847	0.4459	0.509	0.736

A comparison of the modelled flows with the Matrix Solutions Method is provided in Table 13.

Index Flood Method 2 – Moin and Shaw 1985

Moin and Shaw (1985) developed regional flood flow estimates based on regression analyses of observed index floods (e.g. 2 year) for the whole province. For their Region 6, which contains all the watersheds within the LPRCA jurisdiction, the index flood is computed as:

$$Q2 = 0.41(Drainage\ Area)^{0.806}$$

where Q2 is the index flood (in m^3/s), the drainage area is in km^2, C=0.41 and N=0.806. The remaining return period flood flows (5 through 100 year) are taken as ratios of the Index Flood. The applicable ratios for Region 6 are given below.

Index	Q2	Q5	Q10	Q25	Q50	Q100
Ratio	1.0	1.4	1.77	2.27	2.72	3.18

A comparison of the modelled flows with the Moin and Shaw Method is provided in Table 13.

Index Flood Method 3 – Cumming Cockburn Ltd. 2000

In 2000, Cumming Cockburn Ltd. (CCL) was commissioned by the Ontario Ministry of Natural Resources to update the regional analysis undertaken by Moin and Shaw (1985, 1986). For their Region 1, which includes all of southwestern Ontario from Windsor to Oshawa and north to Lake Simcoe, the index flood flow is calculated as:

$$Q2 = 0.757(Drainage\ Area)^{0.7649}$$

where Q2 is the index flood (in m^3/s), the drainage area is in km^2. The remaining return period flood flows (5 through 100 year) are taken as ratios of the Index Flood. The applicable ratios for CCL's Region 1 are given below.

Index	Q2	Q5	Q10	Q25	Q50	Q100
Ratio	1.0	1.464	1.757	2.136	2.393	2.664

A comparison of the modelled flows with the Cumming Cockburn Method is provided in Table 13.

2.5.2 Single Station Frequency Analysis Station Transfer

The single station frequency analysis (SSFA) for both Venison Creek near Walsingham and Big Creek near Walsingham was updated as part of the LPRCA Riverine Flood Hydrology Study (Matrix Solutions, 2024). Wills transposed the results of this SSFA for Venison Creek near Walsingham to the Deer Creek Dam to evaluate the results of the hydrologic model. Wills transposed the results of the SSFA for Big Creek near Walsingham to above the Deer Creek and Big Creek confluence to determine return period floods for the hydraulic model. The transposition of the data was calculated using the following formula:

$$Q2 = Q1\left(\frac{A2}{A1}\right)^{0.75}$$

Where Q2 is the transposed flows, Q1 is the calculated AEP flows, A1 is the drainage area at the gauge, and A2 is the drainage area at the point of interest. The results from the flow transposition can be found in Table 12.

Table 12 – Single Station Frequency Analysis Results

Return Period (years)	Probability	Big Creek near Walsingham SSFA (561.2 km²) (m³/s)	Big Creek Transposed Flow to above Confluence (537.6 km²) (m³/s)	Venison Creek near Walsingham SSFA (68.4 km²) (m³/s)	Venison Creek Transposed Flow to Deer Creek Dam (15.13 km²) (m³/s)
2	0.5	38.4	37.2	6.2	2.0
5	0.2	61.7	59.7	8.8	2.8
10	0.1	80.7	78.1	10.7	3.5
25	0.04	108.5	105.1	13.1	4.2
50	0.02	132.1	127.9	15.0	4.8
100	0.01	158.0	153.0	16.9	5.5
1000	0.001	263.9	255.5	23.7	7.6

2.5.3 Flood Flow Comparison

Wills utilized several index flow methods, and gauge transposition from the Venison Creek near Walsingham for estimating flood flows at the dam to verify the results of the hydrologic model. Additionally, flood flow estimates from several past studies have been included as points of comparison in Table 13.

The Wills hydrology model produced slightly higher peak flows than LPRCA's existing model for the 25-year, 100-year, and 1000-year events, while yielding moderately lower peak flows for the Hurricane Hazel and PMF events. The Matrix Solutions index method for small sandy watersheds aligns closely with the results from the Wills Hydrology Model. Similarly, the Wills model produces slightly higher peak flows than the results from the Venison Creek station transfer, but the comparison is still fairly close. It also compares reasonably well with the estimates from Schroeter & Associates.

In contrast, the Moin and Shaw and Cumming Cockburn index methods generate significantly higher peak flows than those modeled for Deer Creek Dam. Wills notes that the Moin and Shaw method is only suitable for watersheds ranging from 51.3 km² to 697 km² in Region 6, which includes the entire LPRCA watershed and may be influenced by watersheds with till and clay soils. Since the Cumming Cockburn method is an update to the Moin and Shaw method, it likely shares similar base data.

Wills' estimate for the PMF is significantly lower than the inflow design flood calculated in 1977 but falls within the same order of magnitude as the results produced by LPRCA's existing model. Overall, Wills concludes that the results of the hydrology model are acceptable for the purposes of this study.

Table 13 – Flood Flow Comparison of Several Methods for Deer Creek Dam

Hydrologic Element	Drainage Area (km^2)	Peak Discharge (m^3/s)							
		Wills Modeled Flows 2025	LPRCA Existing Model Flows 2024	Matrix Solutions, RFA 2024	Venison Creek near Walsingham Station Transfer	Schroeter & Associates Modeled Flows (2006)	Moin and Shaw RFA (1986)	Cumming Cockburn RFA (2000)	Preliminary Engineering Study and Report Deer Creek Dam (1967)
25-year	15.13	5.3	2.59	5.8	4.2	4.5	8.3	12.9	N/A
100-year	15.13	7.1	5.77	7.7	5.5	9.3	11.6	16.1	N/A
1000-year	15.13	14.9	11.8	11.1	7.6	N/A	N/A	N/A	N/A
Hazel	15.13	12.2	20.3	N/A	N/A	N/A	N/A	N/A	N/A
PMF 12 HR	15.13	73.9	93.3	N/A	N/A	N/A	N/A	N/A	226.5

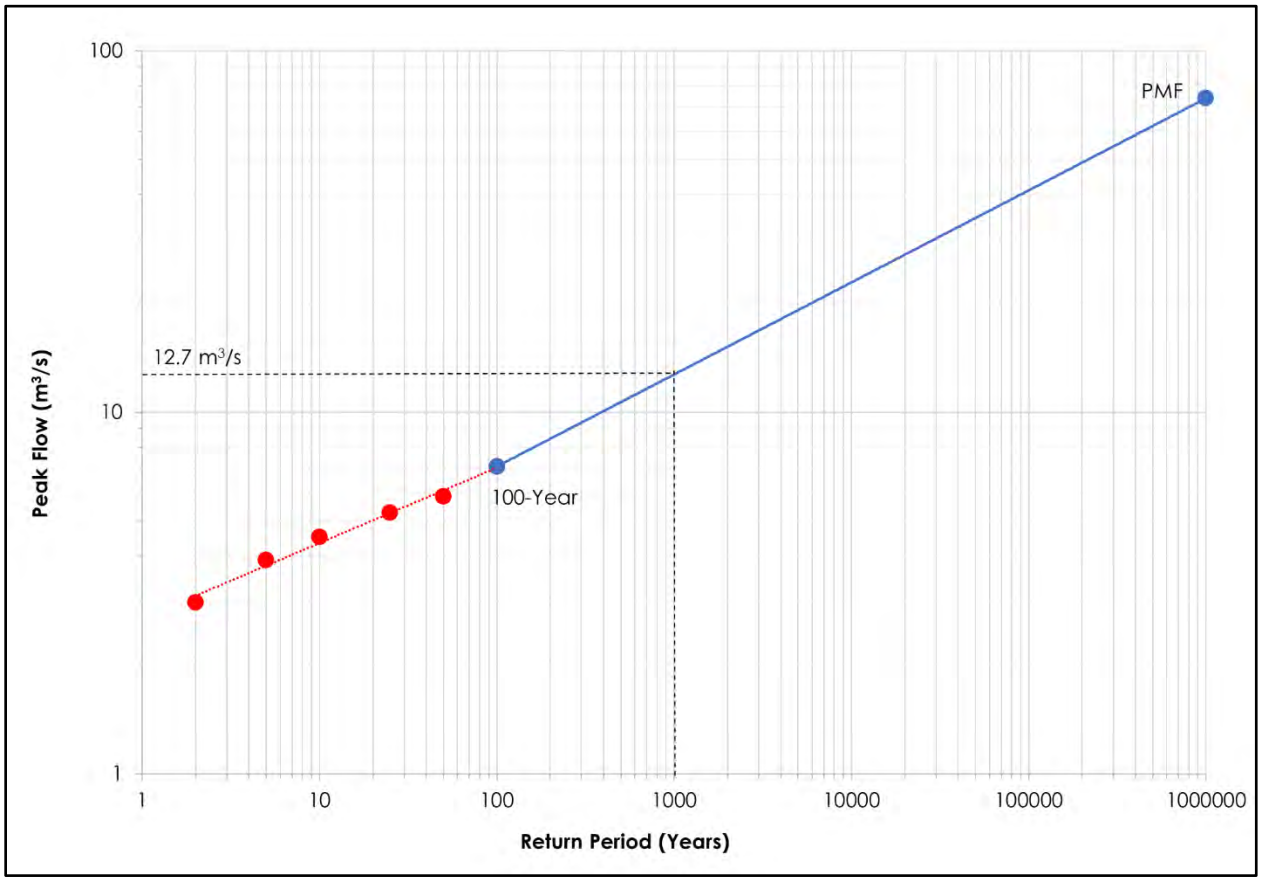
2.6 Determination of 1000-Year Peak Discharge

The MNR's preferred method to determine the 1000-year flood (peak flow and volume of runoff) is by interpolation between the 100-year flood and PMF using log paper where the annual exceedance probability for the PMF is set to 1:1,000,000 as per "Guidelines on Extreme Flood Analysis", Alberta Transportation, Transportation and Civil Engineering Division, Civil Projects Branch – November 2004. Wills used this method to estimate the 1000-year flood for both Deer Creek Dam and Big Creek above the Confluence with Deer Creek.

2.6.1 1000-Year Flood Estimation for Deer Creek Dam

The flood flow results from Wills hydrology model were plotted on log-log paper and can be found in Figure 8. The 1000-year event was estimated using linear interpolation between the PMF event and the modeled 100-year event. This resulted in a 1000-year flood flow estimate of 12.7 m^3/s. This is less conservative than the modeled 1000-year event using the extrapolated IDF curve. Therefore, the modeled 1000-year event was carried forward for the hydraulic analysis.

Figure 8 –1000-Year Flood Estimation for Deer Creek Dam



2.6.2 PMF and 1000-Year Flood Estimation for Big Creek Above Confluence

The LPRCA's existing HEC-HMS model was used to estimate the PMF for Big Creek above the confluence with Deer Creek for use in the hydraulic study, if required. Wills modified the LPRCA's calibrated model to remove the routing effects from the Teeterville Dam and the Lehman Dam as their existing elevation-storage-discharge curves did not extend high enough for the PMF event. This was assumed to be acceptable given both dams have small reservoirs and their routing effects are likely negligible during this high magnitude event. An areal reduction factor of 0.744 was applied to the 12-hour PMP event assuming a circular diameter of 44.4 km and equivalent circular area of 1546 km^2. The PMF for Big Creek above the confluence with Deer Creek was estimated to be 1409 m^3/s.

The flood flow results from Wills hydrology model were plotted on log-log paper and can be found in Figure 9. The estimated 1000-year flood for Big Creek above the confluence was 274.3 m^3/s through interpolation. This result is more conservative than the single station frequency estimate of 255.5 m^3/s and was therefore carried forward for the hydraulic analysis.

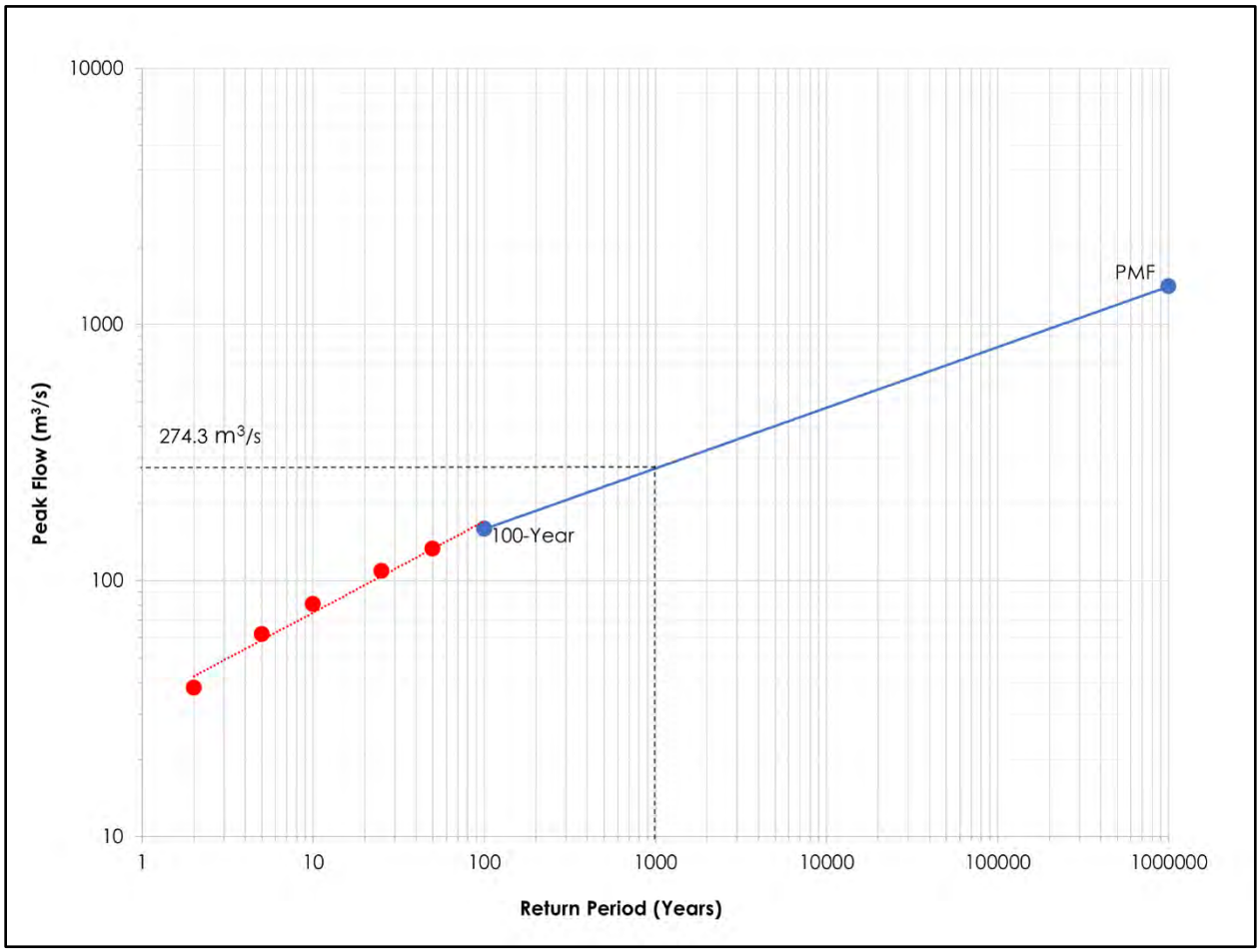
2.7 Summary and Selection of Peak Flows for Hydraulic Analysis

The peak flows selected for hydraulic modeling through various analysis can be found below in Table 14.

Table 14 - Flood Flow Estimates for Hydraulic Analysis

Return Period (years)	Probability	Deer Creek Dam (m^3/s)	Big Creek above Confluence (m^3/s)
Sunny Day	N/A	1.0	4.0
25	0.04	5.3	105.1
100	0.01	7.1	153.0
1000	0.001	14.9	274.3
1/3 Between	N/A	34.3	652.5
PMF	0.000001	73.9	1409.0
Hazel	N/A	11.8	N/A

Figure 9 – Big Creek Above the Confluence 1000-year Estimation using Interpolation



3.0 Hydraulic Assessment

3.1 Introduction

A detailed dam breach assessment was undertaken in order to estimate the potential incremental losses associated with a dam failure and determine the Hazard Potential Classification (HPC) and subsequently the Inflow Design Flood (IDF) for the dam. The dam breach assessment uses the peak flows presented in Section 2.7 and a HEC-RAS model run under unsteady flow conditions to evaluate a number of different dam breach scenarios (sunny day and a range of flood scenarios). The results of the hydraulic modelling are used to estimate the potential incremental losses that would result from a dam failure.

3.2 Available Data

3.2.1 Topographic Data

The primary source of topographic data for this project was the LiDAR DTM that is available through Land Information Ontario (LIO) Ontario GeoHub. The LiDAR-derived

DTM represents the bare earth surface and was generated from the classified LiDAR point cloud data. The User Guide, LiDAR – Digital Terrain Model (MNR, 2025), identifies the coordinate reference systems used as:

- The horizontal coordinate system of the Ontario Digital Terrain Model (LiDAR-Derived) is Universal Transverse Mercator (UTM), Zone 17N. The horizontal datum of the product is the North American Datum of 1983 Canadian Spatial Reference System epoch 2010 (NAD83 (CSRS)). The horizontal unit of measure (coordinate system axis units) for all raster grid cells is meters (m).
- The vertical coordinate system of the study area is CGVD2013. The vertical unit of measure (coordinate system axis units) for all raster grid cells in the products is meters (m).

The DTM was also used as the base dataset to create the inundation maps for the dam breach scenarios. All elevations presented in this report are in the CGVD2013 datum.

Wills created a bathymetrically enforced DTM using HEC-RAS and the survey data collected by Callon Dietz and Wills staff.

3.2.2 Construction Drawings

The LPRCA provided Wills with drawings of the Deer Creek Dam. This information was used to confirm the dam crest and sill elevations. Elevations on the drawings were assumed to use the CGVD28:78 vertical datum and were converted using the methodology described in Section 1.5 for the purposes of the dam break analysis.

3.3 Hydraulic Model Development

3.3.1 Model Selection

The HEC-RAS (Version 6.6) hydraulic model was selected by Wills as the preferred hydraulic model to be used for this project. HEC-RAS is a free hydraulic modeling software developed and maintained by the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Centre (HEC) with a long history of use in Canada and internationally. The software can perform hydraulic calculations in one-dimensional steady flow, one-dimensional (1D) unsteady flow, two-dimensional (2D) unsteady flow, and coupled one-dimensional/two-dimensional (1D/2D) flow conditions for a full range of natural and constructed channels. The software is suitable for many applications including floodplain mapping, open channel and hydraulic structure design, dam breach analysis, rain on grid, and sediment transport modeling. HEC-RAS includes built in GIS tools with which a significant portion of the hydraulic model can be developed, and the modeling results viewed.

The objective of the hydraulic model is to undertake a dam break analysis for a range of flow scenarios to determine the HPC and IDF for the dam. Given that dam break analyses are inherently unsteady flow problems, Wills determined that an 1D unsteady flow model would be the most appropriate for this application.

3.3.2 Cross-Section Data

The cross-section geometry for the hydraulic model was extracted from the LiDAR DTM using HEC-RAS's internal GIS module (RAS Mapper). The use of RAS Mapper ensures a properly georeferenced HEC-RAS model.

The selection of the cross-section spacing is dependent on several factors including bed slope, changing floodplain geometry, changing Manning's roughness conditions, hydraulic structure and obstruction locations, and changing velocity. A flat and wide floodplain with a homogenous slope may require significantly fewer cross sections then a steep watercourse to produce satisfactorily results. A total of 73 cross-sections were cut within the modeling domain. Wills placed the cross sections based on the factors discussed above and engineering experience. The location of each cross section can be found in the inundation mapping provided in Appendix C.

3.3.3 **Manning's Roughness Values**

Wills imported the LPRCA's landcover GIS layer into HEC-RAS to estimate the initial Manning's roughness values for the HEC-RAS model. The Manning's roughness values were then further refined as needed using areal imagery and the cross-section editor within HEC-RAS. A summary of the Manning's roughness values for each type of land cover can be found in Table 15.

Table 15 – Mannings Roughness Values

Raster Value OLCC	Land Cover Description	Mannings Roughness
Manual Input	Watercourse	0.035
1	Cultivated Crop Lands	0.55
2	Bare Soil Areas	0.55
3	Grasslands/Shrubs	0.55
4	Conifer Treed Forest	0.08
5	Deciduous Treed Forest	0.08
6	Mixed Treed Forest	0.08
7	Sparse Treed	0.08
8	Wetlands	0.08
9	Open Water Areas	0.035
10	Barren Areas	0.045
12	Commercial/Industrial	0.055
13	Residential	0.045
14	Settlement Open Areas	0.045
15	Roads	0.015

3.3.4 Hydraulic Structures

There are four hydraulic structures within the HEC-RAS model domain, the Deer Creek Dam and the bridges at Walsingham Townline Road, 6th Concession Road, and Norfolk County Road 60. The Deer Creek Dam was modeled based on the topographic survey completed by Wills and supplemented by the available drawings. The bridges were modelled using field measurements collected by Wills as well as the LiDAR DTM.

The stoplogs in Deer Creek's spillway chute were assumed to remain in place for all scenarios. The reservoir was modeled as a storage area and used the elevation-storage curve developed by Wills and presented in Section 2.3.12.

3.3.5 Boundary Conditions

The study area required several boundary conditions in HEC-RAS as it is an unsteady model. In HEC-RAS the downstream boundary condition requires an initial estimate of the friction slope. Wills estimated the friction slope to be the average bed slope of the downstream cross sections. Lateral Inflow hydrographs are used to introduce hydrographs within the modeling domain. The location, type of boundary condition, and the estimated initial friction slope can be found can be found in Table 16.

Table 16 – Boundary Conditions

Name	Cross Section Number	Boundary Condition Type	Estimated Initial Friction Slope (m/m)
Storage Area 1	Deer Creek Storage Area	Lateral Inflow Hydrograph	N/A
Deer Creek Dam	10994	T.S. Gate Openings	N/A
Big Creek	8591	Lateral Inflow Hydrograph	N/A
Subbasin5	890	Normal Depth	0.0005

3.4 Detailed Dam Breach Model Set-up

Dam breach analyses are completed to simulate the effects of a sudden release ('wave') of water resulting from a dam breach and are used as part of the HPC and IDF selection process. The unsteady flow HEC-RAS model was used to complete the dam breach analysis.

3.4.1 Dam Breach Parameters

When analyzing a dam breach scenario within HEC-RAS, a number of key dam breach parameters are required. The key dam breach parameters include the breach bottom elevation, bottom width, and formation time. The selection of the breach bottom elevation is typically based on the natural ground elevation at the dam and, as a starting point, the breach bottom width and formation time are estimated using

equations developed with data from historical dam failures. The selected failure mode and the associated dam breach parameters are described below.

One failure mode was considered for the purposes of determining the HPC. This failure mode assumed a piping failure of the earth embankment. Wills has prepared a Microsoft Excel spreadsheet to estimate the dam breach parameters for earth embankments using several of the common empirical equations. Estimated dam breach parameters are included in Table 16 and the dam breach estimates are included in Appendix C. The modelled breach bottom elevation was selected as the toe elevation of the downstream embankment. The calculated breach formation time was ranged from 0.5 to 0.8 hours using empirical equations. This was increased to 1.5 hours to maintain numerical stability in the HEC-RAS model which is still between the typical recommended range of 0.5 to 2 hours.

Table 17 – Estimated Dam Breach Parameters – Earth Embankment

Failure Scenario	Modelled Breach Bottom Elevation (m)	Modelled Breach Bottom Width (m)	Modelled Breach Formation Time (hours)	Modelled Breach Side Slopes (H:V)
Sunny Day	191.5	16.9	1.5	0.9:1
25-year	191.5	17.4	1.5	0.9:1
100-year	191.5	17.6	1.5	0.9:1
1000-year	191.5	18.1	1.5	0.9:1
1/3 Between 1000-Year and PMF	191.5	19.3	1.5	0.9:1
PMF	191.5	21.2	1.5	0.9:1

3.4.2 Modelled Inflow Rates

One flow rate for the sunny day dam breach scenario and five flow rates under flood conditions were modelled as part of the dam breach analysis. The inflow hydrograph was routed through the reservoir for Deer Creek. The peak flows on Big Creek were held constant throughout the entirety of the simulations. The peak flows used in the analysis are shown in Table 10.

3.5 Dam Breach Results

As described previously, the unsteady flow HEC-RAS model was used to complete the detailed dam beach analysis. A summary of the flows, incremental flows, and depths due to the dam breach for the failure scenarios are provided in Tables 18. The HEC-RAS results tables and inundation maps are included in Appendix C.

Table 18 – Summary of Model Results Earth Embankment

Flood Event	Dam Condition	Peak Flow Rate (m^3/s)	Max. Incremental Flow (m^3/s)	Max. Incremental Depth (m)
Sunny Day	No Breach	1.0	299.0	4.51
	Dam Breach	300.0		
25-year	No Breach	4.2	312.1	4.05
	Dam Breach	316.3		
100-year	No Breach	5.7	317.0	3.89
	Dam Breach	322.7		
1000-year	No Breach	11.8	331.00	3.59
	Dam Breach	342.8		
1/3 Between	No Breach	27.8	368.2	3.40
	Dam Breach	396.0		
PMF	No Breach	62.8	419.8	3.18
	Dam Breach	482.6		

3.6 Hazard Potential Classification and Inflow Design Flood

3.6.1 Hazard Potential Classification

The Technical Bulletin for Classification and Inflow Design Flood Criteria (MNR, 2011), outlines the requirements for the classification of dams. In Ontario, dams are classified using the HPC system, which categorizes dams according to the potential hazards presented by the dam. The HPC is an assessment of the consequences of dam failure, not the risk of dam failure based on its current condition. The HPC is determined through an assessment of the greatest incremental losses that could result from an uncontrolled release of the reservoir due to the failure of a dam. Potential incremental losses are assessed with respect to life, property, the environment, and cultural-built heritage sites at the dam site, upstream and downstream. Table 19 outlines the four classification categories and describes how the potential incremental losses are assessed for each potential hazard. Dams are classified according to the highest potential incremental hazard. Incremental losses refer to losses from a dam failure that are above and beyond those which may be expected to occur under the same natural conditions with the dam in place but without failure of the dam.

Dams require two HPCs, one based on dam failure during normal (sunny day) conditions and a second based on dam failure under flood conditions. To determine the HPC for the normal (sunny day) and flood scenarios, the inundation maps and HEC-RAS model results were reviewed.

Table 19 – Hazard Potential Classification

Hazard Potential	Life Safety[2]	Property Losses	Environmental Losses	Cultural Built Heritage Losses[3]
Low	No expected loss of life	Very low damage to property < $300,000	Minimal loss of habitat with high capability of restoration	Reversible damage to municipally designated cultural heritage sites
Moderate	No expected loss of life	Moderate damage < $3 million	Moderate loss of habitat with moderate capability of restoration	Irreversible damage to municipally designated cultural heritage sites
High	Expected loss of life 1-10 persons	Appreciable damage < $30 million	Appreciable loss of habitat - reversible damage to habitat	Irreversible damage to provincially or nationally designated cultural heritage sites
Very High	Expected loss of life 11 or more persons	Extensive damage > $30 million	Extensive loss of habitat with no feasibility of recovery	

Note:

1. Abbreviated Table 1 - Hazard Potential Classification (Technical Bulletin for Classification and Inflow Design Flood Criteria (MNR, 2011).
2. Life Safety is assessed using the "2 by 2 Rule" as outlined in the Technical Guide River and Stream Systems: Flooding Hazard Limit. For dam failures under flood conditions, the potential for loss of life is assessed based on permanent dwellings (including habitable buildings and trailer parks) only. For dam failures under normal (sunny day) conditions, the potential for loss of life is assessed based on both permanent dwellings (including habitable buildings, trailer parks and seasonal campgrounds) and transient persons.
3. Cultural-Built Heritage Losses are assessed with respect to sites designated under the Ontario Heritage Act or nationally recognized heritage sites.

3.6.2 Sunny Day Failure

During a sunny day dam failure, the headpond will be at approximately the elevation of the top stoplog/invert of the weir, meaning there is a significant amount of storage within the reservoir. A failure of the dam under this scenario would result in a flood wave peak of approximately 300.0 m^3/s, and a maximum incremental flow of 299.0 m^3/s over

the non-breach peak flow. The extent of the incremental flooding is shown on the inundation mapping included in Appendix C.

Life Safety Assessment

This report considers a detailed dam breach analysis conducted using an unsteady flow HEC-RAS model, which offers an in-depth analysis into risk to life. The life safety assessment for this project considers two methods. The first method is the 2 x 2 rule, which is outlined in the Technical Guide - River and Stream Systems: Flooding Hazard Limit (MNR, 2002). The 2 x 2 rule looks at the product of the depth and velocity in order to check if the life safety threshold is exceeded. The second method is a review of the rate of change of the water levels in the upstream reservoir to determine the potential impacts to users.

The Technical Bulletin for Classification and Inflow Design Flood Criteria (MNR, 2011) requires that the life safety assessment for the sunny day dam failure consider the potential threat to life in permanent dwellings as well as the potential threat to life of transient persons within the downstream area.

The results of the 2 x 2 rule analysis indicate that there are areas downstream of the dam where the 2 x 2 rule is exceeded in several locations and where people may be located. Fishing, hunting, canoeing, and kayaking are known and common recreational activities that occur downstream of the dam. Additionally, Wills staff observed students in the floodplain immediately west of Valley Heights Secondary School while surveying the confluence of Deer Creek and Big Creek. It is likely a common area for students to congregate. Based on the model results it is likely that there would be fatalities if the dam were to fail, and people were within the valleys of Deer Creek or Big Creek upstream of Walsingham Townline Road as the flood wave fills the entire valley in 10-20 minutes. The likelihood of people being downstream of the dam is possible, and the consequences would likely be 1-10 fatalities.

The rate of change of water levels in the reservoir was assessed using the results of the unsteady flow HEC-RAS model. The model results indicate that the reservoir would drop by approximately 3.29 m within 1 hour of the dam breach initiation and 8.07 m after the first 2 hours. The beach area is located 360 m from the breach location and swimmers would likely be able to swim to safety in the event of a breach. There is potential that people boating within the headpond area may be caught in the current if the dam were to fail. Motorized boats would likely be able to get to safety, however canoe and kayakers may struggle if too close the dam.

In summary, the results of the life safety assessment, undertaken using the results of the detailed dam breach analysis and the unsteady HEC-RAS model, indicate that there is a moderate potential for incremental loss of life associated with the failure of the reservoir during the sunny day dam breach scenario estimated between 1-10 persons.

Property Losses

In the case of the sunny day dam failure, no buildings or water crossings are expected to be incrementally inundated. Therefore, property damage is expected to be less than $300,000 for the sunny day dam failure.

Environmental Losses

In the case of the sunny day dam failure, the reservoir will completely drain, resulting in moderate loss of aquatic habitat. It is expected that damages from any sediment release would be temporary, and natural restoration would be possible.

Cultural/Built Heritage Losses

In the case of the sunny day dam failure, the incremental water level due to the dam failure is not expected to have any measurable effect on cultural or built heritage resources.

Based on the above assessment, **the HPC for the sunny day dam breach scenario is assessed as being High** since there is potential for incremental loss of life of 1-10 people.

3.6.3 Flood Failure

During a dam failure in a flood scenario (i.e., 25-year and 100-year), the reservoir is expected to be at full flood stage; therefore, the failure of the dam will result in a wave of water being released downstream, which could result in incremental losses within the overbank areas. The extent of the incremental flooding is shown on the inundation mapping, included in **Appendix C**.

Life Safety Assessment

During a dam failure in a flood scenario, there are several permanent dwellings located within the inundation area impacted by the breach. Wills reviewed the HEC-RAS 2x2 results for all events modeled. Several residential properties were incrementally flooded during the dam breach scenario, transitioning from "safe" to "not safe" using the 2x2 rule. Using Norfolk County's 2021 Census data from Statistics Canada, Wills assumed an average household size of 2.4 people to estimate potential loss of life and rounded up to the nearest whole person. The results of the life safety analysis can be found in **Table 20**.

Table 20 – Life Safety Analysis

Event	Non-Breach Potential Loss of Life	Breach Potential Loss of Life	Incremental Potential Loss of Life
25-year	0	0	0
100-year	0	0	0
1000-year	0	0	0
1/3 Between 1000-Year and PMF	3	5	2
PMF	5	8	3

Property Losses

In the case of the flood dam failure, several buildings and water crossings are expected to be incrementally inundated. Therefore, it is anticipated that the incremental damage from a breach would be more than $300,000, but less than $3,000,000 for some of the flood dam breach scenarios considered.

Environmental Losses

In the case of the sunny day dam failure, the reservoir will completely drain, resulting in moderate loss of aquatic habitat with moderate capability of restoration. It is expected that damages from any sediment release would be temporary, and natural restoration would be possible.

Cultural/Built Heritage Losses

In the case of the flood dam failure, the incremental water level due to the dam failure is not expected to have any measurable effect on cultural or built heritage resources.

Based on the above assessment, **the HPC for the flood dam breach scenario is assessed as being High** since potential loss of life is expected to be between 1-10 people.

3.6.4 Inflow Design Flood

The IDF is the most severe inflow flood for which a dam and its associated facilities are designed. Two methods can be used to determine the IDF. The IDF is typically selected using Table 2 in the Technical Bulletin for Classification and Inflow Design Flood Criteria (MNR, 2011); however, the Technical Bulletin for Classification and Inflow Design Flood Criteria (MNR, 2011) also makes allowance for the use of an incremental analysis for cases where the dam owner wishes to explore the possibility of selecting a lower magnitude IDF. The incremental analysis is discussed in more detail in the Technical Bulletin for Classification and Inflow Design Flood Criteria (MNR, 2011); however, Wills does not believe it would provide a benefit in this case.

Table 10 outlines the peak flows estimated at the Deer Creek Dam, and **Table 21** outlines the range of minimum Inflow Design Floods for the various HPCs. With reference to **Table 21**, and given the High HPC for the flood scenario, the IDF should be selected as the 1/3 between 1000-year flood and PMF.

The peak flow of 34.3 m^3/s, 1/3 between the 1000-year and PMF was selected as the IDF peak flow.

Table 21 – Range of Minimum Inflow Design Floods[1]

Hazard Potential	Range of Minimum Inflow Design Floods			
	Life Safety		Property and Environment	Cultural Built Heritage
Low	25-year flood to 100-year flood			
Moderate	100-year flood to 1000-year flood or Regulatory flood (whichever is greater)			
High	1-10	1/3 between the 1000-year flood and the PMF.	1000-year flood or Regulatory flood (whichever is greater) to 1/3 between the 1000- year flood and PMF.	1000-year flood or Regulatory flood (whichever is greater).
Very High	11-100	2/3 between the 1000-year flood and the PMF.	1/3 between the 1000-year flood and PMF to PMF.	
	Greater than 100	PMF		

Note:

1. Abbreviated Table 2 - Range of Minimum Inflow Design Floods (Technical Bulletin for Classification and Inflow Design Flood Criteria (MNR, 2011).

3.7 Hydraulic Capacity and Freeboard Assessment

3.7.1 Hydraulic Capacity for IDF

The hydraulic capacity for the IDF was assessed using the unsteady-flow HEC-RAS model. Based on the model results, the IDF (1/3 between 1000-year flood and PMF, 34.3 m^3/s) reaches a headwater elevation of 201.21 m. The IDF remains below the lowest point on the earth embankment (203.83 m). Therefore, it is anticipated that the dam has sufficient hydraulic capacity to convey the IDF.

3.7.2 Freeboard Analysis

A freeboard analysis was carried out as recommended in the Technical Bulletin for Spillways and Flood Control Structures (MNR, 2011):

1. No overtopping of the crest of the dam by 95% waves caused by the most critical wind with a return frequency of 1:1000 with the reservoir at its normal maximum.
2. No overtopping of the crest of the dam by 95% of the waves caused by the design wind or, if information to calculate the design wind is not available, a

critical wind with a return frequency of 1:100 with the reservoir at its maximum level during passage of the IDF.

Detailed hourly wind data to calculate the design wind was not available; therefore, critical wind speed with 1000-year return period was approximated by extrapolating the wind pressures provided for Tillsonburg in the Canadian Highway Bridge Design Code. The method used was straight line extrapolation on a log-probability plot. The wind speeds are shown in Table 22.

Table 22 – Wind Speed Data

Return Period	Wind Speed (km/h)	Source
10-year	81.8	Canadian Highway Bridge Design Code
50-year	96.9	Canadian Highway Bridge Design Code
100-year	103.9	Canadian Highway Bridge Design Code
1000-year	132.1	Log-Probability Extrapolation

The significant wave height and wave run-up level with a 5% exceedance probability were calculated using methods laid out in the Coastal Engineering Manual (USACE, 2011).

The elevation of the top stoplog for the dam was adopted as normal maximum operating level for freeboard calculation during non-flood conditions. The dam characteristics used for the freeboard analysis are shown in Table 23, and the results of the freeboard calculations are shown in Table 24. The freeboard calculation spreadsheet is provided in Appendix C.

Table 23 – Dam Characteristics for Freeboard Analysis

Dam Characteristics	Deer Creek Dam
Hazard Classification	High
Crest Elevation (m)	203.83
Average Reservoir Depth (m)	9.5
Embankment Slope (H:V)	3:1
Embankment Material	Rip-Rap
Normal Operating Water Level (m)	200.27
IDF Water Level (m)	201.21

Table 24 – Results of Freeboard Analysis

Description	Deer Creek Dam	
	Normal	IDF
Critical Wind Return Period	1:1000	1:100
Critical Wind Direction	N	N
Wind Speed (km/hr)	132.1	103.9
Effective Fetch (km)	0.73	0.73
Significant Wave Height (m)	0.82	0.61
Effective Fetch for Wind Setup (km)	0.73	0.73
Wave Run-up (5% Exceedance, m)	0.84	0.62
Wind Setup (m)	0.02	0.01
Required Freeboard, Crest (m)	0.86	0.63
Required Freeboard (Top of Impervious Core)	0.02	0.01
Available Freeboard, Crest (m)	3.56	2.62
Available Freeboard (Top of Impervious Core, m)	2.89	1.95
Adequate Freeboard	Yes	Yes

Notes:

1. Wind data taken from Canadian Highway Bridge Design Code.
2. The analysis was checked to be fetch-limited for the normal operating analysis, and fetch-limited for the IDF analysis.
3. Wave run-up Calculations use methods from USACE.

The results of the analysis indicate that the dam has sufficient freeboard for the Normal Operating Level and the Inflow Design Flood Level.

4.0 Structural Stability Assessment

4.1 Introduction

The structural stability assessment was completed in accordance with the Technical Bulletin for Structural Design and Factors of Safety (MNR, 2011). The purpose of the structural stability assessment was to assess the stability of the concrete gravity dam sections with respect to sliding, overturning, and overstressing of the concrete or foundation.

The structural stability assessment was completed for five typical sections as outlined below:

- One section for retaining walls, at the upstream wingwall location.

- One section at control structure consists of Ogee weir and wingwall.
- One typical section at the chute spillway.
- On section at transition between Chute and Stilling Basin.
- One section at Stilling basin location.

The LPRCA provided background information for the structure and design drawings as part of the background materials.

The methodology and the results of the structural stability assessment are presented herein, and the detailed calculations are provided in Appendix D.

4.2 Methodology

The analysis was based on a rigid body limit equilibrium method with the various load combinations treated as static because of the relatively sustained nature of the loads involved. The following loads were considered in the assessment of the concrete structures:

- Dead loads of permanent structures, rock or soil backfill, silt deposited against the structure and any significant equipment loads.
- Hydrostatic loads, including the normal summer, normal winter and IDF headwater levels, with corresponding tailwater levels.
- Internal water pressure and foundation uplift.
- Ice load.
- Earthquake load.

Seismic analyses are performed at different levels of sophistication. In the case of the Deer Creek Dam, a pseudo-static method of analysis was used to assess the earthquake loading on the concrete gravity dam sections.

The position of the resultant force, the normal stresses at the heel and toe and the calculated sliding and overturning factors were reviewed and compared to the requirements of the Technical Bulletin for Structural Design and Factors of Safety (MNR, 2011).

In accordance with the Technical Bulletin for Structural Design and Factors of Safety (MNR, 2011), the following loading combinations were considered:

1. Usual Load (Summer) includes the following loads acting in combination: Dead load; Hydrostatic Load (maximum normal operating level); Soil Load; and Uplift.
2. Usual Load (Winter) includes the following loads acting in combination: Dead load; Hydrostatic Load (winter operating level); Ice Load; Soil Load; and Uplift.
3. Unusual Load (Flood) includes the following loads acting in combination: Dead Load; Hydrostatic Load (IDF flood level); Soil Load; and Uplift Load.

4. Earthquake loading includes the following loads acting in combination; Maximum Design Earthquake, Dead loads, Hydrostatic Load (maximum normal operating level); Soil Load; and Uplift.

4.3 Design Earthquake Selection

The HPC under the normal (sunny day) scenario is used to determine the Design Earthquake (DE). Table 25 outlines the DE selection criteria.

Table 25 – Design Earthquake Criteria

Hazard Potential	Earthquake Design Ground Motion Annual Exceedance Probability			
	Life Safety		Property and Environment	Cultural-Built Heritage
Low	500-year			
Moderate	500-year to 1000-year			
High	1-10	2,500-year	1,000-year to 2,500-year	1,000-year
Very High	11-100	5,000-year	2,500-year to 10,000-year	
	Greater than 100	10,000-year		

Note: 1. Abbreviated Table 1 - Design Earthquake Criteria (Technical Bulletin for Seismic Hazard Criteria, Assessment and Considerations (MNR, 2011).

With reference to Table 25 and given the selection of the High HPC for the normal (sunny day) scenario, the DE should be the 2500-year event.

According to the 2020 National Building Code Seismic Hazard Calculator, the Peak Ground Acceleration (PGA) for a 2500-year return period is 0.177g. This represents Site Class E (Claey -Silt soil), which is conservative given that the dam is assumed to be sitting on a silty clay.

The noted PGA value was used as the horizontal seismic coefficient (S_H) and the vertical seismic coefficient (S_V) was selected as $^2/_3 S_H$.

4.4 Loading Data

Wills determined the loading data to be used in the analysis based on the identified loading combinations and the existing information. The loading data is summarized in in Table 26.

Table 26 – Loading Data – Typical Pier Section

Loading Case	Water Level (m)		Ice Load (kN/m)	Seismic Coefficient (%g)
	Headwater	Tailwater		
1. Usual Load (Summer)	200.27	185.85	0	-
2. Usual Load (Winter)	200.27	185.85	75	-
3. Unusual Load (Flood)	201.21	187.57	0	-
4. Extreme Load (Earthquake)	200.27	185.85	0	17.7

It is noted that the 75 kN/m ice load is based on Technical Bulletin for Structural Design and Factors of Safety (MNR, 2011).

4.5 Results

As described in Section 4.1, the stability analysis was completed for six typical sections. The results of the structural stability assessment are summarized in Tables 27 to 31 and the detailed calculations and results are provided in Appendix D.

Table 27 – Structural Stability Assessment Results – Typical Wingwalls / Retaining Wall Section

Loading Case	FOS Sliding		FOS Overturning		Position of Resultant from Toe (m)	Location of Resultant	Foundation Bearing Pressure (kPa)	
	Required	Computed	Required	Computed			Allowable	Computed
1. Usual Load (Summer)	1.5	4.31	1.5	5.57	1.45	Inside Middle Third	150	108
2. Usual Load (Winter)	1.5	4.31	1.5	5.57	1.45	Inside Middle Third	150	108
3. Unusual Load (Flood)	1.3	2.49	1.3	2.96	1.47	Inside Middle Third	150	102
4. Extreme Load (Earthquake)	1.1	1.65	1.1	1.96	1.07	Outside Middle Third Inside the Base	150	145

Table 28 – Structural Stability Assessment Results – Typical Ogee Section

Loading Case	FOS Sliding		FOS Overturning		Position of Resultant from Toe (m)	Location of Resultant	Foundation Bearing Pressure (kPa)	
	Required	Computed	Required	Computed			Allowable	Computed
1. Usual Load (Summer)	1.5	3.97	1.5	1.86	6.78	Inside Middle Third	150	25
2. Usual Load (Winter)	1.5	1.69	1.5	1.76	6.31	Inside Middle Third	150	30
3. Unusual Load (Flood)	1.3	1.91	1.3	1.43	5.47	Inside Middle Third	150	31
4. Extreme Load (Earthquake)	1.1	1.19	1.1	1.67	6.13	Inside Middle Third	150	28

Note: Allowable foundation bearing pressure based on Ontario Building Code Table 9.4.4.1 (Allowable Bearing Pressure for Soil or Rock), Item 7, Till.

Table 29 – Structural Stability Assessment Results – Typical Stilling Basin Section

Loading Case	FOS Sliding		FOS Overturning		Position of Resultant from Toe (m)	Location of Resultant	Foundation Bearing Pressure (kPa)	
	Required	Computed	Required	Computed			Allowable	Computed
1. Usual Load (Summer)	1.5	15.24	1.5	2.27	3.54	Inside Middle Third	150	31
2. Usual Load (Winter)	1.5	15.24	1.5	2.27	3.54	Inside Middle Third	150	31
3. Unusual Load (Hydraulic Jump)	1.3	5.65	1.3	1.43	3.47	Inside Middle Third	150	20
4. Extreme Load (Earthquake)	1.1	6.32	1.1	2.15	3.39	Inside Middle Third	150	35

Table 30 – Structural Stability Assessment Results – Typical Transition Wall to Stilling Basin

Loading Case	FOS Sliding		FOS Overturning		Position of Resultant from Toe (m)	Location of Resultant	Foundation Bearing Pressure (kPa)	
	Required	Computed	Required	Computed			Allowable	Computed
1. Usual Load (Summer)	1.5	1.6	1.5	2.4	7.5	Inside Middle Third	150	9
2. Usual Load (Winter)	1.5	1.6	1.5	2.4	7.5	Inside Middle Third	150	9
3. Unusual Load (Hydraulic Jump)	1.3	1.6	1.3	2.4	7.5	Inside Middle Third	150	9
4. Extreme Load (Earthquake)	1.1	1.02	1.1	2.2	7	Inside Middle Third	150	28

Table 31- Structural Stability Assessment Results – Typical Chute Slab for Unit Length (1.0m)

Loading Case	FOS Sliding		FOS Overturning		Position of Resultant from Toe (m)	Location of Resultant	Foundation Bearing Pressure (kPa)	
	Required	Computed	Required	Computed			Allowable	Computed
1. Usual Load (Summer)	1.5	3.5	1.5	5	0.4	Inside Middle Third	150	17.5
2. Usual Load (Winter)	1.5	3.5	1.5	5	0.4	Inside Middle Third	150	17.5
3. Unusual Load (Hydraulic Jump)	1.3	3.5	1.3	5	0.4	Inside Middle Third	150	17.5
4. Extreme Load (Earthquake)	1.1	2.22	1.1	1.35	0.2	Outside Middle Third Inside the Base	150	32

Note: Allowable foundation bearing pressure based on Ontario Building Code Table 9.4.4.1 (Allowable Bearing Pressure for Soil or Rock), Till.

4.6 Discussion of Results

The results of the structural stability assessment indicate that the typical sections meet the minimum factors of safety for all loading conditions.

The results of the structural stability assessment also indicate that the typical retaining wall sections meets the minimum factors of safety for the Usual Load (Summer), Unusual Load (Flood) and Extreme Load (Earthquake) loading conditions. The water table level in the backfill behind the retaining walls should not exceed the IDF water level (201.21).

For Transition Walls to Stilling Basin, seismic was a little lower than requirement but it will be acceptable.

5.0 Geotechnical Stability Assessment

5.1 Introduction

Cambium Inc. (Cambium) was retained by Wills to complete a slope stability analysis and visual inspection for the Deer Creek Dam. One typical section was analyzed on the left earth embankment. One typical section through the bypass outlet was also analyzed as a sensitivity analysis. A summary of Cambium's analysis and results is included below, and the full details of Cambium's study are included in their Geotechnical Engineering Report, which is included in Appendix E.

5.2 Subsurface Stratigraphy and Soil Parameters

Cambium reviewed the results of two geotechnical investigations conducted prior to construction of the embankment dam: the 1967 Borehole Investigation by Vance, Needles, Bergendoff and Smith Ltd. and the 1968 Borehole Investigation by William Trow Associates. No post-construction geotechnical data was available for review. These investigations included six boreholes drilled at or near the proposed dam location (refer to Appendix E for borehole locations and logs), and the as-built section drawings provided additional insights into the dam embankment components (impervious core, pervious shell, and granular filter materials). The pre-construction data indicate that the embankment was founded on native soils consisting of up to about three metres of noncohesive alluvium (sand and silt) overlying a stiff to hard clayey silt to silty clay layer. A sheet pile cutoff was installed through the more permeable alluvium and embedded into this lower-permeability stratum.

Neither the embankment composition nor the subsurface conditions below the dam have been verified by a new investigation. Consequently, the current modeling assumptions are high-level and rely on the accuracy of the as-built drawings, which qualitatively match the historic borehole data. The preliminary soil mechanical parameters used for slope stability analyses were derived by integrating the borehole logs, as-built drawings, and Cambium's experience with similar materials and are provided in Table 32. For a more detailed presentation of the original geotechnical data, consult the 1967 and 1968 investigation reports; likewise, refer to the

recommendations in subsequent sections of this report regarding further subsurface investigation necessary to confirm the assumptions used in the analyses.

Table 32 – Preliminary Soil Mechanical Properties for Analysis

Stratum	Soil Mechanic Properties		
	Friction Angle (°)	Cohesion (C) (kN/m^2)	Unit Weight (kN/m^3)
Impervious Core	25	2	19
Pervious Shell	30	0	19.5
Granular Filter	32	0	20.5
Sand and Silt (Alluvium); Loose	28	0	19
Clayey Silt to Silty Clay; stiff to hard	25	5	19

5.3 Groundwater Conditions

Cambium expects that current groundwater conditions at or near the dam are likely to fluctuate seasonally, corresponding to water levels upstream and downstream. The water level within the embankment is expected to decrease from the retained pond upstream to the downstream tailwater elevation, varying with the dam's composition and the permeability of its materials. Because no monitoring wells have been installed through the embankment crest, no data is available regarding water levels within the dam. Further details can be found in Cambium's report in Appendix E.

5.4 Geometry

The geometry of the analyzed embankment sections was primarily derived from as-built drawings, which include a typical cross section and an additional section for sensitivity analysis at the conduit/outlet structure. The impervious core, pervious shells, and granular filter were adopted from these drawings, along with approximate foundation thicknesses. Although the upstream slope exhibits some inconsistent slope inclinations, its average slope inclination (3H:1V) still aligns with the as-built design.

Only one typical embankment section (Section 1-1) was modeled by Cambium. However, a sensitivity analysis was performed for the downstream slope at the outlet structure, extending the slope toe downward to match the outlet's top elevation; this modified section is referred to as Section 1A-1A. Both sections were checked for plausibility using updated topographic/bathymetric data provided by Wills. A summary of these section geometries is provided in Table 33. The section locations can be found in Cambium report in Appendix E.

Table 33 – Embankment Section Geometry Analysis

Parameter	Section 1-1 (Typical Section)	Section 1A-1A (Concrete Outlet/Bypass Channel Location)
Slope Crest Elevation	204.2	204.2
Slope Toe Elevation – Downstream	191.7 (top of stabilizing toe berm)	187.9 (top of outlet structure for concrete conduit)
Slope Toe Elevation – Upstream	189.0	189.0
Slope Height – Downstream	12.5	16.3
Slope Height – Upstream	15.2	15.2
Slope Inclination – Downstream	3.5H:1V	3.5H:1V
Slope Inclination – Upstream (As-Built)	3H:1V (above 195.1 masl) 4H:1V (below 195.1 masl)	3H:1V (above 195.1 masl) 4H:1V (below 195.1 masl)

5.5 Methodology and Loading Cases

Slope stability analyses were carried out to assess the Factor of Safety (FoS) against slope failure for the slopes adjacent to the concrete structure. The slope stability analyses were performed using Slope/W, an industry standard two-dimensional limit equilibrium slope-modelling program. Inputs required for the Slope/W program include soil stratigraphy and geotechnical design parameters. The Morgenstern-Price method of slices was employed for the analyses. Using this method, Factors of Safety were generated for a variety of potential slip surfaces and the slip surface with the lowest calculated Factor of Safety was used.

The loading cases used in the stability evaluation are based on the LRIA Technical Bulletin for Geotechnical Design and Factors of Safety (MNR, 2011). The design criteria contained within the LRIA Technical Bulletins include hydrotechnical, seismic and static stability criteria as determined by the HPC of the dam.

For the pseudostatic analysis, the dam was assumed to have a High HPC with Normal (Sunny Day) water levels applied. According to the LRIA Technical Bulletin, a 1:2,500-year earthquake is the required design scenario for a High HPC dam with a potential loss of life of 10 or fewer persons.

The 1:2,500-year peak ground acceleration (PGA) was determined to be 0.177g, based on the 2020 National Building Code Seismic Hazard Calculation (Natural Resources Canada). This PGA value assumes a Site Class E – Soft Soil classification to provide a conservative analysis, considering that loose soils were encountered beneath the proposed dam footprint during earlier investigations.

The Sunny Day and IDF elevations were determined by Wills using the hydraulic model. The water surface elevations used in the geotechnical analysis are presented below in Table 34.

Table 34 – Embankment Section Geometry Analysis

Scenario	Headwater/Upstream Water Surface Elevation (m)	Tailwater/Downstream Water Surface Elevation (m)
Normal (Sunny Day)	200.27 m	185.85 m
Inflow Design Flood (IDF)	201.21 m	187.57 m

5.6 Results

The results of Cambium's slope stability analyses are provided in Table 35 and additional details are included in the Geotechnical Engineering Report, which is included in Appendix E.

Table 35 – Slope Stability Assessment Results

Loading Case	Location	Factor of Safety	
		Calculated	Minimum
Section 1-1 – Typical Section			
Normal Operating Conditions	Upstream	1.6	1.5
	Downstream	2.1	
Pseudostatic Analysis	Upstream	1.0	1.0
	Downstream	1.3	
Inflow Design Flood	Upstream	1.6	1.3
	Downstream	1.3	
Rapid Drawdown	Upstream	0.9	1.2
Section 2 – Section 1A – 1A Conduit/Outlet Structure			
Normal Operating Conditions	Downstream	1.8	1.5
Pseudostatic Analysis	Downstream	1.2	1.0
Inflow Design Flood	Downstream	1.7	1.3

The results of Cambium's stability analysis indicate that the minimum factors of safety are anticipated to be met on the upstream and downstream slopes under normal operating conditions, psuedostatic conditions, and IDF conditions. The upstream slope does not fulfill LRIA design minimum factor of safety criteria under the rapid drawdown condition, where a factory of safety of 0.9 was calculated. This scenario assumed that

the reservoir would be drawn to the upstream toe of the slope from the normal sunny day operating water level.

6.0 Dam Safety Management

6.1 Dam Inspections

The 2007 CDA Dam Safety Guidelines recommend that the following types of inspections be carried out:

- Routine Inspections – Routine inspections are generally completed on a weekly or monthly frequency to identify any conditions that might indicate changes in dam performance and, therefore, require follow-up. Engineers may carry out these inspections or other staff trained in dam inspection. Of particular significance are new occurrences or noted changes in debris and ice build-up, leaks, erosion, sinkholes, boils, seepage, slope slumping or sliding, settlement, displacement or cracking of structural components, concrete deterioration, clogging of drains and relief wells, etc.
- Engineering Inspections – Engineering inspections are typically performed annually or semi-annually and include a detailed visual examination of the dam and the instrumentation used to monitor the dam's performance. The requirements for the annual engineering inspections include the following:
    - Thorough visual inspection of the earth embankments, geotechnical features, concrete structures, and associated instruments (i.e. water gauges, piezometers, survey monuments, etc.).
    - Documentation of observations regarding the condition of the dam, highlighting deficiencies and any significant condition changes from previous inspections. Some assessment of the severity of observed anomalies as well as recommendations for maintenance, repairs, investigation, or further monitoring should be completed.

  Because of the additional level of assessment, professional engineers with expertise with dams and dam safety generally perform these inspections.

- Special Inspections – Special inspections should be completed during, if possible, and after unusual or extreme events such as heavy rainfall, flooding, windstorms, severe icing, rapid snowmelt, earthquakes, and exceedance of the absolute maximum operating water level. Significant changes to normal operations, nearby construction activity, or other unusual events might also trigger special inspections. Qualified staff should perform these inspections and defects should be reported to and assessed by a qualified professional engineer.

According to the 2023 Dam Operation and Maintenance Manual, the Deer Creek Dam is inspected monthly following a formal schedule. Any deficiencies, hazards or vandalism are documented on inspection forms and any concerns or deficiencies are communicated to the Water Resources Analyst. These inspections would generally be considered to be Routine Inspections.

The Deer Creek Dam is also inspected annually following a formal schedule. The dam is inspected once per year in the fall. Any deficiencies or damage are recorded in an order for repairs to be planned the following year. These inspections would generally be considered to be Engineering Inspections.

During periods of heavy rain or runoff, inspections are conducted to assess the risk to the dam. These inspections are conducted as required, depending on weather conditions. Similar to the Routine Inspections, any concerns are documented on inspection forms and any concerns or deficiencies are communicated to the Water Resources Analyst. These inspections would generally be considered Special Inspections.

6.2 Operation, Maintenance, Surveillance and Safety

The Operation, Maintenance, Surveillance and Safety (OMSS) Manual is a means to provide both experienced and new staff with the information they need to support the safe operation of the dam. The LPRCA provided the following OMS information for the Deer Creek Dam:

- Dam Operation and Maintenance Manual, 2023.

Wills has reviewed the information contained within the document and it contains some of the information outlined in the 2007 CDA Dam Safety Guidelines (CDA, 2013); however, the following deficiencies have been identified:

- There is no title page for the document that identifies the author, the revision number or the date of the OMSS Manual.
- There is no section that lists the revisions that have been completed or when revision were completed.
- There is no section that provides a summary of key dam information, including the purpose and history of the dam, dam ownership, a detailed structure description, site location, access and parking, significant site modifications, and the consequences of dam failure.
- There is no separate section that outlines the operation procedures for the dam. This would include the following:
    - Roles and Responsibilities.
    - Water Management.
    - Operating Procedures (normal operations, flood or drought operations, unusual operations, emergency operations and records[logs]. This section should also include required equipment, tools and personnel).
    - Flow control (equipment and facilities, water level gauge systems, supervisory control systems, emergency systems).
- Emergency staff contacts are not included.
- The maintenance section should be updated to include the following:

    - Maintenance equipment and tools.
    - Descriptions of routine and general maintenance, major maintenance and emergency maintenance.
- There is no separate section that outlines the safety considerations when operating or maintaining the dam. This would include details of operator safety and public safety.
- There is no separate section that outlines the requirements for record keeping.
- There is no section that describes the administrative procedures for the OMSS Manual. This would include the requirements for staff training, annual review, and the procedure for revisions.

The 2023 Dam Operation and Maintenance Manual contains information for multiple dams/structures operated by LPRCA. A new, Deer Creek Dam specific OMSS Manual should be prepared based on the guidelines listed in the Dam Safety Guidelines (CDA, 2013), incorporating the information in the existing 2023 Dam Operation and Maintenance Manual and addressing the deficiencies described above.

6.3 Emergency Preparedness

Dam owners are responsible for the safe management and operation of the dam, the protection of the public and the environment from the effects of dam failure and for exercising due diligence at all stages of the dam's life cycle. The MNR Technical Bulletins and Best Management Practices do not provide specific guidance as to when an Emergency Preparedness Plan (EPP) or Emergency Preparedness and Response Plan (EPRP) would be recommended and there are no laws in Ontario requiring one be prepared. However, the Dam Safety Guidelines (CDA, 2007) suggest that EPPs/EPRPs be in place for all dams where lives are at risk or if the implementation of emergency procedures could significantly reduce the consequences of dam failure.

No EPP or EPRP currently exists for the Deer Creek Dam. An EPRP should be prepared in accordance with the Technical Bulletin: Emergency Management for Dam Safety (CDA, 2019). The EPRP should include the following information:

- Title Page and Table of Contents.
- Description of the Purpose and Scope of the Document.
- Dam Site Data.
- Overview of the Effects of Inundation.
- Roles and Responsibilities of the Parties Involved in an Emergency Response.
- Emergency Response Process and Procedures:
    - Identifying, Verifying, and Classifying and Emergency.
    - Notifications and Communications, including updating the notification flow chart and list of contacts.
    - Response and Remediation Measures.

    - Termination and Recovery.
- Maintenance, Training, and Testing of the EPRP.
- List of Plan Holders.
- Appendices, including Drawings, Stage-Discharge Curves, Inundation Maps, Emergency Response Measures, Forms and Reports, and Historical Flooding Information.

The Technical Bulletin: Emergency Management for Dam Safety (CDA, 2019) provides a typical table of contents for a dam emergency plan. The LPRCA should review this document and ensure that the EPRP meets the CDA's guidelines.

7.0 Public and Operator Safety

7.1 Public Safety

Dams, and their associated structures and operational practices, present a number of potential hazards to the public. Protecting the public from these potential hazards is an important element of a dam owner's due diligence. Public safety should be considered throughout all stages of a dam's life cycle, from design to decommissioning; however, this is most important during the operational phase of the project.

In Ontario, public safety around dams is managed in accordance with the Best Management Practices for Public Safety Around Dams (MNR, 2011) and the Guidelines for Public Safety Around Dams (CDA, 2011). This document recommends that dam owners follow a managed system approach to public safety around dams. An important part of the managed system approach is the preparation of a Public Safety Risk Assessment (PSRA). The PSRA helps dam owners identify the potential hazards for the known activities undertaken by the public on and around the dam site in order to determine the most effective public safety (control) measures. The Best Management Practices for Public Safety Around Dams (MNR, 2011) does not specify a process for the PSRA, therefore the Public Safety Risk Assessment process outlined in the Guidelines for Public Safety Around Dams (CDA, 2011) is used.

7.1.1 Risk Assessment Methodology

The PSRA was completed in accordance with the Guidelines for Public Safety Around Dams (CDA, 2011). To carry out the PSRA, the following Risk Evaluation Criteria have been established:

- Incident Likelihood Rating (ILR): Table 36 identifies the criteria for this rating.
- Incident Consequence Rating (ICR): Table 37 identifies the criteria for this rating.
- Risk Rating (RR) and Risk Level (RL): Table 38 identifies the RL as Low, Medium and High based on the RR. The RR is based on the likelihood (occurrence of people in the hazard area) multiplied by the consequence (most likely outcome when people are exposed to the hazard).

Table 36 – Incident Likelihood Rating Table (CDA, 2011)

Descriptor	Definition of Likelihood	Incident Likelihood Rating (ILR)
Very Frequent	More than 10 occurrences in the hazardous area in any one of the last 3 years or 25 or more occurrences in total in the last 3 years.	5
Frequent	More than 2 occurrences in the hazardous area in any of the last 3 years.	4
Occasional	Any occurrence in the hazardous area in the last 6 years.	3
Possible	Any occurrence in the hazardous area in the last 10 years.	2
Remote	No known occurrences in the last 10 years.	1

Table 37 – Incident Consequence Rating Table (CDA, 2011)

Descriptor	Definition of Consequences	Incident Consequence Rating (ICR)
Fatality	Fatality	5
Critical	Permanent Partial or Total Disability	4
Major	Medical Treatment or Stranding (rescue required)	3
Minor	First Aid or Stranding (self-rescue possible)	2
Insignificant	No first aid required	1

Since accurate monitoring of public visitation was not available, the number of public members in the area and within the hazards during the year can only be estimated based on anecdotal evidence and discussion with LPRCA staff. The dam site is near the Deer Creek Conservation Area; therefore, it can be assumed that members of the public frequently access the dam site.

The risk assessment methodology produces results that can vary between 1 and 25. The proposed risk tolerance levels are shown in the table below with "High" risk levels in the 11 – 25 range; "Medium" in the 5 – 10 range; and "Low" in the 1 – 4 range.

Table 38 – Risk Rating and Risk Level

Risk Rating and Risk Level			Incident Consequences				
			Insignificant	Minor	Major	Critical	Fatality
			1	2	3	4	5
Incident Likelihood	Very Frequent	5	Medium (5)	Medium (10)	High (15)	High (20)	High (25)
	Frequent	4	Low (4)	Medium (8)	High (12)	High (16)	High (20)
	Occasional	3	Low (3)	Medium (6)	Medium (9)	High (12)	High (15)
	Possible	2	Low (2)	Low (4)	Medium (6)	Medium (8)	Medium (10)
	Remote	1	Low (1)	Low (2)	Low (3)	Low (4)	Medium (5)

Risk Level	
High	11 to 25
Medium	5 to 10
Low	1 to 4

The Risk Levels identified above are similar to what has been adopted by the MNR and other Conservation Authorities for use in Public Safety Risk Assessments.

7.1.2 Component Areas

For the purposes of the PSRA, the site was broken down into three different component areas. The component areas include:

- Headpond – The Headpond component area boundary extends from the upstream side of the dam to a distance of approximately 25 m upstream of the dam and encompasses the shoreline to a distance of 2 m back from the normal (summer) water's edge, and an area approximately 10 m in diameter around the intake of the low flow valve.
- Structure – The Structure component area boundary includes the concrete control structure, the two earth embankment sections, the concrete spillway, and the low flow spillway.
- Tailrace – The Tailrace component area extends from the downstream side of the dam to a distance of approximately 70 m downstream.

A separate Risk Assessment Worksheet was completed for each component area.

7.1.3 Activities and Hazards

Based on our review of the Dam and Public Safety Operations Questionnaire completed by LPRCA staff and discussions with LPRCA staff on site, Wills has identified the activities and potential hazards listed in Table 39 for each component area.

Table 39 – Activities and Hazards

Activity Descriptions	Potential Hazards
Headpond	
From Water / Ice	
• Fishing from Boat • Boating • Canoeing / Kayaking / Rowing • Swimming	• Strong currents or undertows • Presence of spillway with stoplogs • Presence of discharge valve/pipe • Floating debris
From Shore/Structure	
• Fishing from Shore • Walking • Hiking	• Steep or slippery banks • Open holes or tripping
Structure	
From Shore/Structure	
• Fishing • Walking • Climbing • Driving • Biking	• Strong currents or undertows • Presence of spillway with stoplogs • Presence of overflow spillway • Falling from height > 3 metres • Slippery surfaces
Tailrace	
From Shore	
• Fishing from Shore • Walking • Hiking	• Steep or slippery banks • Open holes or tripping
From Water	
• Wading	• Strong currents or undertows • Presence of spillway with stoplogs • Presence of overflow spillway or dam • Presence of discharge valve/pipe • Floating debris • Slippery surfaces

7.1.4 Existing Public Safety Measures

The existing public safety measures include:

- A public safety boom (though inadequate).
- Public safety signs, as follows:
    - "Danger, No Trespassing, access Beyond This Point May Result in Drowning" signs on the left and right sides of the concrete structure, facing upstream.
    - "Danger, No Trespassing, access Beyond This Point May Result in Drowning" sign on the top of the low flow valve structure, facing downstream.
    - "Danger, Dam Ahead, Keep Out" signs on the left and right sides of the concrete structure, facing left and right.
    - "Danger, Keep Off, Dam" sign on the fencing on the right side of the dam, facing downstream (hard to read due to rusting).
    - Two "No Trespassing" signs on the fencing on the right side of the dam, facing downstream.
    - "Danger, No Trespassing" sign on the fencing on the left side of the embankment, facing right.
- Chain Link Fencing on tops of concrete abutments/wingwalls and along the concrete spillway.
- Metal grates over the culverts built into the concrete structure on the left and right sides of the dam to restrict access.
- Railings along roadway bridge over the dam.
- Vehicular guardrails along the left and right embankments on the upstream and downstream sides of the dam crest.
- Locked access gate and door of the lift control chamber.

The existing public safety measures were considered in the PSRA.

7.1.5 Risk Assessment Results

The results of the PSRA are provided in Table 40. There are four low hazard activities, ten medium hazard activities and one high hazard activity. The risk rating values are driven primarily by the ILR as the site is favoured by many for recreation.

Table 40 – PSRA Results

Activity	ILR	ICR	RR	RL
Headpond				
From Water / Ice				
Fishing from Boat	3	2	6	Medium
Boating (under power)	3	2	6	Medium
Canoeing/Kayaking/Rowing	3	3	9	Medium
Swimming	3	3	9	Medium
From Shore / Structure				
Fishing from Shore	1	2	2	Low
Walking	1	2	2	Low
Hiking	1	2	2	Low
Structure				
From Structure				
Fishing from Shore	2	2	4	Low
Walking	4	2	8	Medium
Climbing	3	4	12	High
Driving	5	1	5	Medium
Biking	4	1	4	Low
Tailrace				
From Shore				
Fishing from Shore	3	2	6	Medium
Walking	3	2	6	Medium
Hiking	2	3	6	Medium
From Water				
Wading	1	2	2	Medium

The installation of additional public safety measures such as a new public safety boom, public safety buoy, public safety signage, and establishing operational procedures (i.e. not operating the low flow valve if members of the public are in the Headpond) is likely to reduce the Risk Level (i.e. Medium to Low) for the majority of the identified activities

with the exception of walking and driving on the Structure. Driving and walking on the Structure have been assigned a risk level of Medium. Driving and walking on the Structure would remain a Medium risk due to a high ILR. It is anticipated that driving and walking across the Structure would occur in most circumstances when members of the public access the site due to the presence of designated roadway crossing the dam structure.

Climbing on the Structure has a Risk Level of High. It is anticipated that the addition of public safety signage on the downstream side of the dam, at the end of the spillway and fencing with public safety signage at the low flow outlet may deter some people from climbing in this area. This would lower the ILR, likely resulting in the Risk Level moving from High to Low for this activity.

A private hazard buoy installed at the inlet of the low flow gate would help delineate the potentially dangerous and identify the potential hazard posed by the gate structure. This is also anticipated to reduce the Risk Levels of some of the Medium risk activities in the Headpond to Low, and help ensure that the public are adequately warned about the potential hazards (LPRCA due diligence).

7.2 Operator Safety

Operator safety measures are regulated under the Occupational Health and Safety Act (OHSA). The OHSA and its associated regulations are used to assess the adequacy of operator safety measures. For the majority of dam sites, there are two primary operator safety measures, railings and fall protection, the requirements for which depend on specific site conditions.

The Industrial Establishments Regulation of the OHSA (O.Reg. 851) requires a guardrail at the open side of any raised surface. The guard rail must have a top rail located not less than 910 mm and not more than 1070 mm above the surface to be guarded, have a mid rail, have a toe-board that extents at least 125 mm from the surface if tools or other objects may fall on other workers below, be free of splinters and protruding nails and be constructed to meet the structural requirements for guards as set out in the Ontario Building Code. The existing railings on the dam deck generally meet the requirements of O. Reg. 851.

Although the stoplogs are reportedly not operated at the dam site, an operation procedure and health and safety requirements should be developed for removal/replacement of the stoplogs.

An additional operator safety concern would be working from a boat when performing maintenance (i.e., removing debris). While working around water, LPRCA staff would need to follow all applicable LPRCA Health and Safety Policies, including any provisions for the use of personal protective equipment (i.e., life jackets or PFDs)

8.0 Deterioration Locations and Rehabilitation Drawings

Wills has prepared drawings identifying the concrete deterioration throughout the structure based on the site inspection. These drawings along with preliminary rehabilitation drawings can be found in Appendix G. Additionally, cost estimates for rehabilitation and full replacement of the dam can be found in Appendix G.

9.0 Conclusions

9.1 Hydrotechnical

- The Sunny Day peak flow for the Deer Creek Dam was assumed to be 1.0 m^3/s. This is larger than the estimated baseflow of 0.16 m^3/s; however, it is the minimum flow that Wills typically uses for the Sunny Day non-breach condition in order to ensure stable model runs.
- The Sunny Day peak flow for Big Creek was assumed to be 4.0 m^3/s based on a review of flow gauge data for the Deer Creek downstream of the dam.
- The Deer Creek Dam is classified as a High hazard dam for normal (sunny day) dam breach scenario. The maximum potential incremental property damages are anticipated to be less than $300,000 and there is an anticipated incremental loss of life between 1-10 persons.
- The Deer Creek Dam is classified as a High hazard dam for the flood dam breach scenario. The maximum potential incremental property damages are anticipated to be greater than $300,000, but less then $3,000,000 and there is an anticipated incremental loss of life between 1-10 persons.
- The recommended Inflow Design Flood (IDF) for the Deer Creek Dam is the 1/3 between 1000-year flood and PMF, which is estimated as 34.3 m^3/s.
- The headwater elevation for the IDF (34.3 m^3/s) is 201.21 m for the Deer Creek Dam as estimated using the unsteady-flow HEC-RAS model.
- The Deer Creek Dam has sufficient hydraulic capacity to convey the IDF without overtopping.
- The Deer Creek Dam has sufficient freeboard for the normal operating condition.
- The Deer Creek Dam has sufficient freeboard for the IDF condition.

9.2 Civil/Structural

9.2.1 Condition Assessment

- The downstream left earth embankment is in generally good condition with no noted locations of seepage or erosion.
- The upstream left earth embankment surface is protected with rock protection that was in generally good condition; however, some rip-rap is cracking/disintegrating and is not uniformly placed. The sloughing/settlement

that was noted during the previous inspection in 2014 does not appear to have gotten worse. Minor amounts of vegetation were found to be growing between the riprap on the downstream embankment.

- The right earth embankment is in good condition and well vegetated.
- The bypass outlet structure is generally in good to fair condition. However, there is a large area of spalling, delamination, and disintegration occurring at the interface with the angled face of the outlet wall. Severe map cracking with efflorescence is present on both sides of the structure.
- There is ±900 mm of sedimentation at the outlet structure. The baffle blocks / energy dissipators and toe drain outlets are completely buried.
- The left and right upstream wingwalls appear to be in good condition.
- The left and right bridge abutment concrete is in good condition below bearing seat. Erosion at the base of the wall is opening a gap between the abutment wall and spillway slab.
- The walls on either side of the upper spillway slab have been partially refaced and patched. The newer concrete is in good condition, except for a small delamination near the base of the right wall. The exposed original walls are in good condition with localized deterioration in the form of cracks with efflorescence and delamination. The culvert outlet on the right side of the spillway is delaminated around its perimeter as concrete appears to be starting to disintegrate.
- The upper spillway slab is in good condition with light erosion where the original concrete is visible. The majority of the slab concrete is covered with an organic deposit that resembles disintegrated concrete with small aggregates.
- The slab of the middle section of spillway is largely covered in a 20-50 mm thick layer of weak concrete-like material, assumed to be organic, and is superficial to the structural concrete of the underlying slab. Where exposed, the original concrete of the slab appears to be in good condition and is sound to hammer impact. Transverse construction joints are present in the spillway slab and walls at ±7.47 m (24.5') intervals. Deterioration of the walls and slab of the spillway appear to be localized to the construction joint locations. Several wall joints are fully delaminated with some spalls and disintegration of underlying concrete. A PVC waterstop is exposed between slabs 6 and 7 near the interface with the left wall. In general, the construction joints in the slabs appear to be in good condition and functioning well. A review of the embankment-side face of the wall at construction joint locations reveals that several disintegration patterns, and associated delaminations, appear to be reflected on both sides of the wall, full depth spalling is likely to occur at these locations in the future.
- The lowest portion of the spillway (stilling basin) is mostly covered in silt, vegetation, and debris (±500 thick) and could not be inspected due to limited exposure / visibility.
- Inspection of the subdrain system was limited to what was visible from the base of the spillway chute. The 8" perforated CSP subdrain on the left spillway chute

wall had moderate corrosion with section loss at the bottom of the pipe. The 8" perforated CSP subdrain on the right spillway chute wall is in very poor condition and is disjointed/crushed with coarse grained material entering the pipe as well as moderate corrosion. Course grained material upstream of the disjointed area indicates this pipe may be damaged in more than one location.

- The chain link fencing appears to be in good condition
- The top stoplogs are in poor condition with severe rot and decay. The same concrete like organic material that is coating the spillway is present on the downstream side of the stoplogs. Removing a small patch of this material showed the stoplogs to be in fair condition, but inspection was limited. There is minor leakage between the stoplogs.
- The valve system was operated by LPRCA staff during the inspection (25 rotations opened, 25 rotations closed) and appears to be in working condition with significant sediment discharge at the outlet when opened. The paint is peeling on the valve system and light corrosion is visible. The valve stem exits the lift chamber and appears to be in good condition. Inspection of the slide gate was not possible as it was submerged and potentially buried under sediment/organic material.
- The existing public safety signage appears to generally be in good condition.

9.2.2 Stability Assessment

- Given the High HPC for the sunny day dam breach scenario, the Design Earthquake was selected as the 2500-year earthquake.
- The results of the stability analysis indicate that the dam meets the minimum required factors of safety for all loading conditions.

9.3 Geotechnical

- The present slope stability findings are based on a high-level desktop analysis using pre-construction borehole data and preliminary soil parameters. These results should not be viewed as final factors of safety due to limited, dated, or unverified data.
- The results of Cambium's stability analysis indicate that the minimum factors of safety are anticipated to be met on the upstream and downstream slopes under normal operating conditions, psuedostatic conditions, and under IDF conditions.
- The upstream slope does not meet LRIA criteria under rapid drawdown conditions. Calculations indicate a potential for slope failure if this were to occur.
- Instability/sloughing noted on the upstream slope may be linked to past drawdown events.

9.4 Dam Safety Management

- Engineering Inspections are well documented and are carried out with a consistent frequency (i.e. annually).
- Routine and Special Dam Inspections are conducted by the LPRCA following a formal schedule (Routine Inspections, monthly) and as required depending on weather conditions (Special Dam Inspections).
- There is no stand alone OMSS manual for the Deer Creek Dam, rather, the Dam Operation and Maintenance Manual (LPRCA, 2023) covers all dams owned and operated by the LPRCA. The manual covers the main contents recommended within the Dam Safety Guidelines (CDA, 2007); however it does not include sufficient detail in some sections.
- There is no EPP or EPRP for the Deer Creek Dam.

9.5 Public and Operator Safety

- There are a number of public safety measures already in place, including signage, fencing, railings, guardrails, and a public safety boom. However, some of these measures are deficient and require remediation to align with current best practices and standards.
- The existing public safety boom does not conform to the Canadian Dam Association's Technical Bulletin: Booms and Buoys for Public Safety Around Dams (2011). The current boom configuration lacks features that promote self-rescue and are less then 60cm in length.
- Wills undertook a Public Safety Risk Assessment for the dam site and identified four low hazard activities, ten medium hazard activities, and one high hazard activity.
- Climbing on the Structure has a Risk Level of High. The addition of public safety signage on the downstream side of the dam and at the ends of the concrete spillway, and railings around the low flow outlet may deter some people from climbing in this area, reducing the Incident Likelihood Rating, likely resulting in the Risk Level moving from High to Low.
- A private hazard buoy installed at the inlet of the low flow fate would help delineate the potentially dangerous and identify the potential hazard posed by the gate structure. This is also anticipated to reduce the Risk Levels of some of the Medium risk activities in the Headpond to Low, and help ensure that the public are adequately warned about the potential hazards (LPRCA due diligence).
- There were no major operator safety issues identified during the dam inspection.
- Although the stoplogs are reportedly not operated at the dam site, an operation procedure and health and safety requirements should be developed for removal/replacement of the stoplogs.

10.0 Recommendations

Recommendations based on the conclusions presented above are provided below, by discipline. Section 10.6 summarizes the recommendations and provides budget level cost estimates and the prioritization for each recommendation.

10.1 Hydrotechnical

There are no recommendations related to the hydrotechnical aspects of the dam.

10.2 Civil / Structural

10.2.1 Condition Assessment

- Conduct full cleaning of spillway surface to expose original concrete for inspection.
- A CCTV inspection of the 36" concrete bypass pipe, toe drains and subdrains should be completed within the next 2 years. The extent of damage to the 8" CSP subdrains should be investigated, and a rehabilitation plan should be developed based on the results of the CCTV Inspection. Clearing of sediment and possibly dewatering in the bypass stilling basin may need to be completed prior to inspection to access the toe drains.
- The paint should be removed from the peeling section on the valve and repainted to prevent further corrosion within the next 2 years.
- Pending findings of enhanced inspection, plan and carry out rehabilitation of concrete structure(s) within 6-10 years.
- Continue to clear vegetation on the upstream and downstream embankments.
- Clear and grub vegetation from the left upstream embankment.
- Remove hard film on stoplogs to determine condition and replace as required.

10.2.2 Stability Assessment

- The water table level in the backfill behind the retaining walls should not exceed the IDF elevation of 201.21 m.

10.3 Geotechnical

- Confirm embankment composition by advancing boreholes through the dam crest (and from the toe berm on the downstream side, if possible). Boreholes advanced through the dam crest should capture the transition between embankment fill and native soils for accurate seismic assessment. Install monitoring wells in these boreholes to gather water level data that can refine slope stability models and provide long term monitoring.
- As a High Hazard Potential Classification dam, more detailed pseudostatic and liquefaction analyses are advisable. Foundation soils, described as sand-

dominant and potentially saturated, should be sampled and tested (e.g., particle size distribution) to evaluate liquefaction susceptibility.

- Further review of operational records is recommended to see if specific drawdown actions correlate with observed instability.
- Regrade any unstable areas to achieve a shallower, stable slope inclination (flatter than the current 3H:1V) determined through updated slope stability analysis. This work should be supervised by a qualified engineer.
- Restore rip-rap protection to a consistent thickness and placement across the entire upstream slope.
- If the reservoir needs to be drawn down for any reason, this should be done slowly and under the supervision of a qualified engineer.

10.4 Dam Safety Management

- Update the Dam Operation and Maintenance Manual (LPRCA, 2023). The updated manual should be prepared in accordance with guidance outlined in the Dam Safety Guidelines (CDA, 2011) for Operation, Maintenance, Safety and Surveillance (OMSS) Manuals.
- Develop and implement an Emergency Preparedness and Response Plan (EPRP). The EPRP should be prepared in accordance with the guidance outlined in the Technical Bulletin: Emergency Management for Dam Safety (CDA, 2019).

10.5 Operator and Public Safety

- Install a new public safety boom and consider installing one private hazard buoy at the inlet of the low flow gate as this would help delineate the potentially dangerous area around the low flow inlet structure.
- Install large public safety signs on the upstream and downstream side of the dam.
- Install small public safety signs on the ends of the concrete spillway.
- Install railings around the low flow outlet and install a public safety sign on the railings.
- Establish operator procedures for not operating the dam if members of the public are present in the Headpond, and for removal/replacement of the stoplogs.

10.6 Summary, Cost Estimates and Prioritization

Table 41 summarizes the deficiencies/issues, recommendations, priority, and cost estimates associated with the recommended remedial measures. The degree of accuracy for the estimates is approximately +/-50%.

The priorities are classified as “Immediate”, “High”, “Medium” and “Low” and are defined as follows:

- Immediate – Remedial action that needs to be carried out as soon as possible because the deficiency is an immediate high-risk dam safety hazard with a high likelihood of occurrence of loss of life and/or serious environment and/or serious economic consequences.
- High – Remedial action is required within the next 2 years to meet current regulations and/or dam safety requirements and is a high-risk dam safety hazard.
- Medium – These items may include additional work that could improve the performance or issues that may become serious dam deficiencies. These items typically should be addressed within five years.
- Low – These are opportunities to improve safety or only in the long term may become a serious dam safety deficiency. The recommended remedial action is expected to be required only at least six years from now.
- Ongoing – These are items where monitoring by LPRCA staff should continue during the routine and annual (engineering) inspections.

The recommendations are prioritized based on the risk of occurrence, significance of potential negative impacts and resources (cost, time, and effort) required to implement.

Table 41 – Summary of Recommendations and Costs

Structure/Item	Deficiency	Recommendation	Quantity	Estimated Cost	Remarks	Priority
Civil / Structural						
Concrete Structure	The spillway has a buildup of what looks like organic material that limits inspection of the concrete.	1. Conduct full cleaning of spillway surface to expose original concrete for inspection		$20,000		Medium
Conduits	The subdrains are in poor condition in sections and the toe drains and bypass pipe have not been inspected to Wills Knowledge.	2. A CTV inspection of the 36" concrete bypass pipe, toe drains and subdrains should be completed within the next 2 years. The extent of damage to the 8" CSP subdrains should be investigated, and a rehabilitation plan should be developed based on the results of the CTV Inspection. Clearing of sediment and possibly dewatering in the bypass stilling basin may need to be completed prior to inspection to access the toe drains		$16,000	This price assumes LPRCA would facilitate the dewatering of the bypass outlet, and the results would be reviewed by a qualified engineer.	High
Valve	Paint is peeling off the valve casing and stem increasing the risk of corrosion.	3. The paint should be removed from the peeling section on the valve and repainted to prevent further corrosion.		$1000		Medium
Spillway Chute Repairs	The spillway chute has several concrete deficiencies.	4. Pending findings of enhanced inspection, plan and carry out rehabilitation of concrete structure(s) 6-10 years.		$260,960	This item is itemized in Appendix G.	Low
Spillway Chute Subdrain Replacement	The 8" spillway subdrains were noticeably damaged and had severe corrosion	5. Pending findings of enhanced inspection, plan and carry out rehabilitation of subdrains 3-5 years.		$313,550	This item is itemized in Appendix G. This pricing assumes replacement of both 8" CSP subdrains. It does not include replacement of subdrains under the concrete slabs.	Medium
Bypass Outlet Repairs	The bypass pipe outlet structure has several concrete deficiencies.	6. Plan and carry out rehabilitation of concrete structure(s) 6-10 years.		$134,580	This item is itemized in Appendix G.	Low
Left Upstream Embankment	The dam has vegetation on the upstream embankment.	7. Clear and grub vegetation from the left upstream embankment.		$1000		High
Replace Stoplogs	The top stoplogs had severe rot/decay and should be replaced.	8. Stoplog		$500/log	The lower stoplogs conditions should be assessed after removing hard film to determine if these should be replaced as well.	Medium

Structure/Item	Deficiency	Recommendation	Quantity	Estimated Cost	Remarks	Priority
Geotechnical						
Left Embankment	The dam does not have geotechnical information (grain size, and soil mechanical properties) for the as constructed embankment materials or monitoring wells.	9. Confirm embankment composition by advancing boreholes through the dam crest (and from the toe berm on the downstream side, if possible). Boreholes advanced through the dam crest should capture the transition between embankment fill and native soils for accurate seismic assessment. Install monitoring wells in these boreholes to gather water level data that can refine slope stability models and provide long term monitoring.		$22,000	The preliminary analysis indicates the dam meets the factors of safety. Therefore, this work could likely wait until the next dam safety review provided no slope stabilities issues arise before then.	Low
Left Embankment	A detailed pseudostatic and liquefaction analysis may be warranted given the High HPC of the dam.	10. As a High HPC dam, more detailed pseudostatic and liquefaction analyses are advisable. Foundation soils, described as sand-dominant and potentially saturated, should be sampled and tested (e.g., particle size distribution) to evaluate liquefaction susceptibility.		$15,000	The preliminary analysis indicates the dam meets the factors of safety. Therefore, this work could likely wait until the next dam safety review provided no slope stabilities issues arise before then.	Low
Left Upstream Embankment	The cause of the upstream embankment instability is currently unknown; however, it may have been caused by a rapid drawdown.	11. Further review of operational records is recommended to see if specific drawdown actions correlate with observed instability.		N/A	Assumed this could be completed internally by LPRCA staff.	Medium
Left Upstream Embankment	Non uniform grade on the left upstream embankment	12. Regrade any unstable areas to achieve a shallower, stable slope inclination (flatter than the current 3H:1V) determined through updated slope stability analysis. This work should be supervised by a qualified engineer.		$60,000	Priority should be increased if the slope shows further instability.	Low
Riprap on upstream embankment	Riprap size and thickness is non uniform across the dam embankment.	13. Restore rip rap protection to a consistent thickness and placement across the entire upstream slope.		$100,000	Priority should be increased if the slope experiences sign of erosion.	Low
Left Upstream Embankment	The preliminary analysis indicates that the dam does not meet the required factor of safety for the rapid drawdown loading case.	14. If the reservoir needs to be drawn down for any reason, this should be done slowly and under the supervision of a qualified engineer.		N/A	If near immediate or unplanned rapid drawdown is anticipated, begin slope remediation measures as soon as possible to prevent successive failures.	Ongoing

Structure/Item	Deficiency	Recommendation	Quantity	Estimated Cost	Remarks	Priority
Dam Safety Management						
Dam Operation and Maintenance Manual	The Dam Operation and Maintenance Manual is missing information	15. Update the Dam Operation and Maintenance Manual (LPRCA, 2023). The updated manual should be prepared in accordance with guidance outlined in the Dam Safety Guidelines (CDA, 2011) for Operation, Maintenance, Safety and Surveillance (OMSS) Manuals.		N/A	Assumed this could be completed internally by LPRCA staff.	High
Emergency Preparedness and Response Plan	The dam does not have an Emergency Preparedness and Response Plan	16. Consider the development and implementation of an Emergency Preparedness and Response Plan (EPRP). The EPRP should be prepared in accordance with the guidance outlined in the Technical Bulletin: Emergency Management for Dam Safety (CDA, 2019).		N/A	Assumed this could be completed internally by LPRCA staff.	High
Public and Operator Safety						
Public Safety buoy	Public Safety buoy is in adequate and should be replaced with a safety boom.	17. Install a new public safety boom and consider installing one private hazard buoys at the inlet of the low flow fate would help delineate the potentially dangerous and identify the potential hazard posed by the gate structure.		$90,000	Assumes a concrete block in water anchor and utilizing the existing anchors on the wing walls.	Medium
Public Safety Signage	Additional Public Safety Signage	18. Install large public safety signs on the upstream and downstream side of the dam.		$1,000	Assumed this could be completed internally by LPRCA staff.	High
Public Safety Signage	Additional Public Safety Signage	19. Install small public safety signs on the ends of the concrete spillway		$500		High
Public Safety Railing and Signage	Missing Public Safety Railing and Signage	20. Install railings around the low flow outlet and install a public safety sign on the railings.		$8,000		Medium
OMSS Manual update	OMSS Manual is missing information	21. Establish operator procedures for not operating the dam if members of the public are present in the Headpond, and for removal/replacement of the stoplogs		N/A	Assumed this could be completed internally by LPRCA staff.	High

11.0 Closure

We appreciate the opportunity to provide Professional Engineering Services to the LPRCA. Should you have any questions regarding the information contained herein, please do not hesitate to contact the undersigned.

Respectfully submitted,



Matt Churly, P.Eng.
Senior Hydrotechnical Engineer



Mohammad Gilani, P.Eng., PMP
Senior Structural Engineer



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Project Engineer (Dam Safety)



Tim Rosborough, P.Eng.
Structural Engineer



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MC/SO/MG/dg

Appendix A

Site Inspection

Introduction

The purpose of this questionnaire is to provide Wills with basic information regarding the dam and its operation, including the identification of any operation, maintenance, public safety, or operator safety concerns. The questionnaire is broken down into the following sections:

- Part 1 – General Information
- Part 2 – Discharge Facilities and Equipment
- Part 3 – Dam Surveillance
- Part 4 – Dam Operation
- Part 5 – Construction, Maintenance, and Repairs
- Part 6 – Public Safety Around Dams
- Part 7 – Additional Comments/Concerns

If there are any questions regarding this questionnaire, please contact Wills.

Wills' Contact Person		
Name	Matt Churly	
Contact Information	647-244-8754	mchurly@dmwills.com

Part 1 – General Information

Field	**Data**	
Basic Site Information		
Dam Name	Deer Creek Dam	
Owner	Long Point Region Conservation Authority	
MNRF District	Alymer	
Respondent Contact Information		
Name	Matt Churly (on behalf of LPRCA)	
Organization	D.M Wills	
Contact Information	647-244-8754	mchurly@dmwills.com

Part 2 – Discharge Facilities and Equipment

Field	Data
Discharge Facility Information	
Stoplog Sluiceways ☐ N/A	
No. of Stoplog Sluiceways	2
No. Stoplogs per Sluiceway	11
Stoplog Height(s)	150 mm
Gated Sluiceways ☒ N/A	
No. of Gated Sluiceways	
Gated Sluiceway Width(s)	
Gate Height(s)	
Overflow Sections ☐ N/A	
Type of Overflow Section	Ogee Spillway
Presence of Flashboards	☐ Yes ☒ No ☐ Seasonal Installation Only
Flashboard Height(s)	N/A
Do the flashboards automatically collapse on high water levels?	N/A
Low Level Outlet(s) ☐ N/A	
Type, Size and Material of Valve	Slide gate, Armco Valve
Size and Material of Outlet Pipe	914.4 mm (36") Concrete Pipe
Stoplog/Gate Lifting Equipment	
Type(s) of Equipment	N/A
Quantity	N/A
Manufacturer(s)	N/A
Rated Capacity(ies)	N/A

Field	Data	
Monitoring Equipment		
Rain Gauge Information		
Rain Gauge Name / ID	Delhi CS	
Rain Gauge Data Source	EC	
Flow Gauge Information		
Flow Gauge Name / ID	N/A	
Flow Gauge Data Source	N/A	
Headwater Level Gauge Information ☒ N/A		
Type of Gauge	☐ Manual Staff Gauge	☐ Automated Gauge
Gauge Units	☐ Metric (i.e. cm, m)	☐ Imperial (i.e. in, ft)
Vertical Reference System	☐ Geodetic Benchmark	☐ Local Benchmark
Tailwater Level Gauge Information ☒ N/A		
Type of Gauge	☐ Manual Staff Gauge	☐ Automated Gauge
Gauge Units	☐ Metric (i.e. cm, m)	☐ Imperial (i.e. in, ft)
Vertical Reference System	☐ Geodetic Benchmark	☐ Local Benchmark
Piezometers ☒ N/A		
Type of Piezometer	☐ Manual	☐ Automated
Quantity and Location		
Gauge Units	☐ Metric (i.e. cm, m)	☐ Imperial (i.e. in, ft)
Vertical Reference System	☐ Geodetic Benchmark	☐ Local Benchmark

Part 3 – Dam Surveillance

How is the dam accessed at the various times of the year?
The dam is accessed via Norfolk County Road 45.
How frequently are dam inspections or site visits completed?
The dam is inspected by LPRCA staff once per month
Are Form B-1 and Form B-2 completed for each site visit?

No.
How/where are the dam inspection or site visit records stored?
In the LPRCA main office.
Are the records readily available for review?
No

Part 4 – Dam Operation

Who is responsible for dam operations? (i.e., owner, third party, etc.). Describe.
LPRCA
How often is the dam operated?
The low flow valve is operated semi-annually to flush accumulated sediment.
What is done during the operation of the dam?
The low flow valve is operated semi-annually to flush accumulated sediment.
What are the typical log/gate settings for the dam?
The stop logs are in the same setting year-round and are at the height of the weir.
Is there power / back-up power available at the dam and is power required to operate the various stoplog and gate lifting equipment?
No.
Is there an OMS Manual or Rule Curve for the dam? Is if followed for operations?
There is an OMS manual. The dam is not operated for flood control.
Is there an Emergency Preparedness Plan or Emergency Preparedness and Response Plan for the Dam? Are there any precipitation, water level or flow triggers that would initiate dam operations?
No.
If there is a low-level outlet, how often is the gate/valve exercised?
The valve is operated semi-annually.

Are there any issues with ice at the dam? What is the approximate period of ice cover on the reservoir?
No. The reservoir is frozen through the winter and thaws during the freshet.
Can the dam be safely operated during a high flow event? Can the full discharge capacity be achieved? (i.e., Can all of the stoplogs be removed during a flood?)
The valve could be operated if required. The spillway has capacity to discharge the PMF with the stop logs in place.
What are the operator health and safety practices and mitigation measures that are used during the operation of the dam? (i.e., fall arrest/restraint devices, working at heights rescue plan, other site-specific PPE)
N/A
Are there any operator health and safety concerns or recommendations for improvements?
No.
Have there been any incidents regarding worker safety at the dam?
No.
Are there any challenges operating this dam?
No.

Part 5 – Construction, Maintenance, and Repairs

Have there been any major changes to the dam recently or since its original construction?
Wingwalls were repaired in 2022
Has any work been completed since the last Scheduled Dam Inspection?
No.
Are there any seasonal changes experienced at the dam? (i.e., recreational use, etc.)
Yes, there is seasonal camping in the conservation area just upstream of the dam. The reservoir is frequently used for pleasure craft and angling during the spring to falls months.

Can you describe the routine maintenance activities that are completed at the dam?
Vegetation on the downstream side is removed every 1-2 years by a contractor. Vegetation is removed on the upstream side on an as needed basis.
Is debris or sedimentation build-up upstream an issue for the dam? Is there a routine debris removal program?
Woody debris accumulates on the weir and through the spillway chute. It is removed on an as needed basis. Sediment buildup over the low flow valve accumulates and needs to be flushed.

Part 6 – Public Safety Around Dams

Can you describe the public safety measures that are installed at the dam, their condition, and their effectiveness?
Chain-link fencing is installed along the spillway chute and is effective. Signage is present on several locations along the dam. It is noted that boaters often enter the reservoir on the far east side of the embankment even though signs are posted. There is a public safety buoy installed on the upstream side of the dam. To my knowledge no incidents have occurred with boaters going over the spillway.
What activities are members of the public undertaking at the dam site and at what approximate frequency are these activities bring undertaken?
Boating, and fishing are very frequent in the upstream reservoir.
How are records of public safety incidents recorded and stored?
On an as needed basis.
Have there been any public safety incidents at the dam? If yes, please describe.
Not to my knowledge.
Have any public safety issues been identified that have not been corrected/mitigated?
Not to my knowledge.

Part 7 – Additional Comments/Concerns

Do you have any additional comments/concerns regarding the dam or its operation?
No.

Form B1

Pre-Inspection Background Information

Prepared By:	Matt Churly
Name of Dam:	Deer Creek Dam
Latest Construction:	Built in 1969, Repairs made in 2022
Inspection Dates:	Riggs Engineering Inspection Report 2014
Access:	1929 Norfolk County Rd 45, Langton, ON N0E 1G0
Lake Controlled:	Deer Creek Reservoir
Lake Area:	27.48 ha
Watershed:	Deer Creek (Tributary to Big Creek)
Drainage Area:	15.13 km^2
Gauge Info:	N/A
Rule Curves:	N/A
List of Drawings:	1) 1969 Deer Creek Drawing Package – Woodstock Engineering Consultants Limited 2) 2019 Deer Creek Repairs – Vallee Consulting Engineers
Meteorological and Hydrological Data:	LPRCA Riverine Flood Hydrology Study
Topographic Maps:	LiDAR DTM
Soil and Land-Use Maps:	Land Cover 4DM, SOLRIS 3.0, MNRF Soils Complex
Dam Height:	~13.0 m
Dam Length:	160 m
No. of Sluiceways:	15.24 m Ogee Spillway Chute. 2 Sluices built into Chute (1.83 m)
No. of Stop Logs per Bay:	5 Stop Logs

Hydrologic Flows:	Unknown
Hydraulic Analysis:	Unknown
Dam Operation:	Low Flow Valve
Soils Reports:	MNRF Soils Complex
Underwater Inspections:	Unknown
Property Ownership:	LPRCA
Dam Maintenance:	Repairs Completed
Divestment Opportunities:	N/A
Known Problems:	Concrete deterioration and Alkali Silica Reaction on spillway chute, Low Flow Bunker, and low flow outlet. Change of slope noted on upstream side.

Summary of File

Long Point Region Conservation Authority

Form B2

Dam Inspection Form

Date:	October 29, 2024
Name of Dam:	Deer Creek Dam
Municipality:	Norfolk County
Location:	1929 Norfolk County Rd 45, Langton, ON N0E 1G0
GPS Coordinates:	42°42'12.10"N, 80°33'46.75"W
Inspected By:	Matt Churly, P.Eng., David Green, P.Eng., Tim Rosborough, P.Eng., Josef Schweighofer, P.Eng. (Cambium)
Weather:	14°C, cloudy, light rain

1 – Earth Embankment

Left Embankment – A large earth fill embankment exists on the left side of the spillway structure and bridge. The earth dam is in good condition with no noted locations of seepage or erosion. The upstream surface is protected with rock protection that was in generally good condition; however, some riprap is cracking/disintegrating. The sloughing/settlement that was noted during the previous inspection in 2014 does not appear to have gotten worse. Minor amounts of vegetation were found to be growing between the riprap on the downstream embankment. The downstream embankment was cleared of thick vegetation prior to the inspection and looked to be in good condition.

Right Embankment – A more natural embankment exists to the right of the spillway structure and bridge. The roadway embankment is in good condition and well vegetated.

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

Lift Control Chamber, Water Intake Tower, and Concrete Bypass Pipe – Limited inspection due to access. Majority of concrete piping is permanently submerged and / or buried. Review of GoPro footage at outlet reveals surface organics growth. The Lift Control Chamber appeared to be in good condition with recent concrete repairs.

Bypass Outlet Structure – The bypass outlet structure is generally in good to fair condition. However, there is a large area of spalling, delamination, and disintegration occurring at the interface with the angled face of the outlet wall. Severe map cracking with efflorescence is present on both side of the structure. There is a gap between the concrete bypass pipe and the headwall on the downstream outlet structure. Drawings indicate there should be bituminous joint filler that seals this gap that has become dislodged from the headwall.

There is ±900 mm of sedimentation at the outlet structure. Baffles blocks / energy dissipators and toe drain outlets are completely buried.

Left Wingwall – Wingwall has been refaced as part of past rehabilitation. Exposed concrete is in good condition with no spalls or delaminations. Portion of walls

extending into upstream reservoir had limited inspection due to access limitations. The portion of the wall below the water level is covered in sheet piling.

Right Wingwall – Condition is similar to left wingwall.

Left Bridge Abutment – Concrete is in good condition below bearing seat. This portion of spillway is assumed to be a county (Norfolk) asset. Erosion at the base of the wall is opening a gap between the abutment wall and spillway slab.

Right Bridge Abutment – Condition is similar to left abutment.

Spillway Structure (Upper) – Walls on either side of the spillway slab have been partially refaced and patched. The newer concrete is in good condition, with the exception of a small delamination near the base of the right wall. Exposed original walls are in good condition with localized deterioration in the form of cracks with efflorescence and delamination. The culvert outlet on the right side of the spillway is delaminated around its perimeter as concrete appears to be starting to disintegrate.

The spillway slab is in good condition with light erosion where original concrete is visible. The majority of the slab concrete is covered with an organic deposit that resembles disintegrated concrete with small aggregates.

Spillway Structure (Middle) – Similar to the upper portion of the spillway, the slab of the middle section of spillway is largely covered in a 20-50 mm thick layer of weak concrete-like material, assumed to be organic, and is superficial to the structural concrete of the underlying slab. Where exposed, the original concrete of the slab appears to be in good condition and is sound to hammer impact.

Transverse construction joints are present in the spillway slab and walls at ±7.47 m (24.5') intervals. Deterioration of the walls and slab of the spillway appear to be localized to C.J. locations. Several wall joints are fully delaminated with some spalls and disintegration of underlying concrete. A PVC waterstop is exposed between slabs 6 and 7 near the interface with the left wall. In general, C.J.s in the slabs appear to be in good condition and functioning well.

A review of the embankment-side face of the wall at C.J. locations reveals that several disintegration patterns and associated delamination appear to be reflected on both sides of the wall, full depth spalling is likely to occur at these locations in the future.

Lower Spillway Stilling Basin – The lowest portion of the spillway (stilling basin) is mostly covered in silt, vegetation, and debris (±500 thick) and could not be inspected due to limited exposure / visibility.

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

Chain-link Fencing – Galvanized 5' tall fencing is present on both sides of the spillway channel, side mounted near the top of the wall. The fencing and posts are generally in good condition with no obvious forms of deterioration.

Gain Armouring – Limited inspection due to flow. Assumed fair to good condition.

Wing Wall Sheet Piling – Piling is in good condition with light uniform surface rust (patina) above water. Organic film covers the majority of the underwater steel face.

Embankment Sheet Piling – Sheet piles are illustrated on the original plans for the dam in the core of the left embankment. The presence and condition of these piles could not be reviewed during the site inspection.

Subdrain System – Inspection was limited to what was visible from the base of the spillway chute. The 8" perforated CSP subdrain on the left spillway chute wall had moderate corrosion with section loss at the bottom of the pipe. The 8" perforated CSP subdrain on the right spillway chute wall is in very poor condition and is disjointed/crushed with coarse grained material entering the pipe as well as moderate corrosion. Course grained material upstream of the disjointed area indicates this pipe may be damaged in more than one location.

4 – Gates and/or Stop Logs

The top stop logs are in poor condition with severe rot and decay. The same concrete like organic material that is coating the spillway is present on the downstream side of the stop logs. Removing a small patch of this material showed the stop logs to be in fair condition, but inspection was limited. There is minor leakage between the stop logs.

5 – Water Level Gauge (reading and condition)

There is no water level gauge at the Deer Creek Dam. However, it is understood that there is a staff gauge in the reservoir near the boat ramp.

6 – Winches (type and number)

There are no winches at the Deer Creek Dam.

7 – Valves (type and number)

The valve system is operated from the lift control chamber using an Armco CPE-12 with a removable hand crank. The valve system was operated by LPRCA staff during the inspection (25 rotations opened, 25 rotations closed) and appears to be in working condition with significant sediment discharge at the outlet when opened. The paint is peeling on the valve system and light corrosion is visible. The valve stem exits the lift chamber and appears to be in good condition. Inspection of the slide gate was not possible as it was submerged and potentially buried under sediment/organic material.

It is understood that LPRCA operates the valve system semi-annually to flush sediment and greases the valve system with a grease gun.

8 – Boom (driftwood, chains, anchors)

There is a yellow public safety buoy system with 20 floats located upstream of the dam. The boom is securely anchored to the concrete wing wall on the left side and the sheet pile wingwall on the right side. The buoy system is in fair condition with some light corrosion of the chain and anchors and moderate fading of colour.

9 – Erosion (upstream and downstream)

Minor erosion was observed on the banks of the downstream bypass channel.

10 – Seepage or Leaks

There is minor leakage between the stop logs.

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

The dam was accessed via Norfolk County Road 45. The road is paved and in good condition with room to park on the shoulder west of the dam. The road crosses over the spillway chute structure. Keys to unlock the Bypass Outlet tower door are required from LPRCA staff. The handle to operate the valve is stored off site at LPRCA's head office.

12 – Safety Issues (public and operator)

Public Safety – No public safety issues were identified.

Operator Safety – No operator safety issues were identified.

13 – Signage

There are medium public safety signs (1220 mm x 1220 mm) mounted on wooden posts facing upstream on the left and right embankments. The sign reads "DANGER, No Trespassing, Access Beyond This Point May Result in Drowning". The signs also have the name of the dam, and "In An Emergency call 519-842-4242 [LPRCA's office] or 911", as well as LPRCA's logo. The signs appear to be in good condition.

There is a medium public safety sign (1220 mm x 1220 mm) mounted to wooden posts on top of the concrete bypass structure tower facing the road. The sign reads

"DANGER, No Trespassing, Access Beyond This Point May Result in Drowning". The signs also have the name of the dam, and "In An Emergency call 519-842-4242 [LPRCA's office] or 911", as well as LPRCA's logo. The signs appear to be in good condition.

There are small public safety signs (600 mm x 600 mm) mounted to wooden posts on the left and right approaches to the dam. The signs read "DANGER, Dam Ahead – Keep Out". The signs also have the name of the dam, and "In An Emergency Call 911", as well as LPRCA's logo. The signs appear to be in good condition.

There is a small public safety sign (600 mm x 600 mm) mounted to the chainlink fencing facing the road on the far side of the east embankment. The sign reads "DANGER, No Trespassing". The sign appears to be in good condition but is becoming overgrown with vegetation.

There is a small public safety sign (600 mm x 600 mm) mounted to the chainlink fencing facing the road on the far side of the west embankment. The sign reads "DANGER, Keep Off Dam". The sign is in poor condition.

There is a small "No Trespassing" sign mounted to the chainlink fencing facing the road on the far side of the west embankment. The sign is in fair condition.

14 – Divestment and/or Decommissioning Opportunities

None identified.

15 – General Remarks

No additional general remarks.

16 – Recommendations

- Conduct full cleaning of spillway surface to expose original concrete for inspection.
- A CTV inspection of the 36" concrete bypass pipe, toe drains and subdrains should be completed within the next 2 years. The extent of damage to the 8" CSP subdrains should be investigated, and a rehabilitation plan should be developed based on the results of the CTV Inspection. Clearing of sediment and possibly dewatering in the bypass stilling basin may need to be completed prior to inspection to access the toe drains.
- The paint should be removed from the peeling section on the valve and repainted to prevent further corrosion within the next 2 years.
- Pending findings of enhanced inspection, plan and carry out rehabilitation of concrete structure(s) 6-10 years.
- Review the results of the Public Safety Measures Plan currently being completed by Wills and implement the recommended public safety measures.
- When the public safety bouy system needs to be replaced, it should be replaced with one that meets the current guidelines for public safety booms.
- Continue to clear vegetation on the upstream and downstream embankments.
- Remove vegetation from the left upstream embankment.



Filename: DJI_0345.JPG

Photo 1 - October 29, 2024
Deer Creek Dam
Upstream of Spillway



Filename: DJI_0337.JPG

Photo 2 - October 29, 2024
Deer Creek Dam
Left Buoy Anchor



Filename: DJI_0338.JPG

Photo 3 - October 29, 2024
Deer Creek Dam
Upstream Buoy



Filename: DJI_0339.JPG

Photo 4 - October 29, 2024
Deer Creek Dam
Upstream Buoy



Filename: DJI_0341.JPG

Photo 5 - October 29, 2024
Deer Creek Dam
Right Upstream Buoy Anchor



Filename: DJI_0342.JPG

Photo 6 - October 29, 2024
Deer Creek Dam
Upstream Buoy



Filename: DJI_0347.JPG

Photo 7 - October 29, 2024
Deer Creek Dam
Upstream Dam



Filename: DJI_0394.JPG

Photo 8 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: DJI_0377.JPG

Photo 9 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: DJI_0378.JPG

Photo 10 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: DJI_0371.JPG

Photo 11 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: DJI_0372.JPG

Photo 12 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: DJI_0373.JPG

Photo 13 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment, Lift Control Chamber



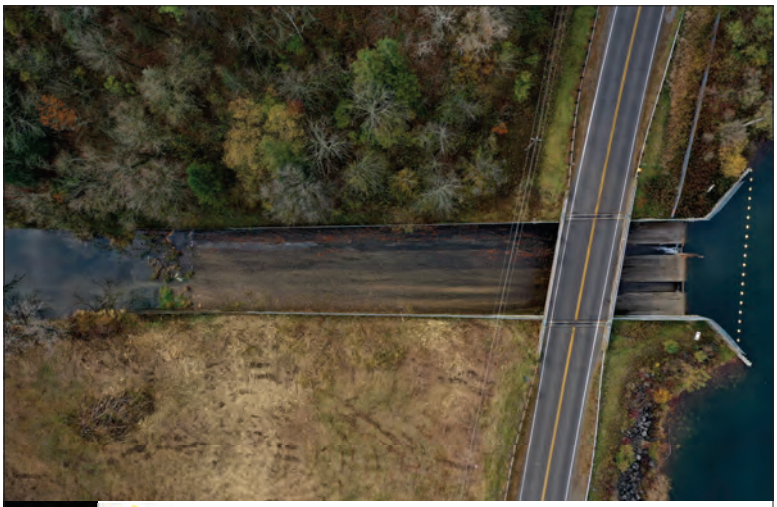
Filename: DJI_0374.JPG

Photo 14 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: DJI_0352.JPG

Photo 15 - October 29, 2024
Deer Creek Dam
Aerial view of spillway



Filename: DJI_0359.JPG

Photo 16 - October 29, 2024
Deer Creek Dam
Aerial view of spillway



Filename: DJI_0368.JPG

Photo 17 - October 29, 2024
Deer Creek Dam
Aerial view of spillway



Filename: DJI_0369.JPG

Photo 18 - October 29, 2024
Deer Creek Dam
Aerial view of spillway



Filename: DJI_0366.JPG

Photo 19 - October 29, 2024
Deer Creek Dam
Aerial view of spillway, stilling basin



Filename: DJI_0383.JPG

Photo 20 - October 29, 2024
Deer Creek Dam
Aerial view of spillway, stilling basin



Filename: DJI_0384.JPG

Photo 21 - October 29, 2024
Deer Creek Dam
Spillway



Filename: DJI_0357.JPG

Photo 22 - October 29, 2024
Deer Creek Dam
Downstream Embankment



Filename: DJI_0367.JPG

Photo 23 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: DJI_0386.JPG

Photo 24 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: DJI_0381.JPG

Photo 25 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: DJI_0387.JPG

Photo 26 - October 29, 2024
Deer Creek Dam
Right Downstream Embankment



Filename: DJI_0380.JPG

Photo 27 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: DJI_0360.JPG

Photo 28 - October 29, 2024
Deer Creek Dam
Downstream of Dam



Filename: DJI_0392.JPG

Photo 29 - October 29, 2024
Deer Creek Dam
Upstream of Dam



Filename: DJI_0397.JPG

Photo 30 - October 29, 2024
Deer Creek Dam
Dam Crest



Filename: DJI_0399.JPG

Photo 31 - October 29, 2024
Deer Creek Dam
Dam Crest



Filename: DSCN4889.JPG

Photo 32 - October 29, 2024
Deer Creek Dam
Right Upstream Embankment



Filename: DSCN4890.JPG

Photo 33 - October 29, 2024
Deer Creek Dam
Right Upstream Abutment



Filename: DSCN4893.JPG

Photo 34 - October 29, 2024
Deer Creek Dam
Right Upstream Embankment, Culvert



Filename: DSCN4896.JPG

Photo 35 - October 29, 2024
Deer Creek Dam
Public Safety Sign, Right Approach



Filename: DSCN4901.JPG

Photo 36 - October 29, 2024
Deer Creek Dam
Public Safety Sign, Right Approach



Filename: DSCN4906.JPG

Photo 37 - October 29, 2024
Deer Creek Dam
No Trespassing Sign, Right Approach



Filename: DJI_0335.JPG

Photo 38 - October 29, 2024
Deer Creek Dam
Public Safety Sign, Right Upstream Embankment



Filename: PXL_20241029_150944487.jpg

Photo 39 - October 29, 2024
Deer Creek Dam
Public Safety Sign, left upstream approach



Filename: PXL_20241029_151001564.jpg

Photo 40 - October 29, 2024
Deer Creek Dam
Public Safety Sign, left upstream approach



Filename: PXL_20241029_151023885.jpg

Photo 41 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_151042696.jpg

Photo 42 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_151322142.jpg

Photo 43 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_151612258.jpg

Photo 44 - October 29, 2024
Deer Creek Dam



Filename: PXL_20241029_151621080.jpg

Photo 45 - October 29, 2024
Deer Creek Dam
Lift Control Chamber, Bypass Valve



Filename: PXL_20241029_151716641.jpg

Photo 46 - October 29, 2024
Deer Creek Dam
Lift Control Chamber, Bypass Valve



Filename: PXL_20241029_151735117.jpg

Photo 47 - October 29, 2024
Deer Creek Dam
Lift Control Chamber, Bypass Valve



Filename: PXL_20241029_151752562.jpg

Photo 48 - October 29, 2024
Deer Creek Dam
Lift Control Chamber, Bypass Valve



Filename: PXL_20241029_151815984.jpg

Photo 49 - October 29, 2024
Deer Creek Dam
Lift Control Chamber, Bypass Valve



Filename: PXL_20241029_155057566.jpg

Photo 50 - October 29, 2024
Deer Creek Dam
Lift Control Chamber, Bypass Valve



Filename: PXL_20241029_151914429.jpg

Photo 51 - October 29, 2024
Deer Creek Dam
Lift Control Chamber, Valve Stem



Filename: PXL_20241029_151917265.jpg

Photo 52 - October 29, 2024
Deer Creek Dam
Lift Control Chamber, Valve Stem



Filename: PXL_20241029_151919843.jpg

Photo 53 - October 29, 2024
Deer Creek Dam
Lift Control Chamber



Filename: PXL_20241029_151921408.jpg

Photo 54 - October 29, 2024
Deer Creek Dam



Filename: PXL_20241029_151936174.jpg

Photo 55 - October 29, 2024
Deer Creek Dam
Lift Control Chamber



Filename: PXL_20241029_151941774.jpg

Photo 56 - October 29, 2024
Deer Creek Dam
Public Safety Sign, Lift Control Chamber



Filename: PXL_20241029_151944109.jpg

Photo 57 - October 29, 2024
Deer Creek Dam
Lift Control Chamber



Filename: PXL_20241029_152139094.jpg

Photo 58 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_152203600.jpg

Photo 59 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_152456803.jpg

Photo 60 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_152504186.jpg

Photo 61 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_153040176.jpg

Photo 62 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_153042098.jpg

Photo 63 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_153203138.jpg

Photo 64 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_153313722.jpg

Photo 65 - October 29, 2024
Deer Creek Dam
Left Upstream wingwall



Filename: DJI_0336.JPG

Photo 66 - October 29, 2024
Deer Creek Dam
Public Safety Sign, Left Upstream Embankment



Filename: PXL_20241029_153356421.jpg

Photo 67 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_153358145.jpg

Photo 68 - October 29, 2024
Deer Creek Dam
Left Upstream Embankment



Filename: PXL_20241029_153424479.jpg

Photo 69 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: PXL_20241029_153431226.jpg

Photo 70 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: PXL_20241029_153440464.jpg

Photo 71 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4910.JPG

Photo 72 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4911.JPG

Photo 73 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4912.JPG

Photo 74 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4914.JPG

Photo 75 - October 29, 2024
Deer Creek Dam
Right Downstream Embankment



Filename: DSCN4915.JPG

Photo 76 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall, Downstream Culvert



Filename: DSCN4916.JPG

Photo 77 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall, Downstream Culvert



Filename: DSCN4917.JPG

Photo 78 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4918.JPG

Photo 79 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4919.JPG

Photo 80 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4921.JPG

Photo 81 - October 29, 2024
Deer Creek Dam



Filename: DSCN4922.JPG

Photo 82 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4923.JPG

Photo 83 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4924.JPG

Photo 84 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4925.JPG

Photo 85 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4926.JPG

Photo 86 - October 29, 2024
Deer Creek Dam



Filename: DSCN4928.JPG

Photo 87 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4929.JPG

Photo 88 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4931.JPG

Photo 89 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4932.JPG

Photo 90 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4933.JPG

Photo 91 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4934.JPG

Photo 92 - October 29, 2024
Deer Creek Dam
Stilling Basin/Left Spillway Chute Wall



Filename: IMG_20241029_105718.jpg

Photo 93 - October 29, 2024
Deer Creek Dam
Stilling Basin



Filename: DSCN5050.JPG

Photo 94 - October 29, 2024
Deer Creek Dam
Stilling Basin



Filename: DSCN5051.JPG

Photo 95 - October 29, 2024
Deer Creek Dam
Stilling Basin



Filename: DSCN5052.JPG

Photo 96 - October 29, 2024
Deer Creek Dam



Filename: DSCN5054.JPG

Photo 97 - October 29, 2024
Deer Creek Dam
Stilling Basin



Filename: DSCN5055.JPG

Photo 98 - October 29, 2024
Deer Creek Dam
Stilling Basin



Filename: DSCN5057.JPG

Photo 99 - October 29, 2024
Deer Creek Dam
Stilling Basin



Filename: IMG_20241029_104330.jpg

Photo 100 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4927.JPG

Photo 101 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: IMG_20241029_104836.jpg

Photo 102 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: IMG_20241029_104338.jpg

Photo 103 - October 29, 2024
Deer Creek Dam
Downstream Side of Bridge



Filename: DSCN4935.JPG

Photo 104 - October 29, 2024
Deer Creek Dam
Lower Spillway



Filename: DSCN4937.JPG

Photo 105 - October 29, 2024
Deer Creek Dam
Left Spillway Subdrain



Filename: IMG_20241029_105704.jpg

Photo 106 - October 29, 2024
Deer Creek Dam
Left Spillway Subdrain



Filename: DSCN4938.JPG

Photo 107 - October 29, 2024
Deer Creek Dam
Left Spillway Subdrain



Filename: DSCN4940.JPG

Photo 108 - October 29, 2024
Deer Creek Dam
Left Spillway Subdrain



Filename: DSCN4941.JPG

Photo 109 - October 29, 2024
Deer Creek Dam
Right Spillway Subdrain



Filename: DSCN4944.JPG

Photo 110 - October 29, 2024
Deer Creek Dam
Right Spillway Subdrain



Filename: DSCN4945.JPG

Photo 111 - October 29, 2024
Deer Creek Dam
Right Spillway Subdrain



Filename: DSCN4946.JPG

Photo 112 - October 29, 2024
Deer Creek Dam
Right Spillway Subdrain



Filename: DSCN4942.JPG

Photo 113 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4943.JPG

Photo 114 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4947.JPG

Photo 115 - October 29, 2024
Deer Creek Dam
Lower Spillway



Filename: DSCN4948.JPG

Photo 116 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4949.JPG

Photo 117 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4950.JPG

Photo 118 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4951.JPG

Photo 119 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4956.JPG

Photo 120 - October 29, 2024
Deer Creek Dam
Lower Spillway



Filename: DSCN4957.JPG

Photo 121 - October 29, 2024
Deer Creek Dam
Lower Spillway



Filename: DSCN4963.JPG

Photo 122 - October 29, 2024
Deer Creek Dam
Lower Spillway



Filename: DSCN4958.JPG

Photo 123 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4959.JPG

Photo 124 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4962.JPG

Photo 125 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4965.JPG

Photo 126 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4968.JPG

Photo 127 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN4969.JPG

Photo 128 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN4972.JPG

Photo 129 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4973.JPG

Photo 130 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN4975.JPG

Photo 131 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4976.JPG

Photo 132 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4977.JPG

Photo 133 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4978.JPG

Photo 134 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4979.JPG

Photo 135 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4980.JPG

Photo 136 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4981.JPG

Photo 137 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN4982.JPG

Photo 138 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4983.JPG

Photo 139 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN4986.JPG

Photo 140 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4985.JPG

Photo 141 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4987.JPG

Photo 142 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN4988.JPG

Photo 143 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4989.JPG

Photo 144 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4990.JPG

Photo 145 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4991.JPG

Photo 146 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4992.JPG

Photo 147 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN4993.JPG

Photo 148 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4994.JPG

Photo 149 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: IMG_20241029_111040.jpg

Photo 150 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: IMG_20241029_110947.jpg

Photo 151 - October 29, 2024
Deer Creek Dam
Spillway Chute, PVC waterstop



Filename: DSCN4995.JPG

Photo 152 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4996.JPG

Photo 153 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4997.JPG

Photo 154 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4998.JPG

Photo 155 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN4999.JPG

Photo 156 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall, Bridge Abutment



Filename: DSCN5000.JPG

Photo 157 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN5001.JPG

Photo 158 - October 29, 2024
Deer Creek Dam
Right Bridge Abutment



Filename: DSCN5002.JPG

Photo 159 - October 29, 2024
Deer Creek Dam
Right Bridge Abutment



Filename: IMG_20241029_111550.jpg

Photo 160 - October 29, 2024
Deer Creek Dam
Underside of Bridge



Filename: IMG_20241029_111557.jpg

Photo 161 - October 29, 2024
Deer Creek Dam
Underside of Bridge



Filename: DSCN5003.JPG

Photo 162 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5004.JPG

Photo 163 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN5005.JPG

Photo 164 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: IMG_20241029_112111.jpg

Photo 165 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: IMG_20241029_112114.jpg

Photo 166 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: IMG_20241029_112149.jpg

Photo 167 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: IMG_20241029_112244.jpg

Photo 168 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN5006.JPG

Photo 169 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5007.JPG

Photo 170 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5008.JPG

Photo 171 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN4892.JPG

Photo 172 - October 29, 2024
Deer Creek Dam
Spillway Weir



Filename: PXL_20241029_153511830.jpg

Photo 173 - October 29, 2024
Deer Creek Dam
Spillway Weir



Filename: PXL_20241029_153626267.jpg

Photo 174 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN5009.JPG

Photo 175 - October 29, 2024
Deer Creek Dam
Spillway Weir



Filename: DSCN5010.JPG

Photo 176 - October 29, 2024
Deer Creek Dam
Spillway Weir



Filename: DSCN5011.JPG

Photo 177 - October 29, 2024
Deer Creek Dam
Spillway Weir



Filename: DSCN5012.JPG

Photo 178 - October 29, 2024
Deer Creek Dam
Spillway Weir



Filename: DSCN5013.JPG

Photo 179 - October 29, 2024
Deer Creek Dam
Spillway Weir



Filename: DSCN5014.JPG

Photo 180 - October 29, 2024
Deer Creek Dam
Spillway Weir



Filename: DSCN5016.JPG

Photo 181 - October 29, 2024
Deer Creek Dam
Spillway Weir



Filename: DSCN5018.JPG

Photo 182 - October 29, 2024
Deer Creek Dam
Spillway Weir, right stoplog bay



Filename: DSCN5019.JPG

Photo 183 - October 29, 2024
Deer Creek Dam
Upstream side of bridge



Filename: DSCN5027.JPG

Photo 184 - October 29, 2024
Deer Creek Dam
Spillway Weir, left stoplog bay



Filename: DSCN5020.JPG

Photo 185 - October 29, 2024
Deer Creek Dam
Spillway Weir, left stoplog bay



Filename: DSCN5021.JPG

Photo 186 - October 29, 2024
Deer Creek Dam
Spillway Weir, left stoplog bay



Filename: DSCN5029.JPG

Photo 187 - October 29, 2024
Deer Creek Dam
Spillway Weir, left stoplog bay



Filename: IMG_20241029_111804.jpg

Photo 188 - October 29, 2024
Deer Creek Dam
Spillway Weir, left stoplog bay



Filename: DSCN5024.JPG

Photo 189 - October 29, 2024
Deer Creek Dam
Spillway Weir, left stoplog bay



Filename: DSCN5025.JPG

Photo 190 - October 29, 2024
Deer Creek Dam
Spillway Weir, left stoplog bay



Filename: DSCN5026.JPG

Photo 191 - October 29, 2024
Deer Creek Dam
Spillway Weir, right stoplog bay



Filename: DSCN5036.JPG

Photo 192 - October 29, 2024
Deer Creek Dam
Spillway Weir, right stoplog bay



Filename: DSCN5037.JPG

Photo 193 - October 29, 2024
Deer Creek Dam
Spillway Weir, right stoplog bay



Filename: DSCN5039.JPG

Photo 194 - October 29, 2024
Deer Creek Dam
Spillway Weir, right stoplog bay



Filename: DSCN5038.JPG

Photo 195 - October 29, 2024
Deer Creek Dam
Spillway Weir, right stoplog bay



Filename: DSCN5030.JPG

Photo 196 - October 29, 2024
Deer Creek Dam
Spillway Weir, right stoplog bay



Filename: DSCN5028.JPG

Photo 197 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall, Upstream Wingwall



Filename: DSCN5042.JPG

Photo 198 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall, Upstream Wingwall



Filename: IMG_20241029_111810.jpg

Photo 199 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5032.JPG

Photo 200 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall, Upstream Wingwall



Filename: DSCN5017.JPG

Photo 201 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall, Upstream Wingwall



Filename: DSCN5033.JPG

Photo 202 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN5040.JPG

Photo 203 - October 29, 2024
Deer Creek Dam
Spillway Weir



Filename: DSCN5041.JPG

Photo 204 - October 29, 2024
Deer Creek Dam
Right Spillway stoplog bay



Filename: DSCN5043.JPG

Photo 205 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN5044.JPG

Photo 206 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN5045.JPG

Photo 207 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN5046.JPG

Photo 208 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5047.JPG

Photo 209 - October 29, 2024
Deer Creek Dam
Right Spillway Chute Wall



Filename: DSCN5048.JPG

Photo 210 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN5049.JPG

Photo 211 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN5056.JPG

Photo 212 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5058.JPG

Photo 213 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5059.JPG

Photo 214 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5060.JPG

Photo 215 - October 29, 2024
Deer Creek Dam
Spillway Chute



Filename: DSCN5061.JPG

Photo 216 - October 29, 2024
Deer Creek Dam
Stilling Basin



Filename: DSCN5063.JPG

Photo 217 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5064.JPG

Photo 218 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5065.JPG

Photo 219 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5066.JPG

Photo 220 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5067.JPG

Photo 221 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5068.JPG

Photo 222 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5069.JPG

Photo 223 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5070.JPG

Photo 224 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5071.JPG

Photo 225 - October 29, 2024
Deer Creek Dam
Left Spillway Chute Wall



Filename: DSCN5073.JPG

Photo 226 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: IMG_20241029_103714.jpg

Photo 227 - October 29, 2024
Deer Creek Dam
Downstream side of bridge



Filename: IMG_20241029_103718.jpg

Photo 228 - October 29, 2024
Deer Creek Dam
Downstream side of bridge



Filename: IMG_20241029_103724.jpg

Photo 229 - October 29, 2024
Deer Creek Dam
Downstream side of bridge



Filename: IMG_20241029_113619.jpg

Photo 230 - October 29, 2024
Deer Creek Dam
Bridge Deck



Filename: IMG_20241029_113621.jpg

Photo 231 - October 29, 2024
Deer Creek Dam
Bridge Deck



Filename: IMG_20241029_113625.jpg

Photo 232 - October 29, 2024
Deer Creek Dam
Bridge Deck



Filename: IMG_20241029_113626.jpg

Photo 233 - October 29, 2024
Deer Creek Dam
Bridge Deck



Filename: IMG_20241029_113630.jpg

Photo 234 - October 29, 2024
Deer Creek Dam
Bridge Deck



Filename: IMG_20241029_113639.jpg

Photo 235 - October 29, 2024
Deer Creek Dam
Bridge Deck



Filename: IMG_20241029_113703.jpg

Photo 236 - October 29, 2024
Deer Creek Dam
Bridge Deck



Filename: IMG_20241029_113708.jpg

Photo 237 - October 29, 2024
Deer Creek Dam
Bridge Deck drain



Filename: PXL_20241029_142439445.jpg

Photo 238 - October 29, 2024
Deer Creek Dam
Bridge deck drain



Filename: PXL_20241029_142425935.MP.jpg

Photo 239 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: PXL_20241029_142432447.jpg

Photo 240 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: DSCN5072.JPG

Photo 241 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: PXL_20241029_142515525.jpg

Photo 242 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: PXL_20241029_143539091.jpg

Photo 243 - October 29, 2024
Deer Creek Dam



Filename: PXL_20241029_144435079.jpg

Photo 244 - October 29, 2024
Deer Creek Dam



Filename: PXL_20241029_143815395.jpg

Photo 245 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: PXL_20241029_143817302.jpg

Photo 246 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: PXL_20241029_143859536.jpg

Photo 247 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: PXL_20241029_143911156.jpg

Photo 248 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: PXL_20241029_144001672.jpg

Photo 249 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: PXL_20241029_144242017.jpg

Photo 250 - October 29, 2024
Deer Creek Dam
Exposed Soil, Left Downstream Embankment



Filename: PXL_20241029_144439757.jpg

Photo 251 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment



Filename: PXL_20241029_145316273.jpg

Photo 252 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: PXL_20241029_145330142.MP.jpg

Photo 253 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: PXL_20241029_145337302.jpg

Photo 254 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: IMG_20241029_120317.jpg

Photo 255 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: PXL_20241029_145410793.jpg

Photo 256 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: PXL_20241029_145432842.jpg

Photo 257 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: PXL_20241029_145452532.jpg

Photo 258 - October 29, 2024
Deer Creek Dam



Filename: PXL_20241029_145455831.MP.jpg

Photo 259 - October 29, 2024
Deer Creek Dam



Filename: PXL_20241029_145506083.jpg

Photo 260 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: IMG_20241029_120314.jpg

Photo 261 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: IMG_20241029_120401.jpg

Photo 262 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: IMG_20241029_121351.jpg

Photo 263 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: IMG_20241029_122128.jpg

Photo 264 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure, Sediment Accumulation



Filename: IMG_20241029_122138.jpg

Photo 265 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure, Sediment Accumulation



Filename: IMG_20241029_122146.jpg

Photo 266 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure, Sediment Accumulation



Filename: IMG_20241029_122156.jpg

Photo 267 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure, Sediment Accumulation



Filename: PXL_20241029_145601615.jpg

Photo 268 - October 29, 2024
Deer Creek Dam
Bypass Outlet Structure



Filename: PXL_20241029_150137882.MP.jpg

Photo 269 - October 29, 2024
Deer Creek Dam
Downstream of Bypass Outlet Structure, Erosion



Filename: PXL_20241029_150255052.jpg

Photo 270 - October 29, 2024
Deer Creek Dam
Downstream of Bypass Outlet Structure, Erosion



Filename: PXL_20241029_150451026.jpg

Photo 271 - October 29, 2024
Deer Creek Dam
Downstream of Spillway



Filename: PXL_20241029_150651331.jpg

Photo 272 - October 29, 2024
Deer Creek Dam
Left Downstream Embankment

WILLS

PARTNERS IN ENGINEERING, PLANNING & ENVIRONMENTAL SERVICES

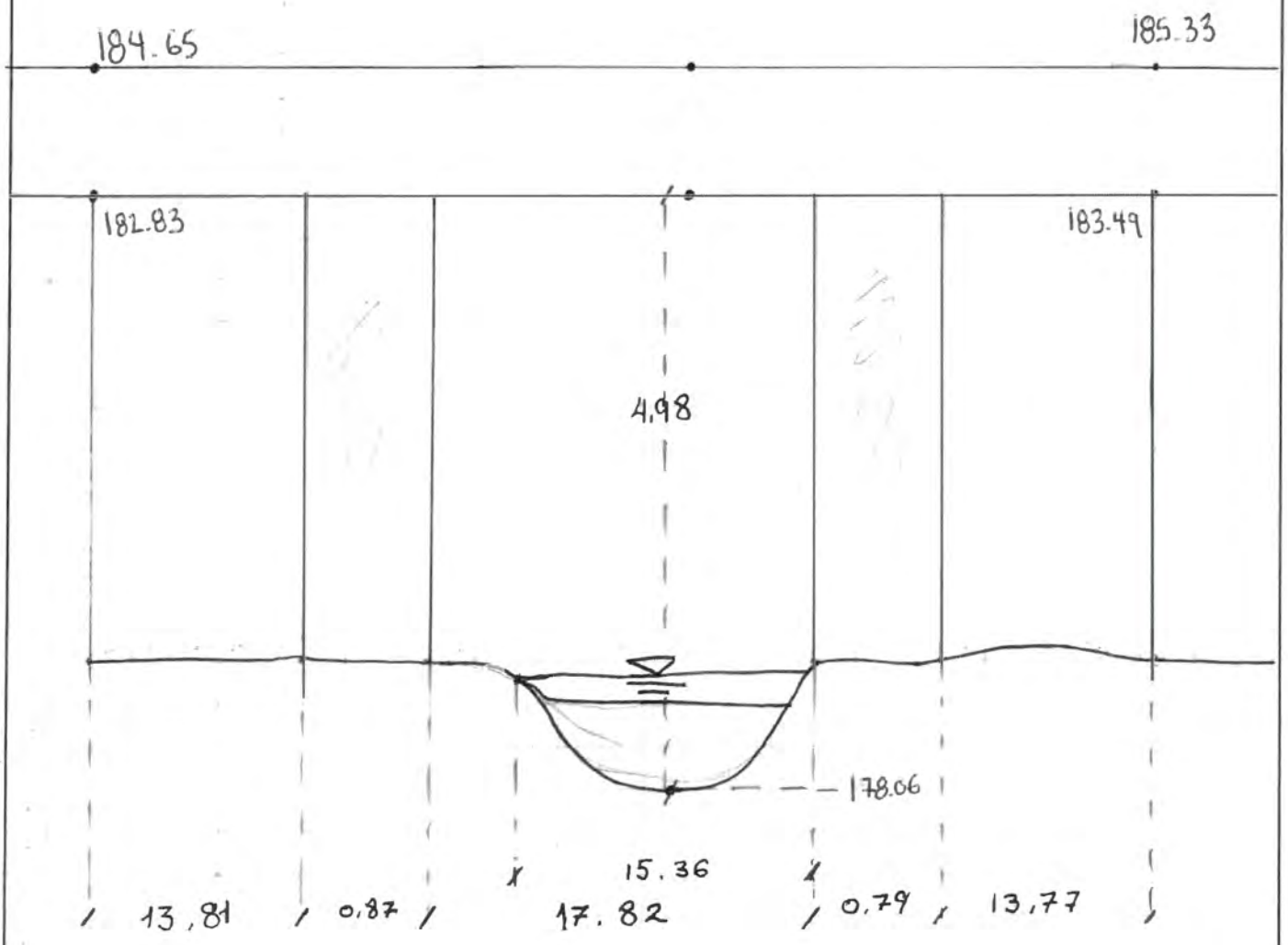
Project No. 5684 Sheet ____ of ____

Project Title ____

Author ____

Subject Walsingham Townline Rd. Bridge

Date ____



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P. 705.742.2297 F. 705.748.9944 E. wills@dmwills.com

WILLS
PARTNERS IN ENGINEERING, PLANNING & ENVIRONMENTAL SERVICES

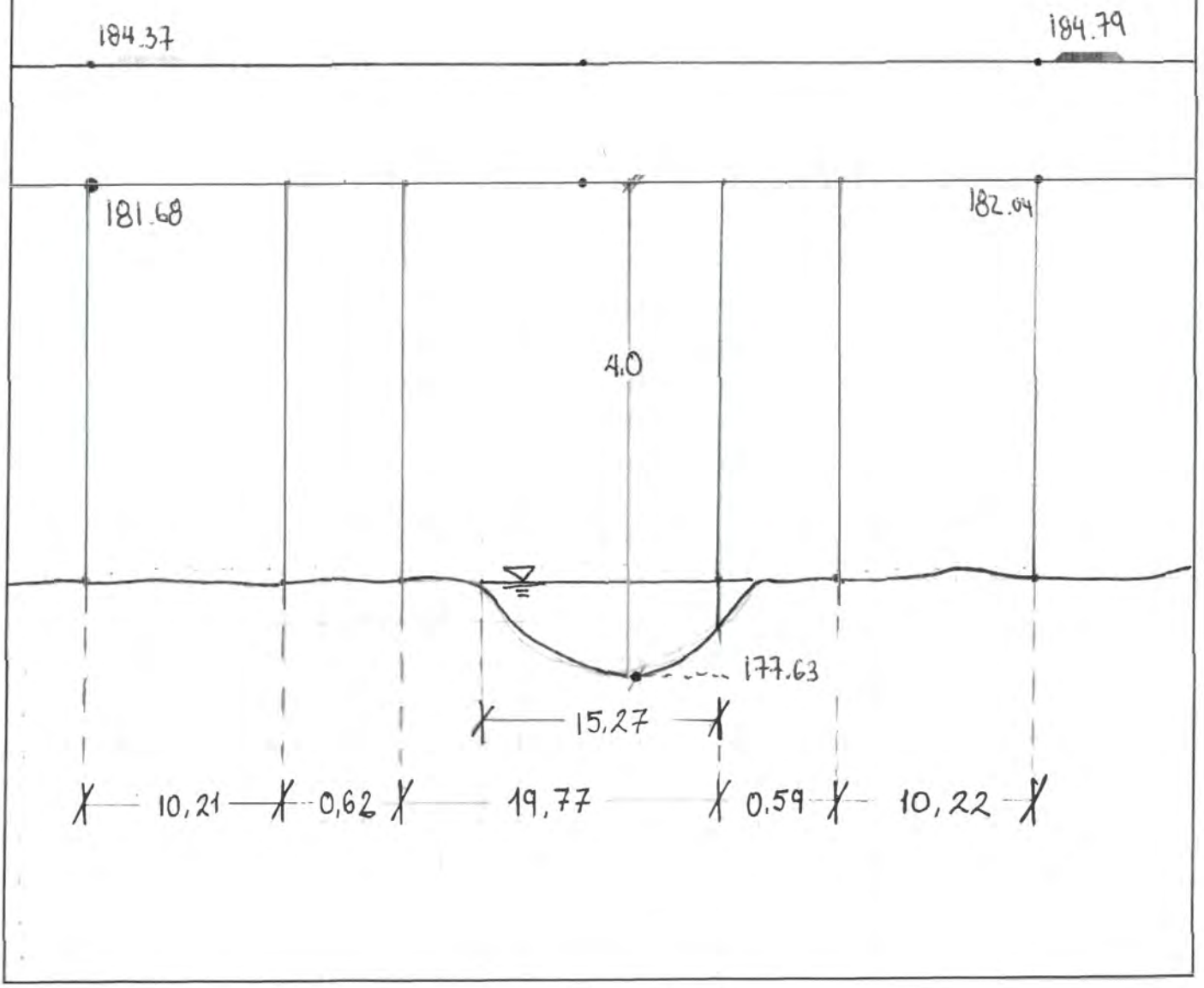
Project No. 5684 Sheet ____ of ____

Project Title ____

Author ____

Subject 6TH Concession Road Bridge

Date ____



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PARTNERS IN ENGINEERING, PLANNING & ENVIRONMENTAL SERVICES

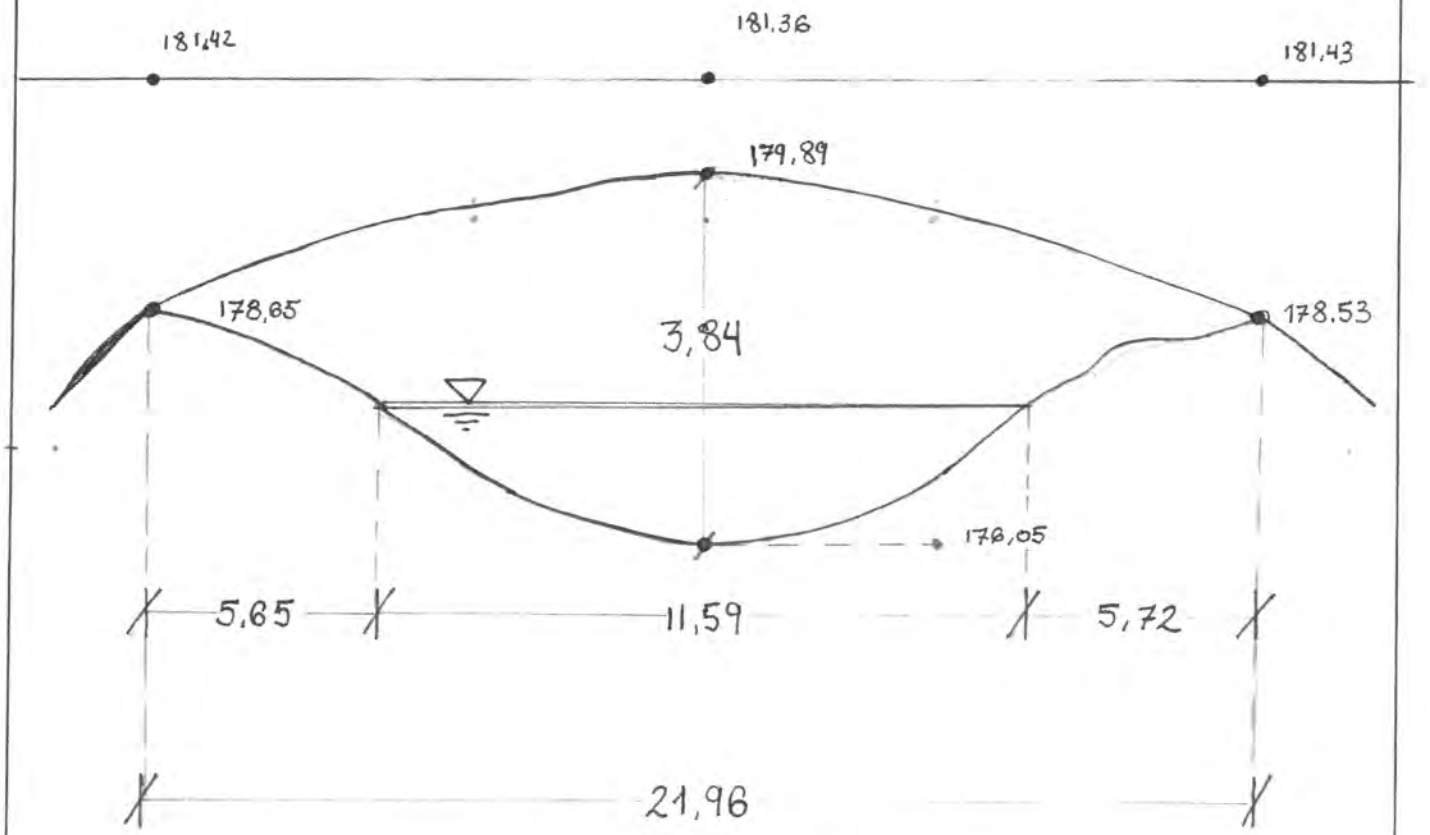
Project No. 5684 Sheet ____ of ____

Project Title ____

Author ____

Subject Norfolk County Road 60 Bridge

Date ____



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Filename: IMG_20241107_142246.jpg

Photo 1 - November 6 – 8, 2024
Deer Creek
Townline Road - Downstream Bridge



Filename: IMG_20241107_142350.jpg

Photo 2 - November 6 – 8, 2024
Deer Creek
Townline Road - Upstream Side of Pier 1



Filename: IMG_20241107_145234.jpg

Photo 3 - November 6 – 8, 2024
Deer Creek
Upstream Side of Pier 2



Filename: IMG_20241107_145316.jpg

Photo 4 - November 6 – 8, 2024
Deer Creek
Townline Road - Right Side of Bridge



Filename: IMG_20241107_160758.jpg

Photo 5 - November 6 – 8, 2024
Deer Creek
6th Concession Road Bridge

Photo 6 - November 6 – 8, 2024
Deer Creek

Photo 7 - November 6 – 8, 2024
Deer Creek

Photo 8 - November 6 – 8, 2024
Deer Creek



Filename: IMG_20241108_100514.jpg

Photo 9 - November 6 – 8, 2024
Deer Creek

Norfolk County Road 60 - Upstream Side of Bridge



Filename: IMG_20241108_102323.jpg

Photo 10 - November 6 – 8, 2024
Deer Creek

Norfolk County Road 60 Downstream Side Fascia of Bridge



Filename: IMG_20241108_102338.jpg

Photo 11 - November 6 – 8, 2024
Deer Creek

Norfolk County Road 60 Downstream Left Bank

Photo 12 - November 6 – 8, 2024
Deer Creek

FIELD SURVEY REPORT

24-5684

LPRCA Deer Creek Dam DSR

Callon Dietz File : 26-26684

PURPOSE

Providing a bathymetric survey to support the water volume calculations for Deer Creek Conservation Area

HORIZONTAL AND VERTICAL DATUMS

Trimble R12 GPS was setup in UTM 17 CSRS and calibrated to GCVD 2013 vertical datum by using Cosine First Order Control Monument 00819788109 (Elevation: 199.105)

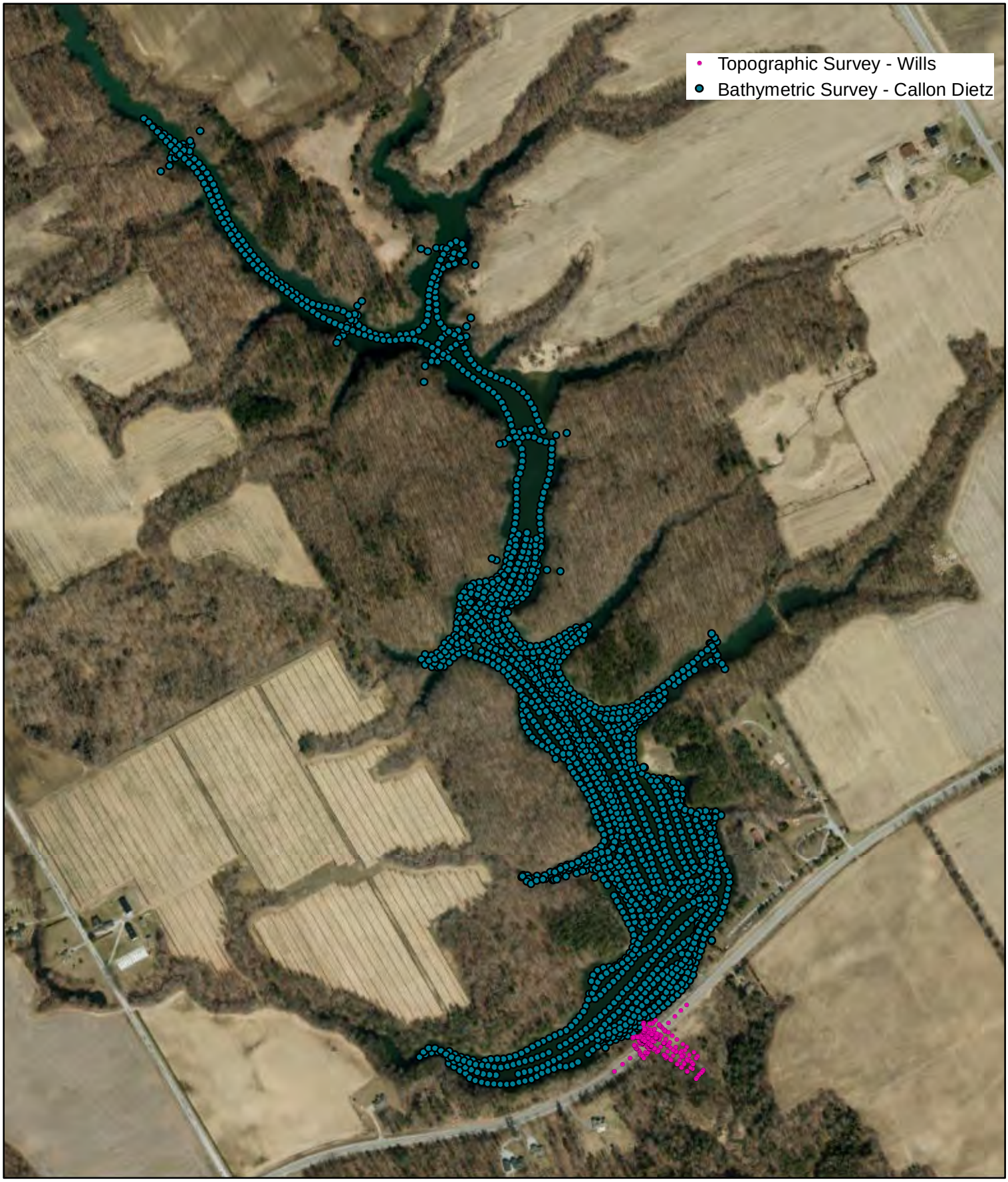
DETAIL SURVEY

Pairing a Trimble R12 GPS with a Sonarmite depth finder, sounding were collected in requested locations around Deer Creek lake with medium density for majority of the lake and high density close to the spillway. Cross sections were collected at 7 locations in shallower areas to the north using a combination of the Sonarmite and traditional GPS and survey pole methods. Edge of water shots were collected around the permitter of the Lake to help scale previously collected Lidar data. A CSV file was provided that included all bathymetric, edge of water, cross-section and Audit information.

SAFETY

A John Boat and 4.4hp motor was used by a 2-person survey crew with one operating the boat and the other colleting the survey data. All water safety rules were followed with life jackets worn and full boaters safety kit on board. A 3rd crew member was onsite to assist with the edge of water shots and communicated with the boat crew Via 2-way radio.

Yours truly,
CALLON DIETZ INC.
Paul Klinkhamer
E-mail: pklinkhamer@callondietz.com



Bathymetric and Topgraphic Survey Points 2024

Data Sources:
SWOOP 2010, GEOHUB 2024
Callon Dietz Bathymetric Survey Completed November 2024
Wills Topgraphic Survey Completed November 2024

NAD 1983 CSRS UTM Zone 17N

1:8,000

Meters

0 100 200

Drawn By:	MC
Checked By:	DG
Map Date:	01/26/25
Project Number:	5684
Map File Number	24-5684

WILLS

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Appendix B

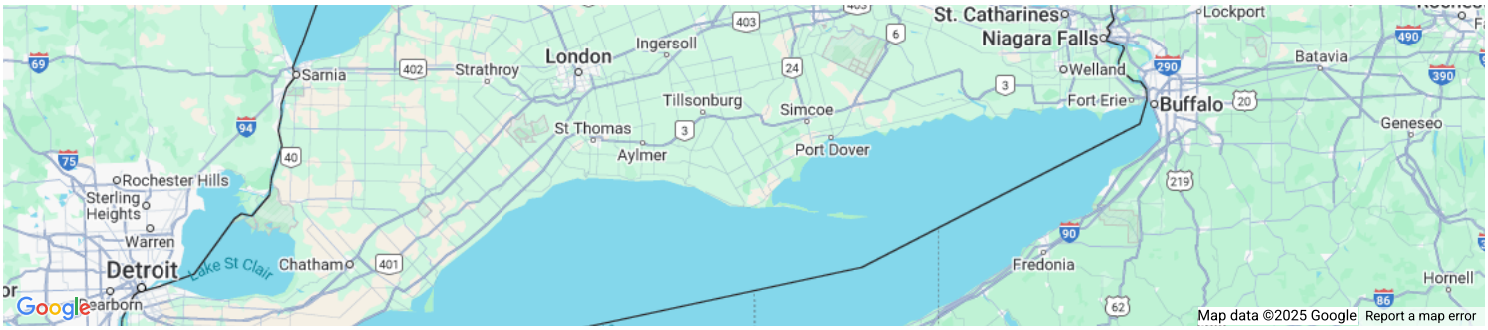
Hydrologic Assessment

Ontario IDF CURVE LOOKUP

Active coordinate

42° 42' 15" N, 80° 33' 45" W (42.704167,-80.562500)

Retrieved: Wed, 15 Jan 2025 15:15:38 GMT



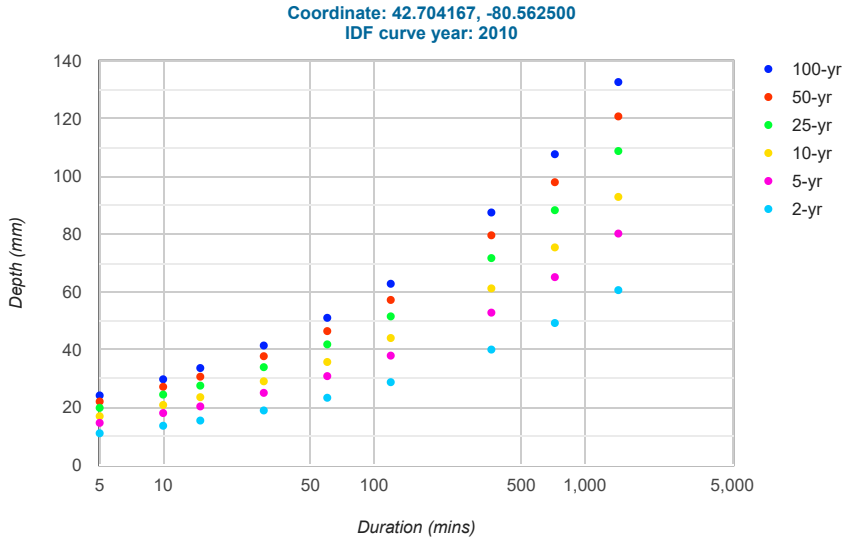
Location summary

These are the locations in the selection.

IDF Curve: 42° 42' 15" N, 80° 33' 45" W (42.704167,-80.562500)

Results

An IDF curve was found.



Coefficient summary

IDF Curve: 42° 42' 15" N, 80° 33' 45" W (42.704167,-80.562500)

Retrieved: Wed, 15 Jan 2025 15:15:38 GMT

Data year: 2010
IDF curve year: 2010

Return period	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
A	23.3	30.8	35.7	41.8	46.4	51.0
B	-0.699	-0.699	-0.699	-0.699	-0.699	-0.699

Statistics

Rainfall intensity (mm hr^{-1})

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr	132.3	81.5	61.4	37.8	23.3	14.4	6.7	4.1	2.5
5-yr	174.9	107.8	81.2	50.0	30.8	19.0	8.8	5.4	3.3
10-yr	202.8	124.9	94.1	58.0	35.7	22.0	10.2	6.3	3.9
25-yr	237.4	146.3	110.2	67.9	41.8	25.7	11.9	7.4	4.5
50-yr	263.6	162.3	122.3	75.3	46.4	28.6	13.3	8.2	5.0
100-yr	289.7	178.4	134.4	82.8	51.0	31.4	14.6	9.0	5.5

Rainfall depth (mm)

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr	11.0	13.6	15.4	18.9	23.3	28.7	40.0	49.2	60.6
5-yr	14.6	18.0	20.3	25.0	30.8	37.9	52.8	65.1	80.2
10-yr	16.9	20.8	23.5	29.0	35.7	44.0	61.2	75.4	92.9
25-yr	19.8	24.4	27.5	33.9	41.8	51.5	71.7	88.3	108.8
50-yr	22.0	27.1	30.6	37.7	46.4	57.2	79.6	98.0	120.8
100-yr	24.1	29.7	33.6	41.4	51.0	62.8	87.5	107.7	132.7

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Last Modified: September 2016

24 hr SCS Type 2 Distribution for Deer Creek DSR

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: 16-Jan-25

Data Year	2010	**Lat**	45.036747	**Areal Reduction Factor:**	0.992
Climate Year	2010	**Long**	-76.813466		

Rainfall

	Incremental 24 HR SCS (mm)							
Time (hrs)	**2-Year**	**5-Year**	**10-Year**	**25-Year**	**50-Year**	**100-Year**	**200-Year**	**1000-Year**
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.25	0.15	0.20	0.23	0.27	0.30	0.33	0.37	0.44
0.50	0.16	0.21	0.24	0.28	0.31	0.34	0.38	0.45
0.75	0.16	0.21	0.24	0.29	0.32	0.35	0.38	0.46
1.00	0.16	0.22	0.25	0.29	0.33	0.36	0.39	0.47
1.25	0.17	0.22	0.26	0.30	0.33	0.37	0.40	0.48
1.50	0.17	0.23	0.26	0.31	0.34	0.38	0.41	0.50
1.75	0.18	0.23	0.27	0.31	0.35	0.38	0.42	0.51
2.00	0.18	0.24	0.27	0.32	0.36	0.39	0.43	0.52
2.25	0.18	0.24	0.28	0.33	0.36	0.40	0.44	0.53
2.50	0.19	0.25	0.28	0.33	0.37	0.41	0.45	0.54
2.75	0.19	0.25	0.29	0.34	0.38	0.42	0.46	0.55
3.00	0.19	0.26	0.30	0.35	0.38	0.42	0.47	0.56
3.25	0.20	0.26	0.30	0.35	0.39	0.43	0.48	0.57
3.50	0.20	0.27	0.31	0.36	0.40	0.44	0.49	0.58
3.75	0.21	0.27	0.31	0.37	0.41	0.45	0.49	0.59
4.00	0.21	0.28	0.32	0.37	0.42	0.46	0.50	0.60
4.25	0.22	0.28	0.33	0.39	0.43	0.47	0.52	0.62
4.50	0.22	0.29	0.34	0.40	0.44	0.48	0.53	0.64
4.75	0.23	0.30	0.35	0.41	0.46	0.50	0.55	0.67
5.00	0.24	0.31	0.36	0.42	0.47	0.52	0.57	0.68
5.25	0.25	0.32	0.38	0.44	0.49	0.54	0.59	0.71
5.50	0.25	0.33	0.39	0.45	0.50	0.55	0.61	0.73
5.75	0.26	0.34	0.40	0.47	0.52	0.57	0.63	0.75
6.00	0.27	0.35	0.41	0.48	0.53	0.58	0.64	0.77
6.25	0.28	0.36	0.42	0.49	0.55	0.60	0.66	0.80
6.50	0.28	0.37	0.43	0.51	0.56	0.62	0.68	0.82
6.75	0.29	0.38	0.44	0.52	0.58	0.63	0.70	0.84
7.00	0.30	0.39	0.46	0.53	0.59	0.65	0.72	0.86
7.25	0.31	0.40	0.47	0.55	0.61	0.67	0.74	0.88
7.50	0.31	0.41	0.48	0.56	0.62	0.68	0.75	0.90
7.75	0.32	0.42	0.49	0.57	0.64	0.70	0.77	0.92
8.00	0.33	0.43	0.50	0.59	0.65	0.72	0.79	0.95
8.25	0.35	0.46	0.54	0.63	0.70	0.77	0.84	1.01
8.50	0.39	0.51	0.59	0.69	0.77	0.85	0.93	1.12
8.75	0.43	0.56	0.65	0.76	0.85	0.93	1.03	1.23
9.00	0.46	0.61	0.71	0.83	0.92	1.01	1.11	1.34
9.25	0.48	0.64	0.74	0.86	0.96	1.05	1.16	1.39
9.50	0.48	0.64	0.74	0.86	0.96	1.05	1.16	1.39
9.75	0.52	0.68	0.79	0.92	1.02	1.12	1.24	1.48
10.00	0.57	0.75	0.87	1.02	1.14	1.25	1.38	1.65
10.25	0.65	0.86	0.99	1.16	1.29	1.42	1.56	1.88
10.50	0.74	0.97	1.13	1.32	1.46	1.61	1.77	2.13
10.75	0.87	1.14	1.32	1.54	1.71	1.88	2.07	2.49
11.00	1.01	1.33	1.54	1.80	2.00	2.20	2.42	2.91
11.25	1.28	1.68	1.95	2.28	2.53	2.78	3.06	3.68
11.50	1.63	2.14	2.48	2.90	3.22	3.54	3.90	4.68
11.75	6.64	8.72	10.10	11.83	13.13	14.42	15.90	19.08
12.00	16.39	21.51	24.92	29.19	32.41	35.60	39.23	47.10
12.25	2.60	3.41	3.95	4.62	5.13	5.64	6.21	7.46
12.50	1.77	2.32	2.69	3.15	3.49	3.84	4.23	5.08
12.75	1.22	1.60	1.85	2.17	2.40	2.64	2.91	3.49
13.00	1.03	1.35	1.56	1.83	2.03	2.23	2.46	2.95
13.25	0.87	1.14	1.32	1.55	1.72	1.89	2.08	2.50
13.50	0.77	1.01	1.17	1.37	1.52	1.67	1.84	2.20
13.75	0.67	0.88	1.02	1.20	1.33	1.46	1.61	1.93
14.00	0.60	0.79	0.91	1.07	1.19	1.30	1.44	1.72
14.25	0.55	0.72	0.83	0.97	1.08	1.19	1.31	1.57
14.50	0.52	0.68	0.79	0.93	1.03	1.13	1.25	1.50
14.75	0.49	0.65	0.75	0.88	0.98	1.07	1.18	1.42
15.00	0.47	0.61	0.71	0.83	0.93	1.02	1.12	1.34
15.25	0.44	0.58	0.67	0.78	0.87	0.96	1.05	1.27
15.50	0.42	0.55	0.63	0.74	0.82	0.90	1.00	1.19
15.75	0.39	0.51	0.59	0.69	0.77	0.84	0.93	1.11
16.00	0.36	0.48	0.55	0.65	0.72	0.79	0.87	1.04
16.25	0.34	0.45	0.52	0.61	0.68	0.75	0.82	0.99
16.50	0.33	0.44	0.51	0.60	0.66	0.73	0.80	0.96
16.75	0.32	0.43	0.49	0.58	0.64	0.70	0.78	0.93
17.00	0.32	0.41	0.48	0.56	0.62	0.69	0.76	0.91
17.25	0.31	0.40	0.46	0.54	0.60	0.66	0.73	0.88
17.50	0.30	0.39	0.45	0.53	0.59	0.65	0.71	0.85
17.75	0.29	0.38	0.44	0.51	0.57	0.62	0.69	0.82
18.00	0.28	0.36	0.42	0.49	0.55	0.60	0.66	0.80
18.25	0.27	0.35	0.41	0.48	0.53	0.58	0.64	0.77
18.50	0.26	0.34	0.39	0.46	0.51	0.56	0.62	0.74
18.75	0.25	0.33	0.38	0.44	0.49	0.54	0.60	0.72
19.00	0.24	0.31	0.36	0.43	0.47	0.52	0.57	0.69
19.25	0.23	0.30	0.35	0.41	0.46	0.50	0.55	0.66
19.50	0.22	0.29	0.34	0.39	0.44	0.48	0.53	0.63
19.75	0.21	0.28	0.32	0.38	0.42	0.46	0.50	0.61
20.00	0.20	0.26	0.31	0.36	0.40	0.44	0.48	0.58
20.25	0.20	0.26	0.30	0.35	0.39	0.43	0.47	0.56
20.50	0.19	0.25	0.29	0.35	0.38	0.42	0.46	0.56
20.75	0.19	0.25	0.29	0.34	0.38	0.42	0.46	0.55
21.00	0.19	0.25	0.29	0.34	0.38	0.41	0.46	0.55
21.25	0.19	0.25	0.29	0.34	0.37	0.41	0.45	0.54
21.50	0.19	0.25	0.28	0.33	0.37	0.41	0.45	0.54
21.75	0.18	0.24	0.28	0.33	0.37	0.40	0.44	0.53
22.00	0.18	0.24	0.28	0.32	0.36	0.40	0.44	0.52
22.25	0.18	0.24	0.27	0.32	0.36	0.39	0.43	0.52
22.50	0.18	0.24	0.27	0.32	0.35	0.39	0.43	0.52
22.75	0.18	0.23	0.27	0.32	0.35	0.38	0.42	0.51
23.00	0.18	0.23	0.27	0.31	0.35	0.38	0.42	0.50
23.25	0.17	0.23	0.26	0.31	0.34	0.38	0.41	0.50
23.50	0.17	0.23	0.26	0.31	0.34	0.37	0.41	0.49
23.75	0.17	0.22	0.26	0.30	0.34	0.37	0.41	0.49
24.00	0.17	0.22	0.25	0.30	0.33	0.36	0.40	0.48
Total	**60.6**	**79.6**	**92.2**	**107.9**	**119.8**	**131.6**	**145.1**	**174.2**

Hurricane Hazel for Deer Creek DSR

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: 28-Jan-25

Data Year 1954

Areal Reduction Factor: 0.992

Rainfall

Time (hrs)	Hazel (mm)	Hazel (mm) with Areal Reduction Factor =0.992
0.00	0.0	0.00
1.00	2.0	1.98
2.00	2.0	1.98
3.00	2.0	1.98
4.00	2.0	1.98
5.00	2.0	1.98
6.00	2.0	1.98
7.00	2.0	1.98
8.00	2.0	1.98
9.00	2.0	1.98
10.00	2.0	1.98
11.00	2.0	1.98
12.00	2.0	1.98
13.00	2.0	1.98
14.00	2.0	1.98
15.00	2.0	1.98
16.00	2.0	1.98
17.00	2.0	1.98
18.00	2.0	1.98
19.00	2.0	1.98
20.00	2.0	1.98
21.00	2.0	1.98
22.00	2.0	1.98
23.00	2.0	1.98
24.00	2.0	1.98
25.00	2.0	1.98
26.00	2.0	1.98
27.00	2.0	1.98
28.00	2.0	1.98
29.00	2.0	1.98
30.00	2.0	1.98
31.00	2.0	1.98
32.00	2.0	1.98
33.00	2.0	1.98
34.00	2.0	1.98
35.00	2.0	1.98
36.00	3.0	2.98
37.00	6.0	5.95
38.00	4.0	3.97
39.00	6.0	5.95
40.00	13.0	12.90
41.00	17.0	16.86
42.00	13.0	12.90
43.00	23.0	22.82
44.00	13.0	12.90
45.00	13.0	12.90
46.00	53.0	52.58
47.00	38.0	37.70
48.00	13.0	12.90
Total	**285.0**	**282.7**

Areal Reduction Factors Hazel

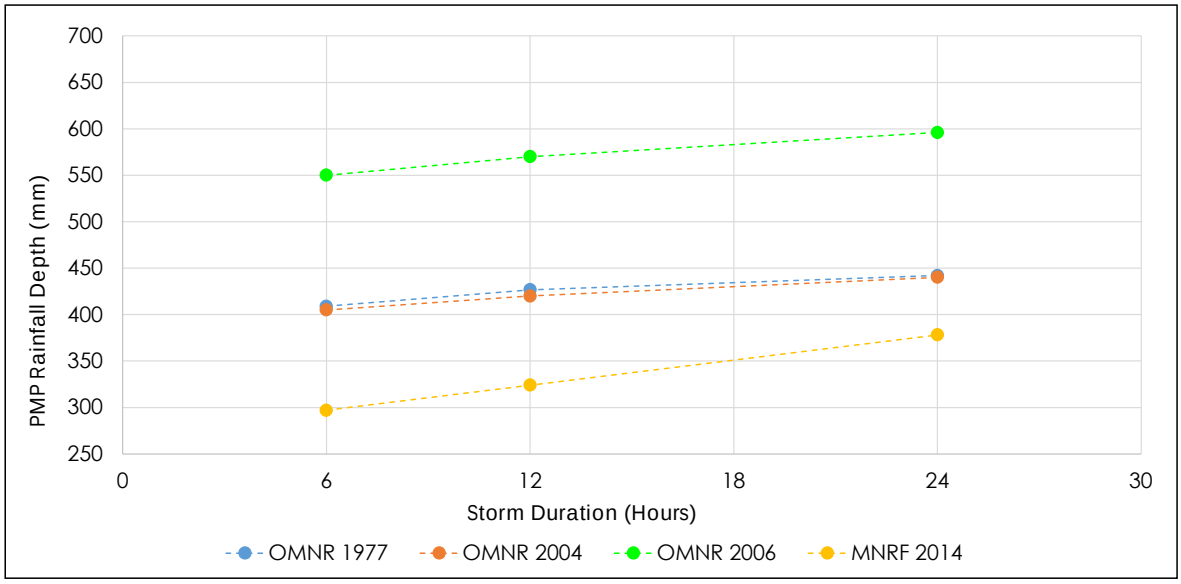
Equivalent Circle Diamter (km^2)	Areal Reduction Factor
0 to 25	100.00
26 to 45	99.20
46 to 65	98.20
66 to 90	97.10
91 to 115	96.30
116 to 140	95.40
141 to 165	94.80
166 to 195	94.20
196 to 220	93.50
221 to 245	92.70
246 to 270	92.00
271 to 450	89.40
451 to 575	86.70
576 to 700	84.00
701 to 850	82.40
851 to 1000	80.80
1001 to 1200	79.30
1201 to 1500	76.60
1501 to 1700	74.40
1701 to 2000	73.30
2001 to 2200	71.70
2201 to 2500	70.20
2501 to 2700	69.00
2701 to 4500	64.40
4501 to 6000	61.40
6001 to 7000	58.90
7001 to 8000	57.40

PMP Value Determination for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: January 20, 2025

Storm Duration (hour)	OMNR 1977 (mm)	OMNR 2004 (mm)	OMNR 2006 (mm)	MNRF 2014 (mm)	Average (mm)
6	409	405	550	297	415
12	427	420	570	324	435
24	442	440	596	378	464

Selected Values (mm)
405
420
440



Time of Concentration Calculations

Project No: 5630
Project Name: Deer Creek Dam
Designed/Checked By: MC/DG
Date: 26-Jan-24

Basin Model: Deer Creek DSR

Time of Concentration TR55 Subbasin1

				X	Y	
Subbasin1				12768.721	235.856	
				12668.721	233.691	
		Sheet Flow	**Slope AB**	**0.02165**		
				12668.721	233.691	
				10192.5	228.57	
		Shallow Concentrated Flow	**Slope BC**	**0.002068079**		
				10192.51	228.57	
				7504.34	226.28	
		Channel flow	**Slope CD**	**0.000851881**		
				7504.34	226.28	
				0	188.19	
		Channel flow	**Slope DE**	**0.00507573**		
		Sheet Flow				
	Segment ID	Segment 1 (SI)	Segment 2 (SI)	Segment 1 (Imperial)	Segment 2 (Imperial)	
1. Surface description (table 3-1)		Residue cover > 20%	N/A	Residue cover > 20%	N/A	
2. Manning's roughness coefficient, n (table 3-1)	n	0.17	0	0.17	0.00	
3. Flow length, L (total L ≤ 300 ft)	L	100	0	328.08	0.00	
4. Two-year 24-hour rainfall, P2	P_2	60.6	0	2.39	0.00	
5. Land slope, s	Land Slope	0.022	0	0.022	0.00	
$T_t = \frac{0.007(nL)^{0.8}}{P_2^{0.5} s^{0.4}}$	T_sheet			**0.52**		**0.52**
6						
		Shallow Concentrated Flow				
	Segment ID	Segment 1 (SI)	Segment 2 (SI)	Segment 1 (Imperial)	Segment 2 (Imperial)	
7. Surface description (paved or unpaved)		Unpaved	Unpaved	Unpaved	Unpaved	
8. Flow length, L	L	2476.2	0	8124.05	0.00	
9. Watercourse slope, s	s	0.002068079	0	0.002068	0.00	
10. Average velocity, V (figure 3-1)	V	N/A		0.73	0.00	
Unpaved $V = 16.1345 S^{0.5}$						
Paved V= $20.3282 S^{0.5}$						
$T_t = \frac{L}{3600V}$	T_shallow			3.08		**3.08**
11						
		Channel flow				
	Segment ID	Segment 1 (SI)	Segment 2 (SI)	Segment 1 (Imperial)	Segment 2 (Imperial)	
12. Cross sectional flow area, a	area	4.94	3.20	53.17	34.44	
13. Wetted perimeter, pw	p_w	7.18	6.79	23.54	22.29	
14. Hydraulic radius, r= Compute r	r			2.26	1.54	
15 Channel slope, s	s			0.001	0.005	
16. Manning's roughness coefficient, n	n	0.035	0.035	0.035	0.035	
$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n}$	V			2.14	4.05	
17						
18. Flow length, L	L	2688.17	7504.34	8819.46	24620.54	
$T_t = \frac{L}{3600V}$	T_channel			1.15	1.69	**2.83**
19						
20. Watershed or subarea Tc or Tt (add Tt in steps 6, 11, and 19)					Tc (Hr)	**6.43**
					Tc (mins)	**385.93**

Project No: 5630
Project Name: Deer Creek Dam
Designed/Checked By: MC/DG
Date: 26-Jan-24

Basin Model: Deer Creek DSR

Time of Concentration TR55 Subbasin2

				X	Y	
Subbasin2				4437.313	214.387	
				4337.916	213.83	
		Sheet Flow	**Slope AB**	**0.005603791**		
				4337.916	213.83	
				2998.5	211.56	
		Shallow Concentrated Flow	**Slope BC**	**0.001694731**		
				2998.5	211.56	
				2079.31	186.58	
		Shallow Concentrated Flow	**Slope CD**	**0.027176988**		
				2079.31	186.58	
				0	181.584	
		Channel flow	**Slope DE**	**0.00240272**		
		Sheet Flow				
	Segment ID	Segment 1 (SI)	Segment 2 (SI)	Segment 1 (Imperial)	Segment 2 (Imperial)	
1. Surface description (table 3-1)		Residue cover > 20%	N/A	Residue cover > 20%	N/A	
2. Manning's roughness coefficient, n (table 3-1)	n	0.170	0	0.170	0.00	
3. Flow length, L (total L ≤ 300 ft)	L	99.397	0	326.11	0.00	
4. Two-year 24-hour rainfall, P2	P_2	60.60	0	2.386	0.00	
5. Land slope, s	Land Slope	0.0056	0	0.0056	0.00	
$T_t = \frac{0.007(nL)^{0.8}}{P_2^{0.5} s^{0.4}}$	T_sheet			**0.90**		**0.90**
6						
		Shallow Concentrated Flow				
	Segment ID	Segment 1 (SI)	Segment 2 (SI)	Segment 1 (Imperial)	Segment 2 (Imperial)	
7. Surface description (paved or unpaved)		Unpaved	Unpaved	Unpaved	Unpaved	
8. Flow length, L	L	1339.4	919.16	4394.51	3015.62	
9. Watercourse slope, s	s	0.0017	0.027176988	0.002	0.03	
10. Average velocity, V (figure 3-1)	V	N/A		0.66	2.66	
Unpaved $V = 16.1345S^{0.5}$						
Paved V= $20.3282S^{0.5}$						
$T_t = \frac{L}{3600V}$	T_shallow			1.84	0.31	**2.15**
11						
		Channel flow				
	Segment ID	Segment 1 (SI)	Segment 2 (SI)	Segment 1 (Imperial)	Segment 2 (Imperial)	
12. Cross sectional flow area, a	area	1.43	0.00	15.40	0.00	
13. Wetted perimeter, pw	p_w	4.99	0.00	16.37	0.00	
14. Hydraulic radius, r= Compute r	r			0.94		
15 Channel slope, s	s			0.0024	0.0024	
16. Manning's roughness coefficient, n	n			0.035	0.035	
$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n}$	V			2.00	#VALUE!	
17						
18. Flow length, L	L	2079.31	0	6821.88	0.00	
$T_t = \frac{L}{3600V}$	T_channel			0.95	#VALUE!	**0.95**
19						
20. Watershed or subarea Tc or Tt (add Tt in steps 6, 11, and 19)					Tc (Hr)	**3.99**
					Tc (mins)	**239.64**

Loss Parameters - Green and Ampt Deer Creek DSR

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: 23-Jan-25

Standard Parameters

Texture Code	Texture	Porosity	Effective Porosity	Soil Suction Head at the wetting front (mm) (Low)	Soil Suction Head at the wetting front (mm) (Average)	Soil Suction Head at the wetting front (mm) (High)	Saturated Hydraulic Conductity (mm/h)	Field Capacity Θ_o	Wilting Point Θ_w
C	Clay	0.475	0.385	320	660	1000	0.3	0.378	0.265
CL	Clay Loam	0.464	0.309	210	210	210	1	0.31	0.187
CSL	Coarse Sandy Loam	0.453	0.412	110	180	250	11	0.19	0.085
FS	Fine Sand	0.437	0.417	49	99.5	150	120	0.062	0.024
FSL	Fine Sandy Loam	0.453	0.412	110	180	250	11	0.19	0.085
L	Loam	0.463	0.434	89	219.5	350	3	0.232	0.116
LFS	Loamy Fine Sand	0.437	0.401	61	155.5	250	30	0.105	0.047
LS	Loamy Sand	0.437	0.401	61	155.5	250	30	0.105	0.047
LVFS	Loamy Very Fine Sand	0.437	0.401	61	155.5	250	30	0.105	0.047
ORG	Organic	0.56	0.56	180	200	220	0.1	0.46	0.27
S	Sand	0.437	0.417	49	99.5	150	120	0.062	0.024
SIC	Silty Clay	0.479	0.423	290	290	290	1	0.371	0.251
SICL	Silty Clay Loam	0.471	0.432	270	270	270	1	0.342	0.21
SIL	Silt Loam	0.501	0.486	170	170	170	7	0.284	0.135
SL	Sandy Loam	0.453	0.412	110	180	250	11	0.19	0.085
VFSL	Very Fine Sandy Loam	0.453	0.412	110	180	250	11	0.19	0.085

1. GREEN-AMPT INFILTRATION PARAMETERS FROM SOILS DATA, W.J. Rawls, D. L. Brakensiek, N. Miller , 1983
2. TECHNICAL GUIDELINES FOR FLOOD HAZARD MAPPING ,Enviromental Water Resource Group, 2017
3. LPRCA hydrology Study 2006 by Schroeter & Associates

Loss Method - Green Ampt Watershed Parameters

Subbasin	Texture Code 1	Texture 1	HYDRO Class 1	PERCENT1	Texture Code 2	Texture 2	HYDRO Class 2	PERCENT2	Porosity	Effective Porosity	Soil Suction Head at the wetting front (mm) (Low)	Soil Suction Head at the wetting front (mm) (Average)	Soil Suction Head at the wetting front (mm) (High)	Saturated Hydraulic Conductity (mm/h)	Field Capacity Θ_o	Wilting Point Θ_w	Area Watershed (km2)	Area Watershed (ha)	Soil Area (ha)	Percentage of Watershed
Subbasin-1	FS	Fine Sand	A	100				0	0.437	0.417	49	99.5	150	120	0.062	0.024	15.129425	1512.9425	178.973	11.83%
Subbasin-1	LS	Loamy Sand	A	100				0	0.437	0.401	61	155.5	250	30	0.105	0.047	15.129425	1512.9425	29.3309	1.94%
Subbasin-1	L	Loam	B	100				0	0.463	0.434	89	219.5	350	3	0.232	0.116	15.129425	1512.9425	66.8735	4.42%
Subbasin-1	LS	Loamy Sand	B	100				0	0.437	0.401	61	155.5	250	30	0.105	0.047	15.129425	1512.9425	57.3464	3.79%
Subbasin-1	LS	Loamy Sand	C	100				0	0.437	0.401	61	155.5	250	30	0.105	0.047	15.129425	1512.9425	28.5138	1.88%
Subbasin-1	SIL	Silt Loam	C	100				0	0.501	0.486	170	170	170	7	0.284	0.135	15.129425	1512.9425	23.9731	1.58%
Subbasin-1	FS	Fine Sand	N	100				0	0.437	0.417	49	99.5	150	120	0.062	0.024	15.129425	1512.9425	38.9133	2.57%
Subbasin-1	S	Sand	N	100				0	0.437	0.417	49	99.5	150	120	0.062	0.024	15.129425	1512.9425	0.040196	0.00%
Subbasin-1	FS	Fine Sand	A	50	FS	Fine Sand	A	50	0.437	0.417	49	99.5	150	120	0.062	0.024	15.129425	1512.9425	14.63	0.97%
Subbasin-1	LS	Loamy Sand	A	50	VFSL	Very Fine Sandy Loam	C	50	0.445	0.4065	85.5	167.75	250	20.5	0.1475	0.066	15.129425	1512.9425	13.3459	0.88%
Subbasin-1	FS	Fine Sand	A	70	FS	Fine Sand	A	30	0.437	0.417	49	99.5	150	120	0.062	0.024	15.129425	1512.9425	360.751	23.84%
Subbasin-1	FS	Fine Sand	A	70	VFSL	Very Fine Sandy Loam	B	30	0.4418	0.4155	67.3	123.65	180	87.3	0.1004	0.0423	15.129425	1512.9425	58.8759	3.89%
Subbasin-1	FS	Fine Sand	A	70	FSL	Fine Sandy Loam	C	30	0.4418	0.4155	67.3	123.65	180	87.3	0.1004	0.0423	15.129425	1512.9425	57.7003	3.81%
Subbasin-1	FS	Fine Sand	A	70	LFS	Loamy Fine Sand	C	30	0.437	0.4122	52.6	116.3	180	93	0.0749	0.0309	15.129425	1512.9425	0.014173	0.00%
Subbasin-1	FS	Fine Sand	A	70	LS	Loamy Sand	C	30	0.437	0.4122	52.6	116.3	180	93	0.0749	0.0309	15.129425	1512.9425	37.0668	2.45%
Subbasin-1	FS	Fine Sand	A	70	VFSL	Very Fine Sandy Loam	C	30	0.4418	0.4155	67.3	123.65	180	87.3	0.1004	0.0423	15.129425	1512.9425	23.2464	1.54%
Subbasin-1	LFS	Loamy Fine Sand	A	70	VFSL	Very Fine Sandy Loam	B	30	0.4418	0.4043	75.7	162.85	250	24.3	0.1305	0.0584	15.129425	1512.9425	7.58032	0.50%
Subbasin-1	LFS	Loamy Fine Sand	A	70	FSL	Fine Sandy Loam	C	30	0.4418	0.4043	75.7	162.85	250	24.3	0.1305	0.0584	15.129425	1512.9425	17.3864	1.15%
Subbasin-1	LS	Loamy Sand	A	70	FS	Fine Sand	A	30	0.437	0.4058	57.4	138.7	220	57	0.0921	0.0401	15.129425	1512.9425	10.7979	0.71%
Subbasin-1	LS	Loamy Sand	A	70	LS	Loamy Sand	B	30	0.437	0.401	61	155.5	250	30	0.105	0.047	15.129425	1512.9425	20.271	1.34%
Subbasin-1	LS	Loamy Sand	A	70	FSL	Fine Sandy Loam	C	30	0.4418	0.4043	75.7	162.85	250	24.3	0.1305	0.0584	15.129425	1512.9425	5.34433	0.35%
Subbasin-1	FSL	Fine Sandy Loam	B	70	LS	Loamy Sand	C	30	0.4482	0.4087	95.3	172.65	250	16.7	0.1645	0.0736	15.129425	1512.9425	20.0996	1.33%
Subbasin-1	L	Loam	B	70	VFSL	Very Fine Sandy Loam	B	30	0.46	0.4274	95.3	207.65	320	5.4	0.2194	0.1067	15.129425	1512.9425	11.7043	0.77%
Subbasin-1	L	Loam	B	70			N	30	0.3241	0.3038	62.3	153.65	245	2.1	0.1624	0.0812	15.129425	1512.9425	40.9093	2.70%
Subbasin-1	LS	Loamy Sand	B	70	FS	Fine Sand	A	30	0.437	0.4058	57.4	138.7	220	57	0.0921	0.0401	15.129425	1512.9425	8.53354	0.56%
Subbasin-1	VFSL	Very Fine Sandy Loam	B	70	SIL	Silt Loam	C	30	0.4674	0.4342	128	177	226	9.8	0.2182	0.1	15.129425	1512.9425	15.0478	0.99%
Subbasin-1	FSL	Fine Sandy Loam	C	70	LFS	Loamy Fine Sand	A	30	0.4482	0.4087	95.3	172.65	250	16.7	0.1645	0.0736	15.129425	1512.9425	0.012515	0.00%
Subbasin-1	FSL	Fine Sandy Loam	C	70	L	Loam	B	30	0.456	0.4186	103.7	191.85	280	8.6	0.2026	0.0943	15.129425	1512.9425	123.841	8.19%
Subbasin-1	L	Loam	C	70	SIL	Silt Loam	C	30	0.4744	0.4496	113.3	204.65	296	4.2	0.2476	0.1217	15.129425	1512.9425	45.8023	3.03%
Subbasin-1	LFS	Loamy Fine Sand	C	70	FS	Fine Sand	A	30	0.437	0.4058	57.4	138.7	220	57	0.0921	0.0401	15.129425	1512.9425	11.5962	0.77%
Subbasin-1	LFS	Loamy Fine Sand	C	70	FSL	Fine Sandy Loam	B	30	0.4418	0.4043	75.7	162.85	250	24.3	0.1305	0.0584	15.129425	1512.9425	16.9563	1.12%
Subbasin-1	SIL	Silt Loam	C	70	L	Loam	C	30	0.4896	0.4704	145.7	184.85	224	5.8	0.2684	0.1293	15.129425	1512.9425	29.0139	1.92%

Loss Method - Green Ampt Watershed Parameters																			
Subbasin	Texture Code 1	Texture 1	HYDRO Class 1	PERCENT1	Texture Code 2	Texture 2	HYDRO Class 2	PERCENT2	Porosity	Effective Porosity	Soil Suction Head at the wetting front (mm) (Low)	Soil Suction Head at the wetting front (mm) (Average)	Soil Suction Head at the wetting front (mm) (High)	Saturated Hydraulic Conductity (mm/h)	Field Capacity Θ_o	Wilting Point Θ_w	Area Watershed (km2)	Area Watershed (ha)	Soil Area (ha)	Percentage of Watershed
Subbasin-1	SIL	Silt Loam	C	70	ORG	Organic	D	30	0.5187	0.5082	173	179	185	4.93	0.3368	0.1755	15.129425	1512.9425	134.309	8.88%
Subbasin-1	VFSL	Very Fine Sandy Loam	C	70	VFSL	Very Fine Sandy Loam	B	30	0.453	0.412	110	180	250	11	0.19	0.085	15.129425	1512.9425	4.13957	0.27%
Subbasin-2	FS	Fine Sand	A	100				0	0.437	0.417	49	99.5	150	120	0.062	0.024	3.196625	319.6625	75.4794	23.61%
Subbasin-2	L	Loam	B	100				0	0.463	0.434	89	219.5	350	3	0.232	0.116	3.196625	319.6625	36.3028	11.36%
Subbasin-2	LFS	Loamy Fine Sand	C	100				0	0.437	0.401	61	155.5	250	30	0.105	0.047	3.196625	319.6625	0.000774	0.00%
Subbasin-2	VFSL	Very Fine Sandy Loam	C	100				0	0.453	0.412	110	180	250	11	0.19	0.085	3.196625	319.6625	6.37271	1.99%
Subbasin-2	S	Sand	N	100				0	0.437	0.417	49	99.5	150	120	0.062	0.024	3.196625	319.6625	15.7743	4.93%
Subbasin-2	FS	Fine Sand	A	70	FS	Fine Sand	A	30	0.437	0.417	49	99.5	150	120	0.062	0.024	3.196625	319.6625	75.9902	23.77%
Subbasin-2	FS	Fine Sand	A	70	VFSL	Very Fine Sandy Loam	B	30	0.4418	0.4155	67.3	123.65	180	87.3	0.1004	0.0423	3.196625	319.6625	20.8506	6.52%
Subbasin-2	LFS	Loamy Fine Sand	A	70	VFSL	Very Fine Sandy Loam	B	30	0.4418	0.4043	75.7	162.85	250	24.3	0.1305	0.0584	3.196625	319.6625	0.175455	0.05%
Subbasin-2	LFS	Loamy Fine Sand	A	70	FSL	Fine Sandy Loam	C	30	0.4418	0.4043	75.7	162.85	250	24.3	0.1305	0.0584	3.196625	319.6625	1.73166	0.54%
Subbasin-2	LS	Loamy Sand	B	70	FSL	Fine Sandy Loam	C	30	0.4418	0.4043	75.7	162.85	250	24.3	0.1305	0.0584	3.196625	319.6625	18.2747	5.72%
Subbasin-2	LFS	Loamy Fine Sand	C	70	FS	Fine Sand	A	30	0.437	0.4058	57.4	138.7	220	57	0.0921	0.0401	3.196625	319.6625	63.0023	19.71%
Subbasin-2	LFS	Loamy Fine Sand	C	70	FSL	Fine Sandy Loam	B	30	0.4418	0.4043	75.7	162.85	250	24.3	0.1305	0.0584	3.196625	319.6625	2.66698	0.83%
Subbasin-2	LS	Loamy Sand	C	70	VFSL	Very Fine Sandy Loam	B	30	0.4418	0.4043	75.7	162.85	250	24.3	0.1305	0.0584	3.196625	319.6625	3.04061	0.95%

Land Cover, Percent Impervious, and Abstractions

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: 23-Jan-25

Standard Parameters

Class	Land Classification	Description	Percent Impervious	Canopy Storage (mm)	Depression Storage (mm)	Total Abstraction (mm)	Manning
1	Cultivated Crop Lands	Agriculture lands annual and perennial crop areas may have pasture lands based on the image scenes used.	0	2	8	10	0.55
2	Bare Soil Areas	Agriculture lands identified as bare soils within image scenes.	0	2	8	10	0.55
3	Grasslands/Shrubs	Lands containing grass, pasture lands, mixed with shrubs (<2m) rural areas.	0	3	8	11	0.55
4	Conifer Treed Forest	Dense conifer trees (~10m>) grouped into homogenous areas where the area is ~60% or greater tree enclosed, and the majority is conifer.	0	3	10	13	0.08
5	Deciduous Treed Forest	Dense deciduous trees (~10m>) grouped into homogenous areas where the area is ~60% or greater treed enclosed and the majority is deciduous.	0	3	10	13	0.08
6	Mixed Treed Forest	Forest areas with ~25-60% treed crown closure with mixed distribution of conifers and deciduous, having heights ~10m>.	0	3	10	13	0.08
7	Sparse Treed	Treed areas with heights ~2m> mixed conifer and deciduous at different heights and treed closure between 10–25%.	0	3	10	13	0.08
8	Wetlands	Wetland locations were digitized through orthophoto interpretation.	0	3	15	18	0.08
9	Open Water Areas	Permanent open water areas-based summer image scene.	100	0	0	0	0.035
10	Barren Areas	Areas of exposed rock, and beaches where vegetation is <25%.	0	1	2	3	0.045
12	Commercial/Industrial	High density mix buildings, roads and other paved areas where ~70% and greater where the surface areas are non-natural.	70	0	2	2	0.055
13	Residential	Medium density mix of buildings, roads, grass, shrubs and trees ~50%-70% of the areas is non-natural.	50	1	5	6	0.045
14	Settlement Open Areas	Areas predominantly open space ~ <50% of the areas is paved and the majority is parks, treed area, grasslands, recreational fields.	10	1	5	6	0.045
15	Roads	Paved arterial roads, collector, highways and freeways as determined and extracted from Ontario Road Network database.	100	0	1	1	0.015

Percent Impervious and Initial Abstraction Basin Parameters

Subbasin ID	gridcode	Land Cover	Description	Percent Impervious	Canopy Storage (mm)	Depression Storage (mm)	Total Abstraction (mm)	Area Watershed (km2)	Area Watershed (ha)	Land Cover Area (ha)	Impervious Area (ha)	Percentage of Watershed
Subbasin-11	1	Cultivated Crop Lands	Agriculture lands annual and perennial crop areas may have pasture lands based on the image scenes used.	0	2	8	10	15.129425	1512.9425	838.816	0.00	55.44%
Subbasin-11	2	Bare Soil Areas	Agriculture lands identified as bare soils within image scenes.	0	2	8	10	15.129425	1512.9425	146.93	0.00	9.71%
Subbasin-11	3	Grasslands/Shrubs	Lands containing grass, pasture lands, mixed with shrubs (<2m) rural areas.	0	3	8	11	15.129425	1512.9425	2.92	0.00	0.19%
Subbasin-11	4	Conifer Treed Forest	Dense conifer trees (~10m>) grouped into homogenous areas where the area is ~60% or greater tree enclosed, and the majority is conifer.	0	3	10	13	15.129425	1512.9425	20.9973	0.00	1.39%
Subbasin-11	5	Deciduous Treed Forest	Dense deciduous trees (~10m>) grouped into homogenous areas where the area is ~60% or greater treed enclosed and the majority is deciduous.	0	3	10	13	15.129425	1512.9425	243.269	0.00	16.08%
Subbasin-11	6	Mixed Treed Forest	Forest areas with ~25-60% treed crown closure with mixed distribution of conifers and deciduous, having heights ~10m>.	0	3	10	13	15.129425	1512.9425	87.6663	0.00	5.79%
Subbasin-11	7	Sparse Treed	Treed areas with heights ~2m> mixed conifer and deciduous at different heights and treed closure between 10–25%.	0	3	10	13	15.129425	1512.9425	4.4	0.00	0.29%
Subbasin-11	9	Open Water Areas	Permanent open water areas-based summer image scene.	100	0	0	0	15.129425	1512.9425	40.9336	40.93	2.71%
Subbasin-11	10	Barren Areas	Areas of exposed rock, and beaches where vegetation is <25%.	0	1	2	3	15.129425	1512.9425	1.2	0.00	0.08%
Subbasin-11	12	Commercial/Industrial	High density mix buildings, roads and other paved areas where ~70% and greater where the surface areas are non-natural.	70	0	2	2	15.129425	1512.9425	26.4223	18.50	1.75%
Subbasin-11	13	Residential	Medium density mix of buildings, roads, grass, shrubs and trees ~50%-70% of the areas is non-natural.	50	1	5	6	15.129425	1512.9425	59.4501	29.73	3.93%
Subbasin-11	15	Roads	Paved arterial roads, collector, highways and freeways as determined and extracted from Ontario Road Network database.	100	0	1	1	15.129425	1512.9425	39.9361	39.94	2.64%
Subbasin-12	1	Cultivated Crop Lands	Agriculture lands annual and perennial crop areas may have pasture lands based on the image scenes used.	0	2	8	10	3.196625	319.6625	132.265	0.00	41.38%
Subbasin-12	2	Bare Soil Areas	Agriculture lands identified as bare soils within image scenes.	0	2	8	10	3.196625	319.6625	8.6034	0.00	2.69%
Subbasin-12	4	Conifer Treed Forest	Dense conifer trees (~10m>) grouped into homogenous areas where the area is ~60% or greater tree enclosed, and the majority is conifer.	0	3	10	13	3.196625	319.6625	19.8854	0.00	6.22%
Subbasin-12	5	Deciduous Treed Forest	Dense deciduous trees (~10m>) grouped into homogenous areas where the area is ~60% or greater treed enclosed and the majority is deciduous.	0	3	10	13	3.196625	319.6625	61.0572	0.00	19.10%
Subbasin-12	6	Mixed Treed Forest	Forest areas with ~25-60% treed crown closure with mixed distribution of conifers and deciduous, having heights ~10m>.	0	3	10	13	3.196625	319.6625	76.0397	0.00	23.79%
Subbasin-12	7	Sparse Treed	Treed areas with heights ~2m> mixed conifer and deciduous at different heights and treed closure between 10–25%.	0	3	10	13	3.196625	319.6625	5.27567	0.00	1.65%
Subbasin-12	9	Open Water Areas	Permanent open water areas-based summer image scene.	100	0	0	0	3.196625	319.6625	0.266444	0.27	0.08%
Subbasin-12	12	Commercial/Industrial	High density mix buildings, roads and other paved areas where ~70% and greater where the surface areas are non-natural.	70	0	2	2	3.196625	319.6625	0.628866	0.44	0.20%
Subbasin-12	13	Residential	Medium density mix of buildings, roads, grass, shrubs and trees ~50%-70% of the areas is non-natural.	50	1	5	6	3.196625	319.6625	4.77119	2.39	1.49%
Subbasin-12	15	Roads	Paved arterial roads, collector, highways and freeways as determined and extracted from Ontario Road Network database.	100	0	1	1	3.196625	319.6625	10.8697	10.87	3.40%

Hydrologic Peak Flow Results - Summer Conditions

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: 26-Jan-24

Basin Model: Deer Creek Basin SCSUH

Areal Reduction Factor: 0.992

Peak Flow Results

1% AEP 24 Hour SCS Type 2

Hydrologic Element	Drainage Area	Peak Discharge	Time of Peak	Volume (mm)
Subbasin-1	15.1	7.1	1 January 2100, 16:00	29.41
Junction-1	15.1	7.1	1 January 2100, 16:00	29.41
Deer Creek Reservoir	15.1	5.5	1 January 2100, 20:15	27.28
Reach-1	15.1	5.5	1 January 2100, 21:45	26.93
Subbasin-2	3.2	1.8	1 January 2100, 14:15	31.56
Sink-1	18.3	6.5	1 January 2100, 21:15	27.74

2% AEP 24 Hour SCS Type 2

Hydrologic Element	Drainage Area	Peak Discharge	Time of Peak	Volume (mm)
Subbasin-1	15.13	5.850	1 January 2100, 16:00	25.7
Junction-1	15.13	5.850	1 January 2100, 16:00	25.7
Deer Creek Reservoir	15.13	4.540	1 January 2100, 20:30	23.7
Reach-1	15.13	4.540	1 January 2100, 22:30	23.4
Subbasin-2	3.20	1.670	1 January 2100, 14:15	28.7
Sink-1	18.33	5.450	1 January 2100, 21:45	24.4

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: 26-Jan-24

Basin Model: Deer Creek Basin SCSUH
Areal Reduction Factor: 0.992

Peak Flow Results

4% AEP 24 Hour SCS Type 2

Hydrologic Element	Drainage Area	Peak Discharge	Time of Peak	Volume (mm)
Subbasin-1	15.1	5.3	1 January 2100, 16:00	23.20
Junction-1	15.1	5.3	1 January 2100, 16:00	23.20
Deer Creek Reservoir	15.1	4.1	1 January 2100, 20:45	21.37
Reach-1	15.1	4.1	1 January 2100, 22:45	21.14
Subbasin-2	3.2	1.5	1 January 2100, 14:15	25.85
Sink-1	18.3	4.9	1 January 2100, 22:00	21.96

10% AEP 24 Hour SCS Type 2

Hydrologic Element	Drainage Area	Peak Discharge	Time of Peak	Volume (mm)
Subbasin-1	15.13	4.510	1 January 2100, 16:00	19.9
Junction-1	15.13	4.510	1 January 2100, 16:00	19.9
Deer Creek Reservoir	15.13	3.410	1 January 2100, 21:00	18.3
Reach-1	15.13	3.400	1 January 2100, 23:15	18.1
Subbasin-2	3.20	1.290	1 January 2100, 14:15	22.1
Sink-1	18.33	4.070	1 January 2100, 22:30	18.8

Hydrologic Peak Flow Results - Summer Conditions

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: 26-Jan-24

Basin Model: Deer Creek Basin SCSUH

Areal Reduction Factor: 0.992

Peak Flow Results

20% AEP 24 Hour SCS Type 2

Hydrologic Element	Drainage Area	Peak Discharge	Time of Peak	Volume (mm)
Subbasin-1	15.1	3.9	1 January 2100, 16:00	17.23
Junction-1	15.1	3.9	1 January 2100, 16:00	17.23
Deer Creek Reservoir	15.1	2.9	1 January 2100, 21:15	15.80
Reach-1	15.1	2.9	1 January 2100, 23:30	15.62
Subbasin-2	3.2	1.1	1 January 2100, 14:15	19.02
Sink-1	18.3	3.5	1 January 2100, 22:45	16.21

50% AEP 24 Hour SCS Type 2

Hydrologic Element	Drainage Area	Peak Discharge	Time of Peak	Volume (mm)
Subbasin-1	15.13	2.980	1 January 2100, 16:00	13.2
Junction-1	15.13	2.980	1 January 2100, 16:00	13.2
Deer Creek Reservoir	15.13	2.150	1 January 2100, 21:30	12.1
Reach-1	15.13	2.150	2 January 2100, 00:45	12.0
Subbasin-2	3.20	0.840	1 January 2100, 14:15	14.5
Sink-1	18.33	2.540	1 January 2100, 24:00	12.4

Hydrologic Peak Flow Results - Summer Conditions

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: 26-Jan-24

Basin Model: Deer Creek Basin SCSUH

Areal Reduction Factor: 0.992

Peak Flow Results

0.1% AEP 24 Hour SCS Type 2

Hydrologic Element	Drainage Area	Peak Discharge	Time of Peak	Volume (mm)
Subbasin-1	15.1	14.9	1 January 2100, 15:45	49.04
Junction-1	15.1	14.9	1 January 2100, 15:45	49.04
Deer Creek Reservoir	15.1	11.7	1 January 2100, 19:15	46.54
Reach-1	15.1	11.6	1 January 2100, 20:30	45.98
Subbasin-2	3.2	3.5	1 January 2100, 14:15	47.36
Sink-1	18.3	13.3	1 January 2100, 20:00	46.22

Hazel

Hydrologic Element	Drainage Area	Peak Discharge	Time of Peak	Volume (mm)
Subbasin-1	15.13	11.760	3 January 2100, 01:00	68.8
Junction-1	15.13	11.760	3 January 2100, 01:00	68.8
Deer Creek Reservoir	15.13	9.960	3 January 2100, 04:00	66.5
Reach-1	15.13	9.950	3 January 2100, 05:15	66.0
Subbasin-2	3.20	3.310	2 January 2100, 24:00	70.2
Sink-1	18.33	11.880	3 January 2100, 04:30	66.7

Hydrologic Peak Flow Results - Summer Conditions

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: 26-Jan-24

Basin Model: Deer Creek Basin SCSUH

Areal Reduction Factor: 1.0

Peak Flow Results

PMP 6 HR

Hydrologic Element	Drainage Area	Peak Discharge	Time of Peak	Volume (mm)
Subbasin-1	15.1	70.7	1 January 2100, 07:30	159.70
Junction-1	15.1	70.7	1 January 2100, 07:30	159.70
Deer Creek Reservoir	15.1	59.3	1 January 2100, 09:45	150.76
Reach-1	15.1	59.2	1 January 2100, 10:30	146.90
Subbasin-2	3.2	19.6	1 January 2100, 06:00	153.56
Sink-1	18.3	67.8	1 January 2100, 09:45	148.06

PMP 12 HR

Hydrologic Element	Drainage Area	Peak Discharge	Time of Peak	Volume (mm)
Subbasin-1	15.13	73.940	1 January 2100, 09:30	166.9
Junction-1	15.13	73.940	1 January 2100, 09:30	166.9
Deer Creek Reservoir	15.13	62.570	1 January 2100, 12:00	155.3
Reach-1	15.13	62.490	1 January 2100, 12:30	150.3
Subbasin-2	3.20	20.350	1 January 2100, 08:00	160.8
Sink-1	18.33	71.790	1 January 2100, 12:00	152.1

Appendix C

Hydraulic Assessment

Earth Dam Breach Estimates Input for Deer Creek Dam

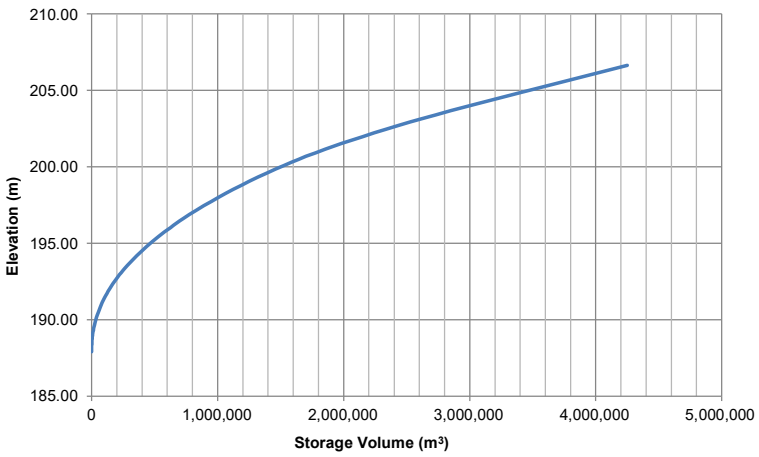
Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: February 9, 2025

Starting Water Surface Elevations

Flood Event	HEC-RAS Plan ID	Non-Breach Peak Flow (m^3/s)	U/S WS Elev. at Breach (m)	Dam Crest Elev. (m)	Dam Breach Invert Elev. (m)
Sunny Day		1.00	200.27	203.83	191.5
25-Year		4.19	200.45	203.83	191.5
100-Year		5.65	200.52	203.83	191.5
1000-Year		11.81	200.75	203.83	191.5
PMF		62.82	201.98	203.83	191.5
One Third		34.27	201.21	203.83	191.5

Reservoir Volume at Time of Failure

Elevation (m)	Storage (m^3)	Next Elevation (m)	Next Storage (m^3)
		187.91	0
187.91	0	188.08	6
188.08	6	188.15	38
188.15	38	188.23	141
188.23	141	188.31	307
188.31	307	188.39	556
188.39	556	188.47	903
188.47	903	188.62	1838
188.62	1838	188.76	3092
188.76	3092	188.90	4922
188.90	4922	189.03	6961
189.03	6961	189.18	9755
189.18	9755	189.34	13111
189.34	13111	189.49	16996
189.49	16996	189.83	26886
189.83	26886	190.16	38395
190.16	38395	190.49	51923
190.49	51923	190.81	67155
190.81	67155	191.13	83741
191.13	83741	191.42	100476
191.42	100476	191.72	119918
191.72	119918	192.02	141657
192.02	141657	192.34	167101
192.34	167101	192.66	194046
192.66	194046	192.95	221353
192.95	221353	193.23	248952
193.23	248952	193.56	284142
193.56	284142	194.18	356754
194.18	356754	194.79	436878
194.79	436878	195.44	529447
195.44	529447	196.04	625427
196.04	625427	196.61	724409
196.61	724409	197.17	829951
197.17	829951	197.75	950615
197.75	950615	198.26	1062101
198.26	1062101	198.79	1186667
198.79	1186667	199.33	1319860
199.33	1319860	199.85	1459228
199.85	1459228	200.29	1583175
200.29	1583175	200.69	1701881
200.69	1701881	201.15	1853459
201.15	1853459	201.53	1983056
201.53	1983056	201.98	2149188
201.98	2149188	202.46	2334150
202.46	2334150	202.93	2527064
202.93	2527064	203.40	2729287
203.40	2729287	204.12	3050279
204.12	3050279	205.64	3772910



Earth Dam Breach Estimates Input for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: February 9, 2025

205.64	3772910	206.64	4249688
206.64	4249688		0
			0
			0

Earth Dam Breach Estimates Input for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: February 9, 2025

Earth Dam Breach Estimates Summary for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

Key Breach Parameters						
Parameter	**Sunny Day**	**25-Year**	**100-Year**	**1000-Year**	**One Third**	**PMF**
Non-Breach Peak Flow (m^3/s)	1.00	4.19	5.65	11.81	34.27	62.82
Water Surface Elev. (m)	200.27	200.45	200.52	200.75	201.21	201.98
Dam Crest Elev. (m)	203.83	203.83	203.83	203.83	203.83	203.83
Breach Invert Elev. (m)	191.50	191.50	191.50	191.50	191.50	191.50
Reservoir Volume (m^3)	1,471,434	1,525,132	1,546,169	1,616,828	1,767,491	2,044,523
Breach Side Slopes (H:V)	0.9:1	0.9:1	0.9:1	0.9:1	0.9:1	0.9:1

Estimated Breach Formation Time, t_f (h)						
Method	**Sunny Day**	**25-Year**	**100-Year**	**1000-Year**	**One Third**	**PMF**
Froehlich (1995)	0.49	0.50	0.50	0.52	0.54	0.59
M&L-M	0.46	0.47	0.47	0.48	0.50	0.54
Von Thun and Gillette	0.80	0.80	0.79	0.79	0.78	0.77
Bureau of Reclamation	0.29	0.30	0.30	0.31	0.32	0.35
Froehlich (2008)	0.55	0.56	0.57	0.58	0.60	0.65
Average	0.52	0.53	0.53	0.53	0.55	0.58
Typical Range	0.5 to 2.0	0.5 to 2.0	0.5 to 2.0	0.5 to 2.0	0.5 to 2.0	0.5 to 2.0

Estimated Breach Bottom Width, B_{bot} (m)						
Method	**Sunny Day**	**25-Year**	**100-Year**	**1000-Year**	**One Third**	**PMF**
Froehlich (1995)	16.3	16.6	16.7	17.1	17.9	19.3
Von Thun and Gillette	16.9	17.4	17.6	18.1	19.3	21.2
Bureau of Reclamation	15.2	15.8	16.0	16.7	18.0	20.3
USACE	26.3	26.9	27.1	27.8	29.1	31.4
Froehlich (2008)	17.0	17.3	17.5	17.9	18.7	20.1
Average	18.3	18.8	18.9	19.5	20.6	22.5
Typical Maximum	35.1	35.8	36.1	37.0	38.8	41.9

Estimated Breach Peak Flow, Q_p (m^3/s)						
Method	**Sunny Day**	**25-Year**	**100-Year**	**1000-Year**	**One Third**	**PMF**
Froehlich (1995)	591.6	613.1	621.5	649.8	708.5	812.9
M&L-M	981.4	1004.3	1013.3	1042.9	1103.7	1209.4
Bureau of Reclamation	1060.7	1101.3	1117.3	1170.6	1280.5	1474.7
USACE	633.5	666.5	679.6	723.8	817.1	988.9
Froehlich (2008)	591.6	613.1	621.5	649.8	708.5	812.9
Average	771.7	799.7	810.7	847.3	923.7	1059.8

Estimated Total Peak Flow, Q_t (m^3/s)						
Method	**Sunny Day**	**25-Year**	**100-Year**	**1000-Year**	**One Third**	**PMF**
Froehlich (1995)	592.6	617.3	627.2	661.6	742.7	875.8
M&L-M	982.4	1008.5	1018.9	1054.7	1138.0	1272.2
Bureau of Reclamation	1061.7	1105.5	1122.9	1182.4	1314.8	1537.5
USACE	634.5	670.7	685.3	735.6	851.4	1051.7
Froehlich (2008)	592.6	617.3	627.2	661.6	742.7	875.8
Average	772.7	803.9	816.3	859.2	957.9	1122.6

25-Year Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

Inputs

Flood Event		25-Year	
Water Surface Elevation at Time of Breach		200.45	m
Dam Crest Elevation		203.83	m
Breach Invert Elevation		191.50	m
Non-Breach Peak Flow		4.19	m^3/s
Reservoir Volume at Time of Failure	V_w =	1,525,132	m^3
Reservoir Surface Area at Time of Failure	A_s =	N/A	ha
Water Depth from Breach Invert	h_w =	8.95	m
Height of Breach	h_b =	12.33	m
Overtopping Multiplier	K_o =	1.0	(1.4 Overtopping, 1.0 Piping)
Breach Side Slopes (Z:1)	Z =	0.9	(1.4 Overtopping, 0.9 Piping)
Offset Factor (Von Thun and Gillette)	C_b =	6.1	m (for $V_w < 1.23x10^6$ m^3)
Estimated Average Breach Width (USACE)	b_w =	26.9	m (between $1h_w$ and $5h_w$)
Soil Erodability (Von Thun and Gillette)		Erosion Resistant	
Type of Structure (M&L-M)		Earthfill	
Gravitational Accleration	g =	9.81	m/s^2

Froehlich (1995)

$B_{avg} = 0.1803K_oV_w^{0.32}h_b^{0.19}$ $t_f = 0.00254V_w^{0.53}h_b^{-0.90}$ $Q_p = 0.607V_w^{0.295}h_w^{1.24}$

Estimated Average Breach Width	B_{avg} =	27.67	m
Estimated Breach Bottom Width	B_{bot} =	16.57	m
Estimated Breach Formation Time	t_f =	0.50	hr
Estimated Breach Peak Flow	Q_p =	613.1	m^3/s
Estimated Total Peak Flow	Q_t =	617.3	m^3/s

MacDonald and Langridge-Monopolis (M&L-M, 1984)

$V_{er} = 0.0261(V_wh_w)^{0.769}$ (earthfills) $V_{er} = 0.00348(V_wh_w)^{0.852}$ (nonearthfills) $t_f = 0.0179V_{er}^{0.364}$ $Q_p = 1.154(V_wh_w)^{0.412}$

Volume of Embankment Material Eroded	V_{er} =	8008.59	m^3
Estimated Breach Formation Time	t_f =	0.47	hr
Estimated Breach Peak Flow	Q_p =	1004.3	m^3/s
Estimated Total Peak Flow	Q_t =	1008.5	m^3/s

Von Thun and Gillette (1990)

$B_{avg} = 2.5H_w + C_b$ $t_f = B_{avg}/4h_w + 61$ (easily erodible soils) $t_f = B_{avg}/4h_w$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	28.48	m
Estimated Breach Bottom Width	B_{bot} =	17.38	m
Estimated Breach Formation Time	t_f =	0.80	hr

Bureau of Reclamation (1982, 1988)

$B_{avg} = 3H_w$ $t_f = 0.011B_{avg}$ $Q_p = 19.1h_w^{1.85}$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	26.85	m
Estimated Breach Bottom Width	B_{bot} =	15.75	m
Estimated Breach Formation Time	t_f =	0.30	hr
Estimated Breach Peak Flow	Q_p =	1101.3	m^3/s
Estimated Total Peak Flow	Q_t =	1105.5	m^3/s

25-Year Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

US Army Corps of Engineers (USACE, 1997)

$Q_p = 0.296 b_w g^{0.5} h_w^{1.5}$

Estimated Breach Peak Flow	Q_p =	666.5	m^3/s
Estimated Total Peak Flow	Q_t =	670.7	m^3/s

Froehlich (2008)

$$B_{ave} = 0.27 \times K_0 \times V_w^{0.32} \times h_b^{0.04} \qquad t_f = \left(63.2 \times \sqrt{\frac{V_w}{g h_b^2}} \right) \Big/ 3600 \qquad Q_p = 0.607 \times V_w^{0.295} \times h_w^{1.24}$$

Estimated Average Breach Width	B_{avg} =	28.42	m
Estimated Breach Bottom Width	B_{bot} =	17.33	m
Estimated Breach Formation Time	t_f =	0.56	hr
Estimated Breach Peak Flow	Q_p =	613.1	m^3/s
Estimated Total Peak Flow	Q_t =	617.3	m^3/s

References:
1. Uncertainty Estimates for Breach Parameters and Peak Flow Prediction Equations by Tony Wahl, Hydraulic Engineer,
2. Hydrologic Engineering Requirements for Reservoirs, Manual EM 1110-2-1420, US Army Corps of Engineers, October 1997

100-Year Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

Inputs

Flood Event		100-Year	
Water Surface Elevation at Time of Breach		200.52	m
Dam Crest Elevation		203.83	m
Breach Invert Elevation		191.50	m
Non-Breach Peak Flow		5.65	m^3/s
Reservoir Volume at Time of Failure	V_w =	1,546,169	m^3
Reservoir Surface Area at Time of Failure	A_s =	N/A	ha
Water Depth from Breach Invert	h_w =	9.02	m
Height of Breach	h_b =	12.33	m
Overtopping Multiplier	K_o =	1.0	(1.4 Overtopping, 1.0 Piping)
Breach Side Slopes (Z:1)	Z =	0.9	(1.4 Overtopping, 0.9 Piping)
Offset Factor (Von Thun and Gillette)	C_b =	6.1	m (for $V_w < 1.23x10^6$ m^3)
Estimated Average Breach Width (USACE)	b_w =	27.1	m (between $1h_w$ and $5h_w$)
Soil Erodability (Von Thun and Gillette)		Erosion Resistant	
Type of Structure (M&L-M)		Earthfill	
Gravitational Accleration	g =	9.81	m/s^2

Froehlich (1995)

$B_{avg} = 0.1803K_oV_w^{0.32}h_b^{0.19}$ $t_f = 0.00254V_w^{0.53}h_b^{-0.90}$ $Q_p = 0.607V_w^{0.295}h_w^{1.24}$

Estimated Average Breach Width	B_{avg} =	27.79	m
Estimated Breach Bottom Width	B_{bot} =	16.69	m
Estimated Breach Formation Time	t_f =	0.50	hr
Estimated Breach Peak Flow	Q_p =	621.5	m^3/s
Estimated Total Peak Flow	Q_t =	627.2	m^3/s

MacDonald and Langridge-Monopolis (M&L-M, 1984)

$V_{er} = 0.0261(V_wh_w)^{0.769}$ (earthfills) $V_{er} = 0.00348(V_wh_w)^{0.852}$ (nonearthfills) $t_f = 0.0179V_{er}^{0.364}$ $Q_p = 1.154(V_wh_w)^{0.412}$

Volume of Embankment Material Eroded	V_{er} =	8142.03	m^3
Estimated Breach Formation Time	t_f =	0.47	hr
Estimated Breach Peak Flow	Q_p =	1013.3	m^3/s
Estimated Total Peak Flow	Q_t =	1018.9	m^3/s

Von Thun and Gillette (1990)

$B_{avg} = 2.5H_w + C_b$ $t_f = B_{avg}/4h_w + 61$ (easily erodible soils) $t_f = B_{avg}/4h_w$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	28.65	m
Estimated Breach Bottom Width	B_{bot} =	17.55	m
Estimated Breach Formation Time	t_f =	0.79	hr

Bureau of Reclamation (1982, 1988)

$B_{avg} = 3H_w$ $t_f = 0.011B_{avg}$ $Q_p = 19.1h_w^{1.85}$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	27.06	m
Estimated Breach Bottom Width	B_{bot} =	15.96	m
Estimated Breach Formation Time	t_f =	0.30	hr
Estimated Breach Peak Flow	Q_p =	1117.3	m^3/s
Estimated Total Peak Flow	Q_t =	1122.9	m^3/s

100-Year Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

US Army Corps of Engineers (USACE, 1997)

$Q_p = 0.296 b_w g^{0.5} h_w^{1.5}$

Estimated Breach Peak Flow	Q_p =	679.6	m^3/s
Estimated Total Peak Flow	Q_t =	685.3	m^3/s

Froehlich (2008)

$$B_{ave} = 0.27 \times K_0 \times V_w^{0.32} \times h_b^{0.04} \qquad t_f = \left(63.2 \times \sqrt{\frac{V_w}{g h_b^2}} \right) \Big/ 3600 \qquad Q_p = 0.607 \times V_w^{0.295} \times h_w^{1.24}$$

Estimated Average Breach Width	B_{avg} =	28.55	m
Estimated Breach Bottom Width	B_{bot} =	17.45	m
Estimated Breach Formation Time	t_f =	0.57	hr
Estimated Breach Peak Flow	Q_p =	621.5	m^3/s
Estimated Total Peak Flow	Q_t =	627.2	m^3/s

References:
1. Uncertainty Estimates for Breach Parameters and Peak Flow Prediction Equations by Tony Wahl, Hydraulic Engineer,
2. Hydrologic Engineering Requirements for Reservoirs, Manual EM 1110-2-1420, US Army Corps of Engineers, October 1997

1000-Year Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

Inputs

Flood Event		1000-Year	
Water Surface Elevation at Time of Breach		200.75	m
Dam Crest Elevation		203.83	m
Breach Invert Elevation		191.50	m
Non-Breach Peak Flow		11.81	m^3/s
Reservoir Volume at Time of Failure	V_w =	1,616,828	m^3
Reservoir Surface Area at Time of Failure	A_s =	N/A	ha
Water Depth from Breach Invert	h_w =	9.25	m
Height of Breach	h_b =	12.33	m
Overtopping Multiplier	K_o =	1.0	(1.4 Overtopping, 1.0 Piping)
Breach Side Slopes (Z:1)	Z =	0.9	(1.4 Overtopping, 0.9 Piping)
Offset Factor (Von Thun and Gillette)	C_b =	6.1	m (for $V_w < 1.23x10^6$ m^3)
Estimated Average Breach Width (USACE)	b_w =	27.8	m (between $1h_w$ and $5h_w$)
Soil Erodability (Von Thun and Gillette)		Erosion Resistant	
Type of Structure (M&L-M)		Earthfill	
Gravitational Accleration	g =	9.81	m/s^2

Froehlich (1995)

$B_{avg} = 0.1803K_oV_w^{0.32}h_b^{0.19}$ $t_f = 0.00254V_w^{0.53}h_b^{-0.90}$ $Q_p = 0.607V_w^{0.295}h_w^{1.24}$

Estimated Average Breach Width	B_{avg} =	28.19	m
Estimated Breach Bottom Width	B_{bot} =	17.09	m
Estimated Breach Formation Time	t_f =	0.52	hr
Estimated Breach Peak Flow	Q_p =	649.8	m^3/s
Estimated Total Peak Flow	Q_t =	661.6	m^3/s

MacDonald and Langridge-Monopolis (M&L-M, 1984)

$V_{er} = 0.0261(V_wh_w)^{0.769}$ (earthfills) $V_{er} = 0.00348(V_wh_w)^{0.852}$ (nonearthfills) $t_f = 0.0179V_{er}^{0.364}$ $Q_p = 1.154(V_wh_w)^{0.412}$

Volume of Embankment Material Eroded	V_{er} =	8591.44	m^3
Estimated Breach Formation Time	t_f =	0.48	hr
Estimated Breach Peak Flow	Q_p =	1042.9	m^3/s
Estimated Total Peak Flow	Q_t =	1054.7	m^3/s

Von Thun and Gillette (1990)

$B_{avg} = 2.5H_w + C_b$ $t_f = B_{avg}/4h_w + 61$ (easily erodible soils) $t_f = B_{avg}/4h_w$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	29.23	m
Estimated Breach Bottom Width	B_{bot} =	18.13	m
Estimated Breach Formation Time	t_f =	0.79	hr

Bureau of Reclamation (1982, 1988)

$B_{avg} = 3H_w$ $t_f = 0.011B_{avg}$ $Q_p = 19.1h_w^{1.85}$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	27.75	m
Estimated Breach Bottom Width	B_{bot} =	16.65	m
Estimated Breach Formation Time	t_f =	0.31	hr
Estimated Breach Peak Flow	Q_p =	1170.6	m^3/s
Estimated Total Peak Flow	Q_t =	1182.4	m^3/s

1000-Year Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

US Army Corps of Engineers (USACE, 1997)

$Q_p = 0.296 b_w g^{0.5} h_w^{1.5}$

Estimated Breach Peak Flow	Q_p =	723.8	m^3/s
Estimated Total Peak Flow	Q_t =	735.6	m^3/s

Froehlich (2008)

$$B_{ave} = 0.27 \times K_0 \times V_w^{0.32} \times h_b^{0.04} \qquad t_f = \left(63.2 \times \sqrt{\frac{V_w}{g h_b^2}} \right) \Big/ 3600 \qquad Q_p = 0.607 \times V_w^{0.295} \times h_w^{1.24}$$

Estimated Average Breach Width	B_{avg} =	28.96	m
Estimated Breach Bottom Width	B_{bot} =	17.86	m
Estimated Breach Formation Time	t_f =	0.58	hr
Estimated Breach Peak Flow	Q_p =	649.8	m^3/s
Estimated Total Peak Flow	Q_t =	661.6	m^3/s

References:
1. Uncertainty Estimates for Breach Parameters and Peak Flow Prediction Equations by Tony Wahl, Hydraulic Engineer,
2. Hydrologic Engineering Requirements for Reservoirs, Manual EM 1110-2-1420, US Army Corps of Engineers, October 1997

PMF Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

Inputs

Flood Event		PMF	
Water Surface Elevation at Time of Breach		201.98	m
Dam Crest Elevation		203.83	m
Breach Invert Elevation		191.50	m
Non-Breach Peak Flow		62.82	m^3/s
Reservoir Volume at Time of Failure	V_w =	2,044,523	m^3
Reservoir Surface Area at Time of Failure	A_s =	N/A	ha
Water Depth from Breach Invert	h_w =	10.48	m
Height of Breach	h_b =	12.33	m
Overtopping Multiplier	K_o =	1.0	(1.4 Overtopping, 1.0 Piping)
Breach Side Slopes (Z:1)	Z =	0.9	(1.4 Overtopping, 0.9 Piping)
Offset Factor (Von Thun and Gillette)	C_b =	6.1	m (for $V_w < 1.23x10^6$ m^3)
Estimated Average Breach Width (USACE)	b_w =	31.4	m (between $1h_w$ and $5h_w$)
Soil Erodability (Von Thun and Gillette)		Erosion Resistant	
Type of Structure (M&L-M)		Earthfill	
Gravitational Accleration	g =	9.81	m/s^2

Froehlich (1995)

$B_{avg} = 0.1803K_oV_w^{0.32}h_b^{0.19}$ $t_f = 0.00254V_w^{0.53}h_b^{-0.90}$ $Q_p = 0.607V_w^{0.295}h_w^{1.24}$

Estimated Average Breach Width	B_{avg} =	30.39	m
Estimated Breach Bottom Width	B_{bot} =	19.29	m
Estimated Breach Formation Time	t_f =	0.59	hr
Estimated Breach Peak Flow	Q_p =	812.9	m^3/s
Estimated Total Peak Flow	Q_t =	875.8	m^3/s

MacDonald and Langridge-Monopolis (M&L-M, 1984)

$V_{er} = 0.0261(V_wh_w)^{0.769}$ (earthfills) $V_{er} = 0.00348(V_wh_w)^{0.852}$ (nonearthfills) $t_f = 0.0179V_{er}^{0.364}$ $Q_p = 1.154(V_wh_w)^{0.412}$

Volume of Embankment Material Eroded	V_{er} =	11327.75	m^3
Estimated Breach Formation Time	t_f =	0.54	hr
Estimated Breach Peak Flow	Q_p =	1209.4	m^3/s
Estimated Total Peak Flow	Q_t =	1272.2	m^3/s

Von Thun and Gillette (1990)

$B_{avg} = 2.5H_w + C_b$ $t_f = B_{avg}/4h_w + 61$ (easily erodible soils) $t_f = B_{avg}/4h_w$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	32.30	m
Estimated Breach Bottom Width	B_{bot} =	21.20	m
Estimated Breach Formation Time	t_f =	0.77	hr

Bureau of Reclamation (1982, 1988)

$B_{avg} = 3H_w$ $t_f = 0.011B_{avg}$ $Q_p = 19.1h_w^{1.85}$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	31.44	m
Estimated Breach Bottom Width	B_{bot} =	20.34	m
Estimated Breach Formation Time	t_f =	0.35	hr
Estimated Breach Peak Flow	Q_p =	1474.7	m^3/s
Estimated Total Peak Flow	Q_t =	1537.5	m^3/s

PMF Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

US Army Corps of Engineers (USACE, 1997)

$Q_p = 0.296 b_w g^{0.5} h_w^{1.5}$

Estimated Breach Peak Flow	Q_p =	988.9	m^3/s
Estimated Total Peak Flow	Q_t =	1051.7	m^3/s

Froehlich (2008)

$$B_{ave} = 0.27 \times K_0 \times V_w^{0.32} \times h_b^{0.04} \qquad t_f = \left(63.2 \times \sqrt{\frac{V_w}{g h_b^2}} \right) \Big/ 3600 \qquad Q_p = 0.607 \times V_w^{0.295} \times h_w^{1.24}$$

Estimated Average Breach Width	B_{avg} =	31.22	m
Estimated Breach Bottom Width	B_{bot} =	20.12	m
Estimated Breach Formation Time	t_f =	0.65	hr
Estimated Breach Peak Flow	Q_p =	812.9	m^3/s
Estimated Total Peak Flow	Q_t =	875.8	m^3/s

References:
1. Uncertainty Estimates for Breach Parameters and Peak Flow Prediction Equations by Tony Wahl, Hydraulic Engineer,
2. Hydrologic Engineering Requirements for Reservoirs, Manual EM 1110-2-1420, US Army Corps of Engineers, October 1997

One Third Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

Inputs

Flood Event		One Third	
Water Surface Elevation at Time of Breach		201.21	m
Dam Crest Elevation		203.83	m
Breach Invert Elevation		191.50	m
Non-Breach Peak Flow		34.27	m^3/s
Reservoir Volume at Time of Failure	V_w =	1,767,491	m^3
Reservoir Surface Area at Time of Failure	A_s =	N/A	ha
Water Depth from Breach Invert	h_w =	9.71	m
Height of Breach	h_b =	12.33	m
Overtopping Multiplier	K_o =	1.0	(1.4 Overtopping, 1.0 Piping)
Breach Side Slopes (Z:1)	Z =	0.9	(1.4 Overtopping, 0.9 Piping)
Offset Factor (Von Thun and Gillette)	C_b =	6.1	m (for $V_w < 1.23x10^6$ m^3)
Estimated Average Breach Width (USACE)	b_w =	29.1	m (between $1h_w$ and $5h_w$)
Soil Erodability (Von Thun and Gillette)		Erosion Resistant	
Type of Structure (M&L-M)		Earthfill	
Gravitational Accleration	g =	9.81	m/s^2

Froehlich (1995)

$B_{avg} = 0.1803K_oV_w^{0.32}h_b^{0.19}$ $t_f = 0.00254V_w^{0.53}h_b^{-0.90}$ $Q_p = 0.607V_w^{0.295}h_w^{1.24}$

Estimated Average Breach Width	B_{avg} =	29.00	m
Estimated Breach Bottom Width	B_{bot} =	17.91	m
Estimated Breach Formation Time	t_f =	0.54	hr
Estimated Breach Peak Flow	Q_p =	708.5	m^3/s
Estimated Total Peak Flow	Q_t =	742.7	m^3/s

MacDonald and Langridge-Monopolis (M&L-M, 1984)

$V_{er} = 0.0261(V_wh_w)^{0.769}$ (earthfills) $V_{er} = 0.00348(V_wh_w)^{0.852}$ (nonearthfills) $t_f = 0.0179V_{er}^{0.364}$ $Q_p = 1.154(V_wh_w)^{0.412}$

Volume of Embankment Material Eroded	V_{er} =	9550.58	m^3
Estimated Breach Formation Time	t_f =	0.50	hr
Estimated Breach Peak Flow	Q_p =	1103.7	m^3/s
Estimated Total Peak Flow	Q_t =	1138.0	m^3/s

Von Thun and Gillette (1990)

$B_{avg} = 2.5H_w + C_b$ $t_f = B_{avg}/4h_w + 61$ (easily erodible soils) $t_f = B_{avg}/4h_w$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	30.38	m
Estimated Breach Bottom Width	B_{bot} =	19.28	m
Estimated Breach Formation Time	t_f =	0.78	hr

Bureau of Reclamation (1982, 1988)

$B_{avg} = 3H_w$ $t_f = 0.011B_{avg}$ $Q_p = 19.1h_w^{1.85}$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	29.13	m
Estimated Breach Bottom Width	B_{bot} =	18.03	m
Estimated Breach Formation Time	t_f =	0.32	hr
Estimated Breach Peak Flow	Q_p =	1280.5	m^3/s
Estimated Total Peak Flow	Q_t =	1314.8	m^3/s

One Third Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

US Army Corps of Engineers (USACE, 1997)

$Q_p = 0.296 b_w g^{0.5} h_w^{1.5}$

Estimated Breach Peak Flow	Q_p =	817.1	m^3/s
Estimated Total Peak Flow	Q_t =	851.4	m^3/s

Froehlich (2008)

$$B_{ave} = 0.27 \times K_0 \times V_w^{0.32} \times h_b^{0.04} \qquad t_f = \left(63.2 \times \sqrt{\frac{V_w}{g h_b^2}} \right) \Big/ 3600 \qquad Q_p = 0.607 \times V_w^{0.295} \times h_w^{1.24}$$

Estimated Average Breach Width	B_{avg} =	29.80	m
Estimated Breach Bottom Width	B_{bot} =	18.70	m
Estimated Breach Formation Time	t_f =	0.60	hr
Estimated Breach Peak Flow	Q_p =	708.5	m^3/s
Estimated Total Peak Flow	Q_t =	742.7	m^3/s

References:
1. Uncertainty Estimates for Breach Parameters and Peak Flow Prediction Equations by Tony Wahl, Hydraulic Engineer,
2. Hydrologic Engineering Requirements for Reservoirs, Manual EM 1110-2-1420, US Army Corps of Engineers, October 1997

Sunny Day Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

Inputs

Flood Event		Sunny Day	
Water Surface Elevation at Time of Breach		200.27	m
Dam Crest Elevation		203.83	m
Breach Invert Elevation		191.50	m
Non-Breach Peak Flow		1.00	m^3/s
Reservoir Volume at Time of Failure	V_w =	1,471,434	m^3
Reservoir Surface Area at Time of Failure	A_s =	N/A	ha
Water Depth from Breach Invert	h_w =	8.77	m
Height of Breach	h_b =	12.33	m
Overtopping Multiplier	K_o =	1.0	(1.4 Overtopping, 1.0 Piping)
Breach Side Slopes (Z:1)	Z =	0.9	(1.4 Overtopping, 0.9 Piping)
Offset Factor (Von Thun and Gillette)	C_b =	6.1	m (for $V_w < 1.23x10^6$ m^3)
Estimated Average Breach Width (USACE)	b_w =	26.3	m (between $1h_w$ and $5h_w$)
Soil Erodability (Von Thun and Gillette)		Erosion Resistant	
Type of Structure (M&L-M)		Earthfill	
Gravitational Accleration	g =	9.81	m/s^2

Froehlich (1995)

$B_{avg} = 0.1803K_oV_w^{0.32}h_b^{0.19}$ $t_f = 0.00254V_w^{0.53}h_b^{-0.90}$ $Q_p = 0.607V_w^{0.295}h_w^{1.24}$

Estimated Average Breach Width	B_{avg} =	27.35	m
Estimated Breach Bottom Width	B_{bot} =	16.25	m
Estimated Breach Formation Time	t_f =	0.49	hr
Estimated Breach Peak Flow	Q_p =	591.6	m^3/s
Estimated Total Peak Flow	Q_t =	592.6	m^3/s

MacDonald and Langridge-Monopolis (M&L-M, 1984)

$V_{er} = 0.0261(V_wh_w)^{0.769}$ (earthfills) $V_{er} = 0.00348(V_wh_w)^{0.852}$ (nonearthfills) $t_f = 0.0179V_{er}^{0.364}$ $Q_p = 1.154(V_wh_w)^{0.412}$

Volume of Embankment Material Eroded	V_{er} =	7670.08	m^3
Estimated Breach Formation Time	t_f =	0.46	hr
Estimated Breach Peak Flow	Q_p =	981.4	m^3/s
Estimated Total Peak Flow	Q_t =	982.4	m^3/s

Von Thun and Gillette (1990)

$B_{avg} = 2.5H_w + C_b$ $t_f = B_{avg}/4h_w + 61$ (easily erodible soils) $t_f = B_{avg}/4h_w$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	28.03	m
Estimated Breach Bottom Width	B_{bot} =	16.93	m
Estimated Breach Formation Time	t_f =	0.80	hr

Bureau of Reclamation (1982, 1988)

$B_{avg} = 3H_w$ $t_f = 0.011B_{avg}$ $Q_p = 19.1h_w^{1.85}$ (erosion resistant soils)

Estimated Average Breach Width	B_{avg} =	26.31	m
Estimated Breach Bottom Width	B_{bot} =	15.21	m
Estimated Breach Formation Time	t_f =	0.29	hr
Estimated Breach Peak Flow	Q_p =	1060.7	m^3/s
Estimated Total Peak Flow	Q_t =	1061.7	m^3/s

Sunny Day Earth Dam Breach Estimates for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: Sunday, February 9, 2025

US Army Corps of Engineers (USACE, 1997)

$Q_p = 0.296 b_w g^{0.5} h_w^{1.5}$

Estimated Breach Peak Flow	Q_p =	633.5	m^3/s
Estimated Total Peak Flow	Q_t =	634.5	m^3/s

Froehlich (2008)

$$B_{ave} = 0.27 \times K_0 \times V_w^{0.32} \times h_b^{0.04} \qquad t_f = \left(63.2 \times \sqrt{\frac{V_w}{g h_b^2}} \right) \Big/ 3600 \qquad Q_p = 0.607 \times V_w^{0.295} \times h_w^{1.24}$$

Estimated Average Breach Width	B_{avg} =	28.10	m
Estimated Breach Bottom Width	B_{bot} =	17.00	m
Estimated Breach Formation Time	t_f =	0.55	hr
Estimated Breach Peak Flow	Q_p =	591.6	m^3/s
Estimated Total Peak Flow	Q_t =	592.6	m^3/s

References:
1. Uncertainty Estimates for Breach Parameters and Peak Flow Prediction Equations by Tony Wahl, Hydraulic Engineer,
2. Hydrologic Engineering Requirements for Reservoirs, Manual EM 1110-2-1420, US Army Corps of Engineers, October 1997

Sunny Day												
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	1	188.13	200.27		200.27	0	0	1591.25	205.8	0
Reach 1	11048	Max WS	1	190.96	200.27	191.07	200.27	0	0	1340.1	348.31	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	1	185.59	185.85		185.86	0.001825	0.39	2.57	14.22	0.29
Reach 1	10747	Max WS	1	185.06	185.67		185.68	0.000424	0.35	2.83	5.69	0.16
Reach 1	10682	Max WS	1	184.94	185.52		185.56	0.003374	0.87	1.15	2.62	0.42
Reach 1	10591	Max WS	1	184.75	185.22		185.25	0.00334	0.77	1.3	3.84	0.42
Reach 1	10453	Max WS	1	184.46	184.92		184.94	0.001262	0.51	1.96	5.35	0.27
Reach 1	10316	Max WS	1	184.17	184.66		184.69	0.002373	0.7	1.43	3.79	0.36
Reach 1	10221	Max WS	1	183.98	184.45		184.48	0.002027	0.65	1.55	4.11	0.34
Reach 1	10093	Max WS	1	183.71	184.18		184.2	0.002283	0.66	1.51	4.29	0.35
Reach 1	9951	Max WS	1	183.42	183.94		183.95	0.001239	0.54	1.87	4.5	0.27
Reach 1	9814	Max WS	1	183.13	183.66		183.69	0.002659	0.76	1.32	3.22	0.38
Reach 1	9599	Max WS	1	182.67	183.13		183.15	0.002265	0.67	1.49	4.06	0.35
Reach 1	9362	Max WS	1	182.17	182.69		182.71	0.001513	0.59	1.68	3.86	0.29
Reach 1	9239	Max WS	1	181.92	182.46		182.48	0.002135	0.7	1.43	3.26	0.34
Reach 1	9104	Max WS	1	181.63	182.23		182.25	0.001294	0.58	1.71	3.53	0.27
Reach 1	9028	Max WS	1	181.48	182.05		182.09	0.003034	0.84	1.2	2.6	0.39
Reach 1	8891	Max WS	1	181.18	181.69		181.72	0.002331	0.71	1.42	3.55	0.36
Reach 1	8813	Max WS	1	181.02	181.54		181.56	0.001627	0.6	1.65	4	0.3
Reach 1	8671	Max WS	1	180.73	181.27		181.29	0.002168	0.69	1.44	3.44	0.34
Reach 1	8591	Max WS	1	180.57	180.98		181.02	0.0046	0.88	1.14	3.44	0.49
Reach 1	8323	Max WS	5	178.77	180.25		180.26	0.000593	0.56	8.97	12.06	0.21
Reach 1	7976	Max WS	5	178.66	180.03		180.05	0.000655	0.59	8.53	11.38	0.22
Reach 1	7762	Max WS	5	178.62	179.88		179.9	0.000697	0.61	8.22	10.89	0.22
Reach 1	7619	Max WS	5	178.59	179.79		179.8	0.00066	0.59	8.42	11.13	0.22
Reach 1	7217	Max WS	5	178.46	179.61		179.62	0.000271	0.4	12.39	15.2	0.14
Reach 1	7061	Max WS	5	178.42	179.56		179.57	0.000356	0.47	10.55	12.22	0.16
Reach 1	6824	Max WS	5	178.25	179.51		179.51	0.000109	0.29	17.37	17.82	0.09
Reach 1	6733	Max WS	5	178.16	179.5		179.5	0.00012	0.3	16.42	16.54	0.1
Reach 1	6684	Max WS	5	178.11	179.49		179.5	0.000169	0.37	13.58	12.99	0.11
Reach 1	6646	Max WS	5	178.06	179.49	178.53	179.49	0.000114	0.31	16.21	15.19	0.1
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	5	178.05	179.47		179.48	0.000153	0.35	14.2	13.54	0.11
Reach 1	6539	Max WS	5	178.07	179.46		179.47	0.000239	0.44	11.39	10.55	0.13
Reach 1	6496	Max WS	5	178.07	179.45		179.46	0.000208	0.41	12.15	11.24	0.13
Reach 1	6400	Max WS	5	178.09	179.44		179.45	0.000056	0.22	22.82	21.48	0.07
Reach 1	6304	Max WS	5	178.11	179.44		179.44	0.000099	0.29	17.48	16.69	0.09
Reach 1	6214	Max WS	5	178.12	179.42		179.43	0.000171	0.37	13.49	12.94	0.12
Reach 1	5812	Max WS	5	178.08	179.32		179.33	0.00031	0.48	10.49	10.68	0.15
Reach 1	5636	Max WS	5	177.99	179.29		179.29	0.000139	0.32	15.58	16.3	0.1
Reach 1	5444	Max WS	5	177.93	179.25		179.26	0.0002	0.38	13.01	13.47	0.12
Reach 1	5148	Max WS	5	177.83	179.18		179.19	0.000284	0.44	11.46	12.8	0.15
Reach 1	4980	Max WS	5	177.77	179.13		179.14	0.0003	0.44	11.34	13	0.15
Reach 1	4759	Max WS	5	177.67	179.02		179.04	0.000585	0.57	8.74	11.11	0.21
Reach 1	4647	Max WS	5	177.65	178.95		178.97	0.000677	0.59	8.41	11.27	0.22
Reach 1	4587	Max WS	5	177.62	178.92		178.93	0.000535	0.55	9.05	11.37	0.2
Reach 1	4542	Max WS	5	177.63	178.88	178.35	178.9	0.000761	0.61	8.15	11.33	0.23
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	5	177.56	178.85		178.87	0.000564	0.53	9.51	13.55	0.2
Reach 1	4441	Max WS	5	177.54	178.82		178.84	0.000499	0.52	9.7	12.99	0.19
Reach 1	4399	Max WS	5	177.53	178.8		178.81	0.000594	0.57	8.74	11.31	0.21
Reach 1	4291	Max WS	5	177.47	178.74		178.75	0.000466	0.5	9.95	13.17	0.18
Reach 1	3909	Max WS	5	177.26	178.56		178.57	0.000381	0.48	10.49	12.91	0.17
Reach 1	3757	Max WS	5	177.17	178.51		178.52	0.000263	0.4	12.37	14.83	0.14
Reach 1	3644	Max WS	5	177.14	178.5		178.5	0.000179	0.34	14.53	16.67	0.12
Reach 1	3510	Max WS	5	177.09	178.47		178.48	0.000258	0.41	12.13	13.84	0.14
Reach 1	3356	Max WS	5	177.02	178.43		178.44	0.000267	0.44	11.43	12.08	0.14
Reach 1	3246	Max WS	5	176.97	178.41		178.41	0.000201	0.39	12.89	13.24	0.13
Reach 1	3047	Max WS	5	176.86	178.03		178.05	0.00344	0.61	8.21	10.41	0.22
Reach 1	2950	Max WS	5	176.81	177.86		177.87	0.000326	0.4	12.46	17.96	0.15
Reach 1	2757	Max WS	5	176.72	177.8		177.81	0.000337	0.42	11.8	16	0.16
Reach 1	2652	Max WS	5	176.66	177.76		177.77	0.000309	0.41	12.07	15.82	0.15
Reach 1	2516	Max WS	5	176.61	177.69		177.71	0.000608	0.57	8.73	11.56	0.21
Reach 1	2363	Max WS	5	176.53	177.6		177.62	0.000588	0.58	8.64	10.89	0.21
Reach 1	2172	Max WS	5	176.44	177.48		177.5	0.000628	0.59	8.41	10.59	0.21
Reach 1	2044	Max WS	5	176.36	177.42		177.43	0.00045	0.51	9.74	12.06	0.18
Reach 1	1898	Max WS	5	176.29	177.33		177.35	0.000648	0.61	8.21	10.19	0.22
Reach 1	1813	Max WS	5	176.25	177.25		177.28	0.001014	0.74	6.77	8.72	0.27
Reach 1	1642	Max WS	5	176.17	177.14		177.15	0.000506	0.54	9.34	11.83	0.19
Reach 1	1552	Max WS	5	176.13	177.1		177.11	0.000432	0.48	10.46	14.18	0.18
Reach 1	1510	Max WS	5	176.11	177.08		177.09	0.000333	0.45	11.19	13.65	0.16
Reach 1	1411	Max WS	5	176.06	177.04	176.4	177.05	0.000434	0.51	9.88	12.17	0.18
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	5	176.04	177.03		177.04	0.000305	0.43	11.71	14.34	0.15
Reach 1	1329	Max WS	5	176.02	177		177.02	0.000616	0.6	8.28	9.96	0.21
Reach 1	1282	Max WS	5	175.99	176.99		177	0.000377	0.48	10.38	12.34	0.17
Reach 1	1207	Max WS	5	175.96	176.94		176.96	0.000605	0.59	8.4	10.25	0.21
Reach 1	1099	Max WS	5	175.91	176.88		176.9	0.000617	0.58	8.58	11.11	0.21
Reach 1	1005	Max WS	5	175.86	176.84		176.85	0.000414	0.48	10.36	13.34	0.17
Reach 1	890	Max WS	5	175.81	176.78	176.17	176.8	0.000499	0.52	9.61	12.59	0.19

Sunny Day with Dam Breach												
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	0.99	188.13	200.26		200.26	0	0	1591	205.79	0
Reach 1	11048	Max WS	0.99	190.96	200.26	191.07	200.26	0	0	1339.83	348.3	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	300.04	185.59	190.36		190.71	0.004732	2.63	114.24	34.69	0.46
Reach 1	10747	Max WS	297.8	185.06	189.88		190.13	0.002485	3.49	192.43	65.24	0.52
Reach 1	10682	Max WS	297.24	184.94	189.77		189.92	0.002634	3.13	211.19	71.08	0.47
Reach 1	10591	Max WS	297.84	184.75	189.45		189.71	0.003127	3.69	192.71	75.1	0.56
Reach 1	10453	Max WS	297.21	184.46	189.11		189.33	0.002136	3.22	208.97	73.42	0.49
Reach 1	10316	Max WS	296.1	184.17	188.79		188.97	0.002834	3.26	210.53	82.29	0.5
Reach 1	10221	Max WS	294.84	183.98	188.52		188.71	0.002617	3.26	211.86	80.59	0.5
Reach 1	10093	Max WS	294.17	183.71	188.23		188.4	0.002147	2.99	221.27	79.27	0.47
Reach 1	9951	Max WS	294.13	183.42	187.88		188.08	0.002508	3.23	205.17	74.21	0.51
Reach 1	9814	Max WS	293.7	183.13	187.59		187.73	0.002291	2.91	229.41	83.54	0.46
Reach 1	9599	Max WS	292.91	182.67	187.12		187.27	0.002008	2.81	232.11	83.81	0.45
Reach 1	9362	Max WS	290.92	182.17	186.12		186.45	0.005508	4.25	161.87	75.1	0.71
Reach 1	9239	Max WS	289.99	181.92	185.85		185.9	0.001344	2.04	342.05	149.09	0.35
Reach 1	9104	Max WS	289.51	181.63	185.58		185.68	0.002074	2.62	257.02	103.39	0.44
Reach 1	9028	Max WS	289.39	181.48	185.32		185.47	0.003862	3.25	204.64	96.23	0.56
Reach 1	8891	Max WS	289.04	181.18	184.93		185.03	0.002356	2.73	273.41	136.6	0.48
Reach 1	8813	Max WS	288.94	181.02	184.42		184.78	0.007013	4.4	148.28	71.78	0.81
Reach 1	8671	Max WS	286.82	180.73	183.67		183.82	0.005139	3.36	208.35	123.13	0.66
Reach 1	8591	Max WS	278.65	180.57	183.38		183.46	0.003397	2.7	271.53	178.71	0.54
Reach 1	8323	Max WS	253.5	178.77	182.88		182.91	0.000648	1.35	563.46	449.79	0.26
Reach 1	7976	Max WS	222.51	178.66	182.66		182.69	0.000599	1.33	507.18	400.51	0.25
Reach 1	7762	Max WS	202.2	178.62	182.51		182.55	0.000679	1.37	405.47	298.79	0.27
Reach 1	7619	Max WS	158.99	178.59	182.45		182.47	0.000314	0.96	491.23	375.5	0.18
Reach 1	7217	Max WS	141.91	178.46	182.34		182.36	0.000258	0.92	427.98	326.12	0.17
Reach 1	7061	Max WS	139.89	178.42	182.3		182.32	0.000323	1.02	409.44	330.05	0.18
Reach 1	6824	Max WS	138.94	178.25	182.23		182.25	0.000268	0.96	379.84	298.47	0.17
Reach 1	6733	Max WS	138.9	178.16	182.21		182.23	0.000222	0.89	437.33	334.31	0.16
Reach 1	6684	Max WS	138.86	178.11	182.2		182.21	0.000255	0.94	445.54	340.93	0.16
Reach 1	6646	Max WS	138.85	178.06	182.1	180.51	182.3	0.001163	2.06	89.84	350.43	0.36
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	117.95	178.05	181.57		181.86	0.002205	2.43	61.32	413.15	0.47
Reach 1	6539	Max WS	117.62	178.07	181.59		181.6	0.000388	0.96	319.91	328.27	0.19
Reach 1	6496	Max WS	117.91	178.07	181.57		181.59	0.00031	0.84	296.34	278.89	0.17
Reach 1	6400	Max WS	117.67	178.09	181.54		181.56	0.000265	0.9	293.45	225.94	0.17
Reach 1	6304	Max WS	117.97	178.11	181.52		181.53	0.000197	0.74	331.4	266.3	0.14
Reach 1	6214	Max WS	109.26	178.12	181.5		181.51	0.000221	0.71	344.75	308.42	0.15
Reach 1	5812	Max WS	101.81	178.08	181.35		181.38	0.000435	0.98	295.72	310.6	0.21
Reach 1	5636	Max WS	99.56	177.99	181.31		181.32	0.000147	0.63	420.97	309.06	0.12
Reach 1	5444	Max WS	98.02	177.93	181.27		181.28	0.000276	0.87	293.87	222.24	0.17
Reach 1	5148	Max WS	96.14	177.83	181.13		181.17	0.000551	1.13	206.66	193.36	0.24
Reach 1	4980	Max WS	95.69	177.77	181.06		181.08	0.000389	0.99	255.99	215.96	0.2
Reach 1	4759	Max WS	95.5	177.67	180.97		180.99	0.000445	1.04	252.14	223.61	0.21
Reach 1	4647	Max WS	95.47	177.65	180.92		180.94	0.000477	1.06	244.48	223.59	0.22
Reach 1	4587	Max WS	95.45	177.62	180.89		180.91	0.000358	0.93	266.47	210.93	0.19
Reach 1	4542	Max WS	95.4	177.63	180.75	180.15	180.98	0.002377	2.29	59.32	203.32	0.49
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	89.56	177.56	180.5		180.71	0.002248	2.13	54.15	192.72	0.47
Reach 1	4441	Max WS	83.07	177.54	180.49		180.5	0.000261	0.7	243.64	246.88	0.16
Reach 1	4399	Max WS	82.87	177.53	180.48		180.49	0.000206	0.61	278.12	276.87	0.14
Reach 1	4291	Max WS	81.8	177.47	180.47		180.47	0.000142	0.55	337.3	364.11	0.12
Reach 1	3909	Max WS	79.04	177.26	180.39		180.4	0.000208	0.7	285.56	362.95	0.15
Reach 1	3757	Max WS	78.18	177.17	180.35		180.36	0.000274	0.8	206.89	219.86	0.17
Reach 1	3644	Max WS	77.92	177.14	180.33		180.34	0.000193	0.67	236.72	213.9	0.14
Reach 1	3510	Max WS	77.42	177.09	180.29		180.32	0.000443	1.03	176.13	153.86	0.21
Reach 1	3356	Max WS	75.82	177.02	180.2		180.24	0.000624	1.16	151.66	151.57	0.25
Reach 1	3246	Max WS	75.4	176.97	180.16		180.18	0.000338	0.89	215.79	198.83	0.18
Reach 1	3047	Max WS	74.41	176.86	180.06		180.07	0.000663	0.54	214.95	209.68	0.11
Reach 1	2950	Max WS	73.87	176.81	180.01		180.03	0.000288	0.85	178.62	167.1	0.17
Reach 1	2757	Max WS	73.35	176.72	179.95		179.98	0.000304	0.88	183.75	175.18	0.18
Reach 1	2652	Max WS	73.37	176.66	179.91		179.94	0.00034	0.9	151.59	127.12	0.19
Reach 1	2516	Max WS	72.9	176.61	179.85		179.89	0.000538	1.13	133.69	132.44	0.23
Reach 1	2363	Max WS	72.28	176.53	179.76		179.8	0.000637	1.14	150.93	179.72	0.25
Reach 1	2172	Max WS	71.03	176.44	179.64		179.68	0.000622	1.15	150.27	175.25	0.24
Reach 1	2044	Max WS	71.01	176.36	179.58		179.6	0.000415	0.96	202.23	259.44	0.2
Reach 1	1898	Max WS	70.34	176.29	179.46		179.52	0.000935	1.31	135.29	223.15	0.29
Reach 1	1813	Max WS	70.06	176.25	179.4		179.44	0.000781	1.21	162.04	233.03	0.26
Reach 1	1642	Max WS	69.57	176.17	179.26		179.32	0.000765	1.3	125.49	214.06	0.27
Reach 1	1552	Max WS	69.47	176.13	179.22		179.25	0.0004	0.92	173.94	226.68	0.2
Reach 1	1510	Max WS	69.48	176.11	179.21		179.23	0.000321	0.92	206.49	243.54	0.18
Reach 1	1411	Max WS	69.33	176.06	179.07	177.73	179.23	0.001255	1.8	40.56	203.45	0.36
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	69.35	176.04	179.1		179.21	0.00084	1.49	48.1	244.74	0.29
Reach 1	1329	Max WS	69.32	176.02	178.97		179.2	0.002809	2.13	35.38	252.27	0.5
Reach 1	1282	Max WS	69.08	175.99	178.93		178.96	0.000546	1.06	179.93	306.6	0.23
Reach 1	1207	Max WS	68.95	175.96	178.88		178.92	0.000707	1.16	175.44	329.84	0.25
Reach 1	1099	Max WS	68.91	175.91	178.82		178.85	0.000543	1.04	196.63	344.95	0.22
Reach 1	1005	Max WS	68.88	175.86	178.77		178.8	0.000437	1.02	194.94	297.42	0.21
Reach 1	890	Max WS	68.86	175.81	178.72	177.47	178.75	0.000498	1	191.69	273.6	0.21

25-Year Flood												
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	4.2	188.13	200.45		200.45	0	0	1629.45	206.51	0
Reach 1	11048	Max WS	4.2	190.96	200.45	191.18	200.45	0	0	1382.12	351.5	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	4.2	185.59	186.4		186.41	0.000286	0.36	11.67	17.89	0.14
Reach 1	10747	Max WS	4.2	185.06	186.32		186.34	0.000526	0.62	6.75	6.29	0.19
Reach 1	10682	Max WS	4.2	184.94	186.07		186.19	0.004942	1.5	3.05	4.94	0.51
Reach 1	10591	Max WS	4.2	184.75	185.74		185.81	0.003127	1.2	3.49	4.57	0.44
Reach 1	10453	Max WS	4.2	184.46	185.49		185.52	0.001101	0.78	5.36	6.42	0.27
Reach 1	10316	Max WS	4.2	184.17	185.21		185.27	0.002519	1.12	3.73	4.52	0.4
Reach 1	10221	Max WS	4.2	183.98	185		185.05	0.002079	1.03	4.07	4.92	0.36
Reach 1	10093	Max WS	4.2	183.71	184.77		184.81	0.001721	0.92	4.55	5.9	0.34
Reach 1	9951	Max WS	4.2	183.42	184.57		184.6	0.001184	0.83	5.05	5.58	0.28
Reach 1	9814	Max WS	4.2	183.13	184.24		184.32	0.003025	1.22	3.45	4.4	0.42
Reach 1	9599	Max WS	4.2	182.67	183.75		183.8	0.001815	0.97	4.34	5.25	0.34
Reach 1	9362	Max WS	4.2	182.17	183.37		183.41	0.0014	0.91	4.64	4.95	0.29
Reach 1	9239	Max WS	4.2	181.92	183.15		183.21	0.001963	1.03	4.49	15.44	0.34
Reach 1	9104	Max WS	4.2	181.63	182.97		183	0.001054	0.82	9.79	71.68	0.26
Reach 1	9028	Max WS	4.2	181.48	182.81	182.24	182.87	0.002618	1.17	5.29	44.58	0.37
Reach 1	8891	Max WS	4.19	181.18	182.59		182.62	0.000939	0.77	7.08	37.83	0.25
Reach 1	8813	Max WS	4.19	181.02	182.55		182.56	0.000286	0.47	20.14	64.65	0.14
Reach 1	8671	Max WS	4.16	180.73	182.53		182.53	0.000027	0.17	72.55	114.65	0.04
Reach 1	8591	Max WS	4.16	180.57	182.53		182.53	0.000008	0.1	125.29	169.36	0.02
Reach 1	8323	Max WS	109.22	178.77	182.46		182.48	0.000372	0.91	377.04	443.13	0.19
Reach 1	7976	Max WS	109.2	178.66	182.34		182.35	0.000332	0.91	377.9	399.01	0.18
Reach 1	7762	Max WS	109.19	178.62	182.26		182.28	0.000355	0.93	331.07	296.72	0.19
Reach 1	7619	Max WS	109.18	178.59	182.22		182.23	0.00026	0.83	403.53	373.1	0.16
Reach 1	7217	Max WS	109.17	178.46	182.11		182.13	0.00025	0.86	353.71	321.64	0.16
Reach 1	7061	Max WS	109.17	178.42	182.07		182.09	0.000341	1	333.22	328.84	0.19
Reach 1	6824	Max WS	109.16	178.25	181.99		182.02	0.000265	0.91	311.28	289.65	0.17
Reach 1	6733	Max WS	109.16	178.16	181.97		181.99	0.000223	0.85	359.96	328.48	0.16
Reach 1	6684	Max WS	109.16	178.11	181.96		181.98	0.000267	0.92	365.74	339.99	0.16
Reach 1	6646	Max WS	109.15	178.06	181.88	180.17	182.03	0.000937	1.77	79.23	328.97	0.32
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	109.12	178.05	181.62		181.85	0.001753	2.2	63.78	414.16	0.42
Reach 1	6539	Max WS	109.13	178.07	181.66		181.67	0.000275	0.82	344.38	329.55	0.16
Reach 1	6496	Max WS	109.12	178.07	181.65		181.66	0.000216	0.72	318.48	279.79	0.14
Reach 1	6400	Max WS	109.12	178.09	181.63		181.64	0.000193	0.78	312.96	226.34	0.14
Reach 1	6304	Max WS	109.11	178.11	181.62		181.62	0.000137	0.63	356.26	266.74	0.12
Reach 1	6214	Max WS	109.12	178.12	181.6		181.61	0.000173	0.64	375.17	309.57	0.13
Reach 1	5812	Max WS	109.11	178.08	181.49		181.51	0.000347	0.92	339.08	315.22	0.19
Reach 1	5636	Max WS	109.1	177.99	181.46		181.46	0.000132	0.62	465.96	310.86	0.12
Reach 1	5444	Max WS	109.09	177.93	181.41		181.43	0.000253	0.86	327.08	223.41	0.16
Reach 1	5148	Max WS	109.09	177.83	181.3		181.33	0.000487	1.11	238.18	193.9	0.23
Reach 1	4980	Max WS	109.09	177.77	181.23		181.25	0.000346	0.98	292.96	216.48	0.19
Reach 1	4759	Max WS	109.08	177.67	181.15		181.17	0.000379	1.01	293.2	224.73	0.2
Reach 1	4647	Max WS	109.08	177.65	181.11		181.13	0.000395	1.01	287.41	224.31	0.2
Reach 1	4587	Max WS	109.08	177.62	181.09		181.11	0.000308	0.9	307.66	212.04	0.18
Reach 1	4542	Max WS	109.08	177.63	180.94	180.29	181.18	0.002244	2.35	67.45	204.35	0.48
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	109.05	177.56	180.77		180.98	0.002026	2.19	65.76	194.03	0.46
Reach 1	4441	Max WS	109.05	177.54	180.8		180.81	0.000202	0.67	319.37	249.24	0.14
Reach 1	4399	Max WS	109.05	177.53	180.79		180.8	0.000156	0.58	363.53	278.63	0.13
Reach 1	4291	Max WS	109.06	177.47	180.78		180.78	0.000106	0.52	451.35	366.05	0.11
Reach 1	3909	Max WS	109.05	177.26	180.73		180.73	0.000142	0.63	407.27	365.52	0.12
Reach 1	3757	Max WS	109.05	177.17	180.69		180.7	0.000223	0.79	282.95	222.75	0.15
Reach 1	3644	Max WS	109.05	177.14	180.68		180.69	0.000174	0.7	310.82	216.11	0.14
Reach 1	3510	Max WS	109.05	177.09	180.64		180.67	0.000433	1.11	229.58	155.93	0.21
Reach 1	3356	Max WS	109.04	177.02	180.55		180.59	0.000586	1.24	205.01	153.49	0.25
Reach 1	3246	Max WS	109.04	176.97	180.51		180.53	0.000324	0.96	286.41	200.62	0.19
Reach 1	3047	Max WS	109.04	176.86	180.43		180.44	0.000547	0.54	292.35	211.73	0.1
Reach 1	2950	Max WS	109.04	176.81	180.38		180.41	0.000304	0.96	240.9	169.36	0.18
Reach 1	2757	Max WS	109.04	176.72	180.32		180.35	0.000322	0.99	249.06	179.51	0.19
Reach 1	2652	Max WS	109.04	176.66	180.28		180.31	0.000376	1.04	198.04	129	0.2
Reach 1	2516	Max WS	109.03	176.61	180.21		180.25	0.000554	1.25	181.97	135.73	0.24
Reach 1	2363	Max WS	109.03	176.53	180.13		180.17	0.00059	1.22	218.13	185.75	0.24
Reach 1	2172	Max WS	109.03	176.44	180.01		180.05	0.000595	1.24	217.63	188.88	0.24
Reach 1	2044	Max WS	109.03	176.36	179.96		179.98	0.000359	0.99	302.11	266.99	0.19
Reach 1	1898	Max WS	109.03	176.29	179.88		179.92	0.000674	1.26	227.72	227.2	0.26
Reach 1	1813	Max WS	109.03	176.25	179.83		179.86	0.000546	1.14	262.92	236.36	0.23
Reach 1	1642	Max WS	109.03	176.17	179.73		179.77	0.00056	1.26	227.01	217.81	0.24
Reach 1	1552	Max WS	109.03	176.13	179.7		179.72	0.000311	0.93	283.65	233.24	0.18
Reach 1	1510	Max WS	109.03	176.11	179.69		179.71	0.000259	0.92	324.13	246.02	0.17
Reach 1	1411	Max WS	109.03	176.06	179.47	178.25	179.76	0.001851	2.41	49.76	218.69	0.44
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	109.03	176.04	179.51		179.71	0.001247	2	57.84	300.85	0.37
Reach 1	1329	Max WS	109.03	176.02	179.22		179.62	0.004585	2.83	47.88	292.23	0.64
Reach 1	1282	Max WS	109.03	175.99	179.25		179.28	0.000557	1.16	269.73	328.93	0.23
Reach 1	1207	Max WS	109.03	175.96	179.2		179.23	0.000642	1.21	277.93	372.26	0.25
Reach 1	1099	Max WS	109.03	175.91	179.14		179.17	0.0005	1.09	303.96	372.75	0.22
Reach 1	1005	Max WS	109.03	175.86	179.1		179.13	0.000448	1.12	304	342.65	0.21
Reach 1	890	Max WS	109.03	175.81	179.04	178.39	179.07	0.000498	1.1	288.57	306.27	0.22

25-Year Flood with Dam Breach												
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	4.2	188.13	200.45		200.45	0	0	1629.45	206.51	0
Reach 1	11048	Max WS	4.2	190.96	200.45	191.18	200.45	0	0	1382.12	351.5	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	316.32	185.59	190.45		190.82	0.004945	2.69	117.54	35.19	0.47
Reach 1	10747	Max WS	315.03	185.06	189.98		190.24	0.002529	3.57	198.89	65.64	0.52
Reach 1	10682	Max WS	314.86	184.94	189.87		190.03	0.002681	3.2	218.26	71.59	0.48
Reach 1	10591	Max WS	313.67	184.75	189.55		189.81	0.003123	3.74	200.05	75.82	0.57
Reach 1	10453	Max WS	312.96	184.46	189.2		189.43	0.002159	3.28	215.84	73.95	0.5
Reach 1	10316	Max WS	311.89	184.17	188.88		189.07	0.002813	3.29	218.5	82.6	0.5
Reach 1	10221	Max WS	311.01	183.98	188.62		188.81	0.002612	3.31	219.76	80.98	0.51
Reach 1	10093	Max WS	310.73	183.71	188.33		188.5	0.002167	3.05	228.78	79.58	0.47
Reach 1	9951	Max WS	310.29	183.42	187.97		188.18	0.002526	3.3	212.18	74.61	0.51
Reach 1	9814	Max WS	309.37	183.13	187.69		187.82	0.002289	2.95	237.31	83.81	0.46
Reach 1	9599	Max WS	309.13	182.67	187.22		187.37	0.002021	2.87	239.94	84.12	0.45
Reach 1	9362	Max WS	291.49	182.17	186.21		186.51	0.004919	4.08	168.61	75.79	0.67
Reach 1	9239	Max WS	306.01	181.92	185.93		185.99	0.001336	2.06	355.28	150.27	0.35
Reach 1	9104	Max WS	295.34	181.63	185.68		185.78	0.001913	2.57	267.39	104.04	0.42
Reach 1	9028	Max WS	305.44	181.48	185.41		185.56	0.003849	3.3	212.94	98.09	0.56
Reach 1	8891	Max WS	297.18	181.18	185.04		185.14	0.00212	2.65	288.51	137.8	0.45
Reach 1	8813	Max WS	299.13	181.02	184.52		184.86	0.00656	4.35	155.12	72.19	0.79
Reach 1	8671	Max WS	297.21	180.73	183.91		184.03	0.003659	3.01	238.11	125.36	0.57
Reach 1	8591	Max WS	279.25	180.57	183.71		183.76	0.001838	2.15	332.67	184.55	0.4
Reach 1	8323	Max WS	305.4	178.77	183.47		183.49	0.000293	1.03	834.88	461.5	0.18
Reach 1	7976	Max WS	279.68	178.66	183.38		183.4	0.000242	0.98	797.42	403.79	0.17
Reach 1	7762	Max WS	269.94	178.62	183.32		183.34	0.000293	1.07	649.43	303.74	0.18
Reach 1	7619	Max WS	266.16	178.59	183.29		183.3	0.000197	0.9	808.05	380.71	0.15
Reach 1	7217	Max WS	261.94	178.46	183.21		183.23	0.000207	0.98	717.23	342.77	0.16
Reach 1	7061	Max WS	261.44	178.42	183.17		183.19	0.000238	1.03	700.72	334.21	0.16
Reach 1	6824	Max WS	260.97	178.25	183.12		183.14	0.000226	1.04	654.78	323.43	0.17
Reach 1	6733	Max WS	260.91	178.16	183.1		183.12	0.000183	0.95	742.04	347.07	0.15
Reach 1	6684	Max WS	260.92	178.11	183.09		183.11	0.000196	0.96	752.76	344.22	0.15
Reach 1	6646	Max WS	260.91	178.06	182.93	181.62	183.29	0.001682	2.87	130.76	375.01	0.45
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	209.31	178.05	182.39		182.78	0.002359	2.99	101.49	427.89	0.5
Reach 1	6539	Max WS	255.03	178.07	182.48		182.5	0.000284	1	618.98	341.54	0.17
Reach 1	6496	Max WS	248.54	178.07	182.47		182.48	0.000221	0.87	550.64	288.25	0.15
Reach 1	6400	Max WS	251.58	178.09	182.44		182.47	0.000287	1.12	498.95	230.61	0.18
Reach 1	6304	Max WS	244.14	178.11	182.42		182.44	0.000169	0.83	573.95	270.95	0.14
Reach 1	6214	Max WS	242.67	178.12	182.41		182.42	0.000184	0.81	631.24	322.44	0.14
Reach 1	5812	Max WS	236.07	178.08	182.32		182.34	0.000307	1.05	603.45	326.85	0.19
Reach 1	5636	Max WS	212.88	177.99	182.29		182.29	0.000131	0.73	725.69	315.46	0.12
Reach 1	5444	Max WS	211.74	177.93	182.24		182.26	0.000255	1.02	515.91	235.17	0.17
Reach 1	5148	Max WS	208.47	177.83	182.13		182.17	0.0004	1.23	401.91	196.5	0.22
Reach 1	4980	Max WS	208.44	177.77	182.08		182.1	0.000298	1.09	478.11	218.89	0.18
Reach 1	4759	Max WS	208.34	177.67	182.02		182.04	0.000301	1.09	489.61	229.73	0.19
Reach 1	4647	Max WS	208.38	177.65	181.98		182	0.000305	1.08	485.14	227.58	0.19
Reach 1	4587	Max WS	208.39	177.62	181.97		181.99	0.000267	1.02	495.76	216.46	0.18
Reach 1	4542	Max WS	208.36	177.63	181.76	180.97	182.14	0.002615	3.08	102.62	233.52	0.55
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	186.06	177.56	181.38		181.69	0.002303	2.73	92.34	201	0.51
Reach 1	4441	Max WS	186.09	177.54	181.44		181.45	0.000166	0.72	481.18	253.54	0.14
Reach 1	4399	Max WS	186.15	177.53	181.44		181.44	0.000127	0.63	544.25	281.86	0.12
Reach 1	4291	Max WS	186.04	177.47	181.43		181.43	0.000082	0.53	689.95	369.63	0.1
Reach 1	3909	Max WS	185.18	177.26	181.39		181.39	0.000097	0.6	650.78	369.3	0.11
Reach 1	3757	Max WS	185.01	177.17	181.36		181.37	0.00018	0.83	433.54	227.44	0.14
Reach 1	3644	Max WS	185.36	177.14	181.35		181.36	0.000158	0.77	457	220.02	0.14
Reach 1	3510	Max WS	176.71	177.09	181.31		181.34	0.000388	1.22	335.72	160.17	0.21
Reach 1	3356	Max WS	174.99	177.02	181.24		181.28	0.000463	1.29	311.84	158.02	0.23
Reach 1	3246	Max WS	175.08	176.97	181.21		181.23	0.000261	1	427.5	204.4	0.17
Reach 1	3047	Max WS	172.29	176.86	181.15		181.16	0.000354	0.51	446.49	215.76	0.09
Reach 1	2950	Max WS	171.52	176.81	181.11		181.14	0.000241	1	375.39	192.57	0.17
Reach 1	2757	Max WS	170.36	176.72	181.07		181.09	0.000243	1.01	387.5	192.92	0.17
Reach 1	2652	Max WS	169.8	176.66	181.03		181.07	0.000294	1.08	296.81	132.45	0.19
Reach 1	2516	Max WS	169.27	176.61	180.99		181.02	0.000368	1.2	289.57	140.59	0.2
Reach 1	2363	Max WS	168.59	176.53	180.94		180.97	0.000325	1.09	374.95	200.79	0.19
Reach 1	2172	Max WS	168.1	176.44	180.88		180.91	0.00031	1.08	409.94	250.48	0.18
Reach 1	2044	Max WS	168.01	176.36	180.86		180.87	0.000166	0.81	569.36	337.03	0.14
Reach 1	1898	Max WS	167.89	176.29	180.83		180.84	0.000243	0.94	459.83	270.85	0.16
Reach 1	1813	Max WS	167.84	176.25	180.81		180.82	0.000203	0.86	498.19	245.75	0.14
Reach 1	1642	Max WS	167.76	176.17	180.77		180.79	0.000206	0.94	461.3	254.33	0.15
Reach 1	1552	Max WS	167.73	176.13	180.76		180.77	0.00013	0.75	555.28	349.75	0.12
Reach 1	1510	Max WS	167.72	176.11	180.76		180.77	0.000101	0.7	667.02	424.26	0.11
Reach 1	1411	Max WS	167.7	176.06	180.74	178.89	180.76	0.000166	0.91	603.41	394.74	0.14
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	167.7	176.04	179.87		180.25	0.001989	2.73	66.36	409.74	0.47
Reach 1	1329	Max WS	167.35	176.02	179.54	179.42	180.12	0.005955	3.56	64.08	319.71	0.75
Reach 1	1282	Max WS	167.44	175.99	179.58		179.62	0.000579	1.29	371.09	398.8	0.24
Reach 1	1207	Max WS	167.39	175.96	179.54		179.57	0.000627	1.3	397.2	419.8	0.25
Reach 1	1099	Max WS	167.35	175.91	179.49		179.51	0.000412	1.08	467.93	381.52	0.2
Reach 1	1005	Max WS	167.35	175.86	179.45		179.48	0.000439	1.2	424.35	343.95	0.22
Reach 1	890	Max WS	167.35	175.81	179.39	178.57	179.42	0.000499	1.19	396.17	308.5	0.22

100-Year Flood												
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	5.65	188.13	200.52		200.52	0	0	1643.05	206.78	0
Reach 1	11048	Max WS	5.65	190.96	200.52	191.22	200.52	0	0	1397.12	352.01	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	5.65	185.59	186.6		186.6	0.000228	0.37	15.25	18.67	0.13
Reach 1	10747	Max WS	5.65	185.06	186.52		186.54	0.000575	0.71	7.98	6.47	0.2
Reach 1	10682	Max WS	5.65	184.94	186.25		186.38	0.005054	1.63	4.03	6.4	0.51
Reach 1	10591	Max WS	5.65	184.75	185.92		186	0.003068	1.31	4.32	4.77	0.44
Reach 1	10453	Max WS	5.65	184.46	185.68		185.72	0.00108	0.86	6.6	6.67	0.27
Reach 1	10316	Max WS	5.65	184.17	185.39		185.47	0.002556	1.24	4.57	4.65	0.4
Reach 1	10221	Max WS	5.65	183.98	185.18		185.25	0.002103	1.14	4.97	5.07	0.37
Reach 1	10093	Max WS	5.65	183.71	184.96		185.01	0.001598	0.99	5.7	6.08	0.33
Reach 1	9951	Max WS	5.65	183.42	184.77		184.81	0.001233	0.91	6.19	6.03	0.29
Reach 1	9814	Max WS	5.65	183.13	184.43		184.52	0.003052	1.32	4.46	6.15	0.43
Reach 1	9599	Max WS	5.65	182.67	183.93		183.99	0.001852	1.06	5.33	5.6	0.35
Reach 1	9362	Max WS	5.65	182.17	183.51		183.57	0.001663	1.06	5.46	7	0.32
Reach 1	9239	Max WS	5.65	181.92	183.27		183.33	0.002227	1.15	8.1	49.57	0.37
Reach 1	9104	Max WS	5.64	181.63	183.08		183.11	0.000872	0.78	19.24	87.27	0.23
Reach 1	9028	Max WS	5.64	181.48	183.01		183.03	0.001095	0.83	18.67	71.27	0.25
Reach 1	8891	Max WS	5.59	181.18	182.92		182.93	0.000269	0.5	30.83	90.69	0.14
Reach 1	8813	Max WS	5.55	181.02	182.91		182.91	0.00009	0.31	43.55	66.24	0.08
Reach 1	8671	Max WS	5.49	180.73	182.9		182.9	0.000012	0.13	115.65	117.73	0.03
Reach 1	8591	Max WS	5.5	180.57	182.9		182.9	0.000004	0.08	188.73	172.48	0.02
Reach 1	8323	Max WS	158.45	178.77	182.85		182.86	0.000271	0.86	550.45	449.13	0.17
Reach 1	7976	Max WS	158.43	178.66	182.76		182.77	0.000244	0.87	546.39	400.93	0.16
Reach 1	7762	Max WS	158.41	178.62	182.7		182.71	0.000284	0.93	461.57	299.82	0.17
Reach 1	7619	Max WS	158.4	178.59	182.67		182.68	0.0002	0.81	571.31	377.24	0.15
Reach 1	7217	Max WS	158.38	178.46	182.58		182.6	0.000203	0.86	506.71	330.22	0.15
Reach 1	7061	Max WS	158.37	178.42	182.55		182.56	0.000247	0.94	492.1	331.38	0.16
Reach 1	6824	Max WS	158.36	178.25	182.49		182.51	0.000214	0.91	459.22	302.29	0.16
Reach 1	6733	Max WS	158.36	178.16	182.48		182.49	0.000175	0.83	527.66	337.91	0.14
Reach 1	6684	Max WS	158.36	178.11	182.47		182.48	0.000195	0.86	538.17	341.99	0.14
Reach 1	6646	Max WS	158.36	178.06	182.37	180.69	182.57	0.001109	2.12	103.09	361.96	0.35
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	158.23	178.05	182.03		182.35	0.002083	2.63	84.14	421.56	0.47
Reach 1	6539	Max WS	158.24	178.07	182.09		182.1	0.000219	0.81	487.59	335.44	0.15
Reach 1	6496	Max WS	158.23	178.07	182.08		182.09	0.000176	0.72	440.58	284.89	0.13
Reach 1	6400	Max WS	158.23	178.09	182.06		182.08	0.000196	0.86	411.6	228.95	0.15
Reach 1	6304	Max WS	158.22	178.11	182.05		182.06	0.000127	0.67	472.53	269.13	0.12
Reach 1	6214	Max WS	158.21	178.12	182.04		182.05	0.000147	0.66	512.11	316.98	0.12
Reach 1	5812	Max WS	158.2	178.08	181.95		181.97	0.000261	0.89	485.56	321.22	0.17
Reach 1	5636	Max WS	158.18	177.99	181.92		181.93	0.000122	0.66	611.73	313.38	0.12
Reach 1	5444	Max WS	158.18	177.93	181.88		181.9	0.000237	0.92	432.71	227.62	0.16
Reach 1	5148	Max WS	158.16	177.83	181.78		181.81	0.000401	1.14	332.63	195.4	0.21
Reach 1	4980	Max WS	158.14	177.77	181.73		181.75	0.000291	1	400.88	217.94	0.18
Reach 1	4759	Max WS	158.12	177.67	181.66		181.68	0.000299	1.01	408.98	227.37	0.18
Reach 1	4647	Max WS	158.12	177.65	181.63		181.65	0.000303	1.01	405.04	226.29	0.18
Reach 1	4587	Max WS	158.12	177.62	181.61		181.63	0.000255	0.93	419.8	214.68	0.17
Reach 1	4542	Max WS	158.06	177.63	181.45	180.66	181.74	0.00222	2.65	89.31	221.14	0.49
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	157.93	177.56	181.26		181.52	0.001954	2.45	87.21	197.58	0.47
Reach 1	4441	Max WS	157.98	177.54	181.31		181.32	0.00015	0.67	447.84	252.91	0.13
Reach 1	4399	Max WS	157.98	177.53	181.31		181.31	0.000114	0.58	507.28	280.99	0.11
Reach 1	4291	Max WS	157.95	177.47	181.3		181.3	0.000075	0.49	641.77	368.99	0.09
Reach 1	3909	Max WS	157.96	177.26	181.26		181.27	0.000089	0.56	604.21	368.67	0.1
Reach 1	3757	Max WS	157.95	177.17	181.24		181.25	0.000161	0.76	405.51	226.59	0.14
Reach 1	3644	Max WS	157.94	177.14	181.22		181.24	0.000138	0.7	430.29	219.18	0.13
Reach 1	3510	Max WS	157.93	177.09	181.19		181.22	0.000366	1.15	316.87	159.49	0.2
Reach 1	3356	Max WS	157.93	177.02	181.12		181.16	0.000448	1.24	293.78	157.26	0.22
Reach 1	3246	Max WS	157.92	176.97	181.1		181.11	0.00025	0.96	404.23	203.83	0.17
Reach 1	3047	Max WS	157.92	176.86	181.04		181.05	0.000356	0.5	422.13	215.13	0.09
Reach 1	2950	Max WS	157.91	176.81	181		181.03	0.000239	0.97	353.9	188.36	0.17
Reach 1	2757	Max WS	157.91	176.72	180.95		180.98	0.000245	0.99	365.79	189.5	0.17
Reach 1	2652	Max WS	157.91	176.66	180.92		180.95	0.000294	1.06	281.9	131.79	0.18
Reach 1	2516	Max WS	157.89	176.61	180.87		180.91	0.000377	1.19	273.58	139.98	0.2
Reach 1	2363	Max WS	157.89	176.53	180.83		180.85	0.00034	1.09	351.7	198.45	0.19
Reach 1	2172	Max WS	157.89	176.44	180.77		180.79	0.00031	1.06	380.82	243.43	0.18
Reach 1	2044	Max WS	157.89	176.36	180.74		180.75	0.000177	0.82	529.77	327.25	0.14
Reach 1	1898	Max WS	157.89	176.29	180.7		180.72	0.000261	0.95	427.43	265.34	0.17
Reach 1	1813	Max WS	157.88	176.25	180.69		180.7	0.000215	0.87	468.22	243.98	0.15
Reach 1	1642	Max WS	157.88	176.17	180.65		180.67	0.000218	0.95	430.56	230.9	0.16
Reach 1	1552	Max WS	157.88	176.13	180.63		180.65	0.000137	0.75	517.59	314.08	0.13
Reach 1	1510	Max WS	157.88	176.11	180.63		180.64	0.000109	0.71	616.36	414.53	0.11
Reach 1	1411	Max WS	157.88	176.06	180.61	178.77	180.63	0.000212	1.01	489.75	389.73	0.16
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	157.88	176.04	179.82		180.16	0.00186	2.61	65.14	395.62	0.46
Reach 1	1329	Max WS	157.88	176.02	179.49	179.35	180.05	0.005765	3.45	61.67	309.75	0.74
Reach 1	1282	Max WS	157.88	175.99	179.53		179.57	0.000579	1.27	355.47	381.25	0.24
Reach 1	1207	Max WS	157.88	175.96	179.49		179.52	0.000634	1.29	378.24	411.16	0.25
Reach 1	1099	Max WS	157.88	175.91	179.44		179.46	0.000416	1.07	447.93	381.31	0.2
Reach 1	1005	Max WS	157.88	175.86	179.39		179.42	0.00044	1.19	406.32	343.81	0.21
Reach 1	890	Max WS	157.88	175.81	179.34	178.56	179.37	0.000499	1.18	380.01	308.22	0.22

100-Year Flood with Dam Breach												
Reach	**River Sta**	**Profile**	**Q Total**	**Min Ch El**	**W.S. Elev**	**Crit W.S.**	**E.G. Elev**	**E.G. Slope**	**Vel Chnl**	**Flow Area**	**Top Width**	**Froude # Chl**
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	5.65	188.13	200.52		200.52	0	0	1642.99	206.78	0
Reach 1	11048	Max WS	5.64	190.96	200.52	191.22	200.52	0	0	1397.05	352.01	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	322.76	185.59	190.49		190.87	0.005006	2.71	119.07	35.41	0.47
Reach 1	10747	Max WS	322.13	185.06	190.01		190.29	0.002548	3.6	201.45	65.76	0.53
Reach 1	10682	Max WS	321.18	184.94	189.91		190.07	0.002686	3.23	221.08	71.79	0.48
Reach 1	10591	Max WS	320.76	184.75	189.59		189.85	0.003134	3.77	203.01	76.09	0.57
Reach 1	10453	Max WS	319.42	184.46	189.24		189.47	0.002165	3.31	218.64	74.09	0.5
Reach 1	10316	Max WS	318.74	184.17	188.92		189.11	0.002814	3.31	221.65	82.73	0.5
Reach 1	10221	Max WS	317.8	183.98	188.66		188.86	0.002615	3.33	222.87	81.14	0.51
Reach 1	10093	Max WS	313.04	183.71	188.37		188.54	0.002106	3.03	232.11	79.71	0.47
Reach 1	9951	Max WS	314.03	183.42	188.01		188.22	0.002486	3.29	215.05	74.79	0.51
Reach 1	9814	Max WS	314.36	183.13	187.73		187.87	0.002248	2.95	241.17	83.94	0.46
Reach 1	9599	Max WS	315.06	182.67	187.26		187.4	0.002016	2.88	243.15	84.25	0.45
Reach 1	9362	Max WS	299.82	182.17	186.28		186.58	0.004754	4.06	173.92	76.19	0.66
Reach 1	9239	Max WS	313.11	181.92	185.97		186.03	0.001334	2.08	360.97	150.77	0.35
Reach 1	9104	Max WS	302.32	181.63	185.75		185.84	0.001861	2.56	274	104.45	0.42
Reach 1	9028	Max WS	299.86	181.48	185.45		185.59	0.003508	3.17	217.05	98.54	0.53
Reach 1	8891	Max WS	303.12	181.18	185.11		185.2	0.002003	2.61	297.68	138.27	0.44
Reach 1	8813	Max WS	305.31	181.02	184.6		184.93	0.006127	4.27	160.84	72.53	0.76
Reach 1	8671	Max WS	301.91	180.73	184.08		184.18	0.002912	2.79	258.83	126.52	0.51
Reach 1	8591	Max WS	253.28	180.57	183.96		183.99	0.001014	1.69	378.7	187.78	0.3
Reach 1	8323	Max WS	343.33	178.77	183.82		183.83	0.000215	0.95	994.83	463.18	0.16
Reach 1	7976	Max WS	326.91	178.66	183.75		183.76	0.000195	0.94	945.65	405.46	0.15
Reach 1	7762	Max WS	320.13	178.62	183.7		183.72	0.00025	1.06	763.98	305.68	0.17
Reach 1	7619	Max WS	317.83	178.59	183.67		183.68	0.000168	0.89	953.45	383.05	0.14
Reach 1	7217	Max WS	314.52	178.46	183.6		183.62	0.000182	0.98	853.29	351.09	0.15
Reach 1	7061	Max WS	313.9	178.42	183.57		183.59	0.000203	1.01	833.52	335.64	0.15
Reach 1	6824	Max WS	313.66	178.25	183.52		183.54	0.000197	1.04	786.46	327.27	0.16
Reach 1	6733	Max WS	313.64	178.16	183.51		183.52	0.000158	0.94	883.85	349.95	0.14
Reach 1	6684	Max WS	313.6	178.11	183.5		183.52	0.000169	0.95	893.47	345.89	0.14
Reach 1	6646	Max WS	302.9	178.06	183.34	181.95	183.71	0.001567	2.95	150.61	383.86	0.44
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	267.5	178.05	182.83		183.28	0.002372	3.25	123.21	435.19	0.52
Reach 1	6539	Max WS	256.3	178.07	182.89		182.9	0.000155	0.79	759.46	346.81	0.13
Reach 1	6496	Max WS	267.86	178.07	182.87		182.88	0.000143	0.76	666.37	292	0.13
Reach 1	6400	Max WS	250.72	178.09	182.87		182.89	0.000167	0.92	599.17	232.13	0.14
Reach 1	6304	Max WS	266.04	178.11	182.83		182.84	0.000117	0.74	684.47	272.65	0.12
Reach 1	6214	Max WS	245.14	178.12	182.84		182.85	0.000102	0.65	771.01	327.69	0.11
Reach 1	5812	Max WS	254.27	178.08	182.76		182.77	0.000186	0.89	748.56	332.83	0.15
Reach 1	5636	Max WS	231.07	177.99	182.75		182.76	0.000087	0.65	873.4	318.07	0.1
Reach 1	5444	Max WS	247.27	177.93	182.7		182.71	0.000199	0.98	624.21	241.92	0.15
Reach 1	5148	Max WS	221.17	177.83	182.66		182.68	0.000227	1.02	505	198.18	0.17
Reach 1	4980	Max WS	231.87	177.77	182.58		182.6	0.000196	0.96	587.12	220.22	0.15
Reach 1	4759	Max WS	212.84	177.67	182.56		182.58	0.000156	0.86	616.29	234.24	0.14
Reach 1	4647	Max WS	224.63	177.65	182.51		182.52	0.000179	0.91	605.17	229.61	0.15
Reach 1	4587	Max WS	205.92	177.62	182.53		182.55	0.000131	0.79	619.74	221.91	0.13
Reach 1	4542	Max WS	234.67	177.63	182.47	181.11	182.48	0.000208	0.99	580.96	258.95	0.16
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	267.2	177.56	181.89		182.3	0.002531	3.18	114.45	219.97	0.55
Reach 1	4441	Max WS	245.29	177.54	181.89		181.9	0.000148	0.75	595.29	256.07	0.13
Reach 1	4399	Max WS	244.04	177.53	181.87		181.88	0.000114	0.65	667.24	284.21	0.12
Reach 1	4291	Max WS	244.19	177.47	181.87		181.88	0.000072	0.54	854.55	371.72	0.09
Reach 1	3909	Max WS	246.16	177.26	181.83		181.84	0.000084	0.61	815.25	371.45	0.1
Reach 1	3757	Max WS	245.43	177.17	181.82		181.84	0.000162	0.86	539.67	230.42	0.14
Reach 1	3644	Max WS	241.8	177.14	181.79		181.81	0.000148	0.81	556.38	222.63	0.13
Reach 1	3510	Max WS	244.46	177.09	181.78		181.82	0.000407	1.36	412.05	163.03	0.22
Reach 1	3356	Max WS	242.14	177.02	181.68		181.73	0.00049	1.44	383.59	163.86	0.24
Reach 1	3246	Max WS	243.9	176.97	181.67		181.7	0.000277	1.12	523.15	207.45	0.18
Reach 1	3047	Max WS	241.48	176.86	181.6		181.61	0.000372	0.56	542.97	224.63	0.09
Reach 1	2950	Max WS	243.46	176.81	181.57		181.61	0.000275	1.16	468.27	212.14	0.19
Reach 1	2757	Max WS	236.29	176.72	181.5		181.54	0.000269	1.14	473.56	201.7	0.18
Reach 1	2652	Max WS	242.81	176.66	181.49		181.54	0.000359	1.3	359.35	141.88	0.21
Reach 1	2516	Max WS	237.08	176.61	181.41		181.46	0.000419	1.38	349.67	143.32	0.22
Reach 1	2363	Max WS	241.98	176.53	181.37		181.41	0.00037	1.26	464.28	213.06	0.21
Reach 1	2172	Max WS	237.2	176.44	181.29		181.32	0.00034	1.22	516.68	272.35	0.2
Reach 1	2044	Max WS	241.01	176.36	181.27		181.29	0.000194	0.95	714.06	369.21	0.15
Reach 1	1898	Max WS	236.94	176.29	181.23		181.25	0.000271	1.07	577.39	320.5	0.17
Reach 1	1813	Max WS	239.78	176.25	181.22		181.23	0.000235	0.99	610.8	317.42	0.16
Reach 1	1642	Max WS	236.6	176.17	181.16		181.19	0.000248	1.1	571.02	325.96	0.17
Reach 1	1552	Max WS	238.55	176.13	181.15		181.17	0.000157	0.88	703.65	428.07	0.14
Reach 1	1510	Max WS	236.47	176.11	181.14		181.15	0.000114	0.79	833.65	446.33	0.12
Reach 1	1411	Max WS	236.91	176.06	181.13	179.53	181.15	0.000176	1	762.81	425.1	0.15
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	236.91	176.04	180.2		180.81	0.002877	3.49	74.06	437.44	0.58
Reach 1	1329	Max WS	234.71	176.02	179.85	179.8	180.62	0.006864	4.16	80.58	422.32	0.82
Reach 1	1282	Max WS	234.97	175.99	179.91		179.94	0.000545	1.34	547.56	430.59	0.24
Reach 1	1207	Max WS	234.87	175.96	179.88		179.9	0.000499	1.25	582.42	437.23	0.23
Reach 1	1099	Max WS	234.81	175.91	179.83		179.85	0.000395	1.14	597.34	382.77	0.2
Reach 1	1005	Max WS	234.79	175.86	179.79		179.82	0.000433	1.28	541.29	344.85	0.22
Reach 1	890	Max WS	234.77	175.81	179.73	178.74	179.76	0.000499	1.28	501.15	310.35	0.23

1000-Year Flood												
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	11.82	188.13	200.75		200.75	0	0.01	1691.44	207.69	0
Reach 1	11048	Max WS	11.82	190.96	200.75	191.35	200.75	0	0.01	1450.49	353.2	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	11.82	185.59	187.04		187.05	0.000315	0.49	23.9	20.7	0.15
Reach 1	10747	Max WS	11.82	185.06	186.93		186.97	0.000773	0.99	21.1	49.94	0.24
Reach 1	10682	Max WS	11.81	184.94	186.76		186.87	0.003533	1.75	16.03	55.96	0.45
Reach 1	10591	Max WS	11.81	184.75	186.46		186.58	0.002684	1.58	10.64	30.22	0.43
Reach 1	10453	Max WS	11.8	184.46	186.25		186.31	0.000964	1.07	16.04	55.65	0.28
Reach 1	10316	Max WS	11.8	184.17	185.92		186.05	0.003085	1.63	8.41	23.66	0.44
Reach 1	10221	Max WS	11.8	183.98	185.65		185.78	0.002677	1.57	8.64	24.43	0.42
Reach 1	10093	Max WS	11.8	183.71	185.41		185.49	0.001701	1.26	14.64	52.46	0.35
Reach 1	9951	Max WS	11.79	183.42	185.18		185.26	0.001517	1.24	16.33	63.33	0.33
Reach 1	9814	Max WS	11.79	183.13	184.87		184.97	0.002863	1.57	15.71	62.1	0.43
Reach 1	9599	Max WS	11.79	182.67	184.4		184.46	0.001709	1.22	18.89	63.3	0.34
Reach 1	9362	Max WS	11.74	182.17	184.01		184.08	0.001526	1.26	18.54	52.54	0.32
Reach 1	9239	Max WS	11.53	181.92	183.93		183.93	0.000164	0.42	79.85	122.93	0.11
Reach 1	9104	Max WS	11.34	181.63	183.91		183.91	0.000068	0.31	94.07	92.46	0.07
Reach 1	9028	Max WS	11.26	181.48	183.91		183.91	0.000077	0.32	85.39	77.71	0.07
Reach 1	8891	Max WS	11.07	181.18	183.9		183.9	0.000024	0.22	140.74	122.46	0.05
Reach 1	8813	Max WS	11.08	181.02	183.9		183.9	0.000025	0.23	111.05	70.12	0.05
Reach 1	8671	Max WS	11.01	180.73	183.9		183.9	0.000005	0.11	236.16	125.24	0.02
Reach 1	8591	Max WS	11.02	180.57	183.9		183.9	0.000002	0.08	367.27	187.23	0.01
Reach 1	8323	Max WS	285.24	178.77	183.86		183.87	0.000139	0.77	1016.57	463.41	0.13
Reach 1	7976	Max WS	285.13	178.66	183.82		183.82	0.000136	0.79	972.97	405.86	0.13
Reach 1	7762	Max WS	285.11	178.62	183.78		183.79	0.000179	0.91	788.78	306.04	0.15
Reach 1	7619	Max WS	285.14	178.59	183.76		183.77	0.000121	0.77	987.09	383.46	0.12
Reach 1	7217	Max WS	285.11	178.46	183.71		183.72	0.000132	0.84	890.83	352.31	0.13
Reach 1	7061	Max WS	285.1	178.42	183.69		183.7	0.000146	0.87	872.02	336.18	0.13
Reach 1	6824	Max WS	285.08	178.25	183.65		183.67	0.00014	0.89	828.61	328.11	0.13
Reach 1	6733	Max WS	285.08	178.16	183.64		183.65	0.000112	0.81	930.31	350.89	0.12
Reach 1	6684	Max WS	285.08	178.11	183.64		183.65	0.000119	0.81	940.1	346.33	0.12
Reach 1	6646	Max WS	285.03	178.06	183.51	181.85	183.8	0.001198	2.64	159.09	388.36	0.38
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	284.17	178.05	183.04		183.47	0.002166	3.21	133.6	438.5	0.5
Reach 1	6539	Max WS	284.3	178.07	183.13		183.14	0.000139	0.78	843.08	349.39	0.12
Reach 1	6496	Max WS	284.29	178.07	183.12		183.13	0.000116	0.71	741.23	294.17	0.11
Reach 1	6400	Max WS	284.26	178.09	183.1		183.12	0.000167	0.95	652.25	232.89	0.14
Reach 1	6304	Max WS	284.25	178.11	183.09		183.1	0.000098	0.71	756.33	273.72	0.11
Reach 1	6214	Max WS	284.25	178.12	183.09		183.1	0.000101	0.68	852.1	330.83	0.11
Reach 1	5812	Max WS	284.16	178.08	183.03		183.04	0.000163	0.87	840.76	336.32	0.14
Reach 1	5636	Max WS	284.12	177.99	183.01		183.02	0.000099	0.72	956.14	319.14	0.11
Reach 1	5444	Max WS	284.08	177.93	182.98		183	0.000192	1	692.67	245.23	0.15
Reach 1	5148	Max WS	283.97	177.83	182.9		182.93	0.000283	1.19	553.3	198.97	0.19
Reach 1	4980	Max WS	283.94	177.77	182.86		182.88	0.000215	1.06	650.01	221.19	0.16
Reach 1	4759	Max WS	283.88	177.67	182.82		182.84	0.000212	1.04	676.23	238.39	0.16
Reach 1	4647	Max WS	283.85	177.65	182.79		182.81	0.000208	1.03	671.06	230.79	0.16
Reach 1	4587	Max WS	283.84	177.62	182.78		182.8	0.000191	1	675.22	224.11	0.15
Reach 1	4542	Max WS	283.85	177.63	182.77	181.36	182.79	0.000216	1.06	648.53	265.01	0.16
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	283.48	177.56	182.48		182.49	0.000164	0.9	563.1	231.99	0.14
Reach 1	4441	Max WS	283.5	177.54	182.48		182.49	0.000097	0.68	746.77	260.68	0.11
Reach 1	4399	Max WS	283.49	177.53	182.47		182.48	0.000073	0.59	839.49	287.11	0.1
Reach 1	4291	Max WS	283.5	177.47	182.47		182.47	0.000046	0.48	1077.98	374.72	0.08
Reach 1	3909	Max WS	283.48	177.26	182.45		182.45	0.000051	0.52	1045.49	374.33	0.08
Reach 1	3757	Max WS	283.44	177.17	182.43		182.44	0.000106	0.76	681.17	235.06	0.12
Reach 1	3644	Max WS	283.46	177.14	182.42		182.43	0.000101	0.75	697.25	226.43	0.11
Reach 1	3510	Max WS	283.46	177.09	182.39		182.42	0.000287	1.26	513.16	167.76	0.19
Reach 1	3356	Max WS	283.41	177.02	182.34		182.38	0.000318	1.3	493.45	169.17	0.2
Reach 1	3246	Max WS	283.42	176.97	182.33		182.34	0.000184	1.02	665.43	231.14	0.15
Reach 1	3047	Max WS	283.39	176.86	182.29		182.3	0.000225	0.49	720.83	284.71	0.07
Reach 1	2950	Max WS	283.38	176.81	182.26		182.29	0.000189	1.07	625.97	252.08	0.16
Reach 1	2757	Max WS	283.35	176.72	182.23		182.25	0.000177	1.04	622.45	210.42	0.15
Reach 1	2652	Max WS	283.34	176.66	182.2		182.23	0.000242	1.19	473.55	200.33	0.18
Reach 1	2516	Max WS	283.36	176.61	182.16		182.2	0.000303	1.31	478.72	221.15	0.19
Reach 1	2363	Max WS	283.34	176.53	182.13		182.15	0.000225	1.11	637.7	251.77	0.17
Reach 1	2172	Max WS	283.32	176.44	182.09		182.11	0.000189	1.03	748.79	307.73	0.15
Reach 1	2044	Max WS	283.34	176.36	182.08		182.09	0.000105	0.78	1034.89	421.4	0.11
Reach 1	1898	Max WS	283.32	176.29	182.06		182.07	0.000139	0.87	877.13	384.44	0.13
Reach 1	1813	Max WS	283.32	176.25	182.05		182.06	0.000117	0.8	901.78	373.77	0.12
Reach 1	1642	Max WS	283.32	176.17	182.03		182.04	0.000125	0.88	871.92	363.62	0.12
Reach 1	1552	Max WS	283.32	176.13	182.02		182.03	0.000071	0.67	1109.89	494.03	0.1
Reach 1	1510	Max WS	283.32	176.11	182.02		182.02	0.000054	0.61	1228.97	458.36	0.08
Reach 1	1411	Max WS	283.32	176.06	182.01	179.89	182.02	0.000083	0.77	1143.83	443.9	0.1
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	283.32	176.04	180.48		180.52	0.000434	1.42	628.5	443.5	0.23
Reach 1	1329	Max WS	283.32	176.02	180.05	180.04	180.94	0.00744	4.55	90.7	425.63	0.87
Reach 1	1282	Max WS	283.31	175.99	180.13		180.16	0.000506	1.35	639.65	431.43	0.23
Reach 1	1207	Max WS	283.31	175.96	180.09		180.12	0.000464	1.26	677.13	439.1	0.22
Reach 1	1099	Max WS	283.32	175.91	180.05		180.07	0.000387	1.18	680.84	383.55	0.2
Reach 1	1005	Max WS	283.32	175.86	180		180.03	0.00043	1.33	617.43	355.09	0.22
Reach 1	890	Max WS	283.31	175.81	179.95	178.81	179.98	0.000499	1.34	568.91	311.54	0.23

1000-Year Flood with Dam Breach												
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	11.82	188.13	200.75		200.75	0	0.01	1691.44	207.69	0
Reach 1	11048	Max WS	11.82	190.96	200.75	191.35	200.75	0	0.01	1450.49	353.2	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	331.95	185.59	190.63		190.99	0.004794	2.68	123.86	36.02	0.46
Reach 1	10747	Max WS	342.82	185.06	190.13		190.42	0.00258	3.68	209.37	66.15	0.53
Reach 1	10682	Max WS	342.56	184.94	190.03		190.19	0.002725	3.3	229.73	72.36	0.48
Reach 1	10591	Max WS	330.78	184.75	189.71		189.96	0.002937	3.72	212.41	77.04	0.55
Reach 1	10453	Max WS	335.05	184.46	189.37		189.6	0.002101	3.32	228.2	74.57	0.49
Reach 1	10316	Max WS	333.39	184.17	189.06		189.24	0.002653	3.28	232.95	83.19	0.49
Reach 1	10221	Max WS	335.62	183.98	188.8		188.99	0.002523	3.34	233.96	81.66	0.5
Reach 1	10093	Max WS	334.16	183.71	188.5		188.68	0.002096	3.08	242.8	80.18	0.47
Reach 1	9951	Max WS	334.66	183.42	188.14		188.35	0.002479	3.35	224.73	75.41	0.51
Reach 1	9814	Max WS	329.91	183.13	187.85		187.99	0.002193	2.96	250.84	84.25	0.45
Reach 1	9599	Max WS	329.52	182.67	187.37		187.52	0.001954	2.89	253.01	84.64	0.45
Reach 1	9362	Max WS	332.9	182.17	186.36		186.7	0.005294	4.35	180.11	76.72	0.7
Reach 1	9239	Max WS	331.32	181.92	186.12		186.17	0.001248	2.06	383.26	152.7	0.34
Reach 1	9104	Max WS	330.07	181.63	185.87		185.97	0.001933	2.67	286.99	105.55	0.43
Reach 1	9028	Max WS	329.14	181.48	185.64		185.78	0.00331	3.19	236.13	100.6	0.52
Reach 1	8891	Max WS	237.73	181.18	185.39		185.43	0.00085	1.79	336.97	141.44	0.29
Reach 1	8813	Max WS	236.6	181.02	185.1		185.23	0.001964	2.67	197.73	74.26	0.44
Reach 1	8671	Max WS	234.77	180.73	185.05		185.08	0.00051	1.41	385.52	132.59	0.22
Reach 1	8591	Max WS	232.15	180.57	184.92		184.93	0.000248	1	563.24	197	0.16
Reach 1	8323	Max WS	494.36	178.77	184.86		184.87	0.000127	0.86	1482.81	467.85	0.13
Reach 1	7976	Max WS	488.28	178.66	184.8		184.81	0.000134	0.91	1375.88	410.87	0.13
Reach 1	7762	Max WS	462.42	178.62	184.8		184.81	0.000165	1.02	1102.76	311.56	0.15
Reach 1	7619	Max WS	477.35	178.59	184.75		184.76	0.000122	0.89	1369.19	389.8	0.12
Reach 1	7217	Max WS	431.38	178.46	184.72		184.73	0.000109	0.88	1253.65	365.64	0.12
Reach 1	7061	Max WS	477.12	178.42	184.67		184.69	0.000149	1.01	1207.65	342.18	0.14
Reach 1	6824	Max WS	407.74	178.25	184.67		184.68	0.000103	0.87	1165.5	333.78	0.12
Reach 1	6733	Max WS	477.17	178.16	184.63		184.64	0.00012	0.95	1279.92	357.59	0.13
Reach 1	6684	Max WS	392.21	178.11	184.65		184.66	0.000083	0.77	1301.67	369.68	0.1
Reach 1	6646	Max WS	429.78	178.06	184.5	182.58	184.72	0.00088	2.57	357.05	417.99	0.34
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	418.07	178.05	183.6		184.24	0.002824	3.98	161.02	446.55	0.58
Reach 1	6539	Max WS	495.5	178.07	183.76		183.78	0.000204	1.04	1067.9	356.06	0.15
Reach 1	6496	Max WS	490.5	178.07	183.75		183.77	0.000171	0.95	927.79	299.09	0.14
Reach 1	6400	Max WS	490	178.09	183.73		183.77	0.000269	1.32	799.52	235.05	0.19
Reach 1	6304	Max WS	482.65	178.11	183.71		183.73	0.00015	0.96	926.58	277.56	0.14
Reach 1	6214	Max WS	484.57	178.12	183.7		183.72	0.000152	0.91	1059.45	341.42	0.14
Reach 1	5812	Max WS	479.09	178.08	183.63		183.65	0.00024	1.16	1044.38	343.84	0.17
Reach 1	5636	Max WS	468.87	177.99	183.6		183.62	0.000153	0.98	1145.26	321.39	0.14
Reach 1	5444	Max WS	463.15	177.93	183.55		183.58	0.00029	1.34	835.39	249.72	0.19
Reach 1	5148	Max WS	460.12	177.83	183.45		183.5	0.000428	1.59	662.19	200.52	0.24
Reach 1	4980	Max WS	451.11	177.77	183.39		183.42	0.000325	1.4	766.98	223.58	0.2
Reach 1	4759	Max WS	452.51	177.67	183.32		183.35	0.000326	1.4	797.52	243.51	0.2
Reach 1	4647	Max WS	442.34	177.65	183.29		183.32	0.000308	1.35	785.91	232.44	0.2
Reach 1	4587	Max WS	442.72	177.62	183.27		183.3	0.000293	1.33	785.9	228.83	0.19
Reach 1	4542	Max WS	442.39	177.63	183.26	181.9	183.29	0.000323	1.39	755.77	267.99	0.2
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	413.04	177.56	182.9		182.93	0.000222	1.12	649.94	237.08	0.17
Reach 1	4441	Max WS	413.27	177.54	182.9		182.92	0.000133	0.85	858.7	264.36	0.13
Reach 1	4399	Max WS	413.29	177.53	182.9		182.91	0.000101	0.74	962.25	290.74	0.11
Reach 1	4291	Max WS	413.35	177.47	182.89		182.9	0.000063	0.6	1237.59	377.17	0.09
Reach 1	3909	Max WS	412.63	177.26	182.87		182.87	0.000069	0.65	1202.4	376.63	0.09
Reach 1	3757	Max WS	412.41	177.17	182.84		182.86	0.000149	0.96	777.83	238.3	0.14
Reach 1	3644	Max WS	412.35	177.14	182.83		182.85	0.000145	0.95	789.55	228.81	0.14
Reach 1	3510	Max WS	412.27	177.09	182.78		182.83	0.000424	1.61	578.92	169.98	0.23
Reach 1	3356	Max WS	411.99	177.02	182.71		182.77	0.000472	1.66	556.03	172.49	0.24
Reach 1	3246	Max WS	411.92	176.97	182.68		182.72	0.000276	1.31	749.94	240.44	0.19
Reach 1	3047	Max WS	411.77	176.86	182.63		182.65	0.000331	0.62	821.86	302.17	0.09
Reach 1	2950	Max WS	411.75	176.81	182.58		182.63	0.000303	1.41	710.37	262.82	0.2
Reach 1	2757	Max WS	411.7	176.72	182.53		182.57	0.000283	1.36	688.02	223.6	0.19
Reach 1	2652	Max WS	411.67	176.66	182.48		182.55	0.000391	1.58	537.25	246.74	0.23
Reach 1	2516	Max WS	411.68	176.61	182.42		182.49	0.000514	1.77	542.3	270.75	0.25
Reach 1	2363	Max WS	411.68	176.53	182.37		182.41	0.000381	1.49	699.75	264.34	0.22
Reach 1	2172	Max WS	411.64	176.44	182.31		182.34	0.000315	1.37	815.32	313.03	0.2
Reach 1	2044	Max WS	411.59	176.36	182.28		182.3	0.000174	1.04	1122.08	432.86	0.15
Reach 1	1898	Max WS	411.52	176.29	182.25		182.27	0.000237	1.17	952.38	398.98	0.17
Reach 1	1813	Max WS	411.67	176.25	182.23		182.25	0.000215	1.11	975.04	415.01	0.16
Reach 1	1642	Max WS	411.15	176.17	182.19		182.21	0.00022	1.19	936.04	408.53	0.16
Reach 1	1552	Max WS	368.3	176.13	182.18		182.19	0.000098	0.81	1190	495.96	0.11
Reach 1	1510	Max WS	411.06	176.11	182.18		182.19	0.000096	0.83	1303.03	466.95	0.11
Reach 1	1411	Max WS	330.47	176.06	182.16	180.22	182.17	0.000096	0.84	1212.68	449	0.11
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	411.54	176.04	181		181.03	0.000361	1.41	900.93	448.84	0.21
Reach 1	1329	Max WS	397.89	176.02	180.6		180.64	0.000703	1.57	706.27	429.59	0.27
Reach 1	1282	Max WS	409.56	175.99	180.6		180.63	0.000464	1.41	842.97	433.19	0.23
Reach 1	1207	Max WS	408.11	175.96	180.57		180.59	0.000425	1.32	886.46	443.63	0.22
Reach 1	1099	Max WS	406.16	175.91	180.52		180.55	0.00038	1.27	864.32	385.27	0.21
Reach 1	1005	Max WS	402.58	175.86	180.49		180.52	0.000418	1.42	792.44	370.33	0.22
Reach 1	890	Max WS	407.39	175.81	180.44	179.05	180.48	0.000499	1.47	723.92	314.96	0.23

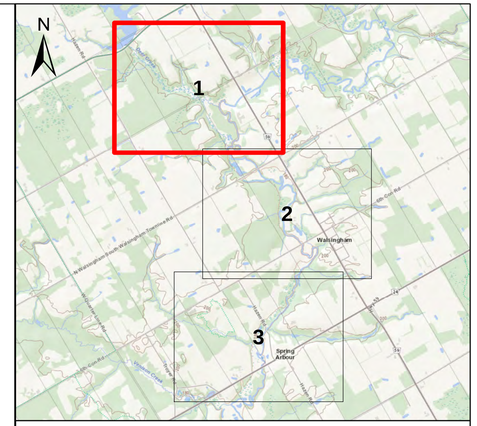
1/3 Between 1000-Year and PMF Flood												
Reach	**River Sta**	**Profile**	**Q Total**	**Min Ch El**	**W.S. Elev**	**Crit W.S.**	**E.G. Elev**	**E.G. Slope**	**Vel Chnl**	**Flow Area**	**Top Width**	**Froude # Chl**
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	27.84	188.13	201.21		201.21	0	0.02	1787.81	209.79	0
Reach 1	11048	Max WS	27.81	190.96	201.21	191.55	201.21	0	0.02	1556.52	355.54	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	27.81	185.59	187.51		187.54	0.000681	0.82	34.02	22.31	0.21
Reach 1	10747	Max WS	27.81	185.06	187.33		187.4	0.00122	1.44	41.95	52.6	0.32
Reach 1	10682	Max WS	27.81	184.94	187.22		187.29	0.002351	1.7	43.67	60.51	0.39
Reach 1	10591	Max WS	27.8	184.75	186.97		187.09	0.002565	1.9	32.8	54.78	0.44
Reach 1	10453	Max WS	27.8	184.46	186.75		186.81	0.001045	1.34	46.44	63.68	0.3
Reach 1	10316	Max WS	27.79	184.17	186.44		186.57	0.002947	1.95	30.96	58.77	0.45
Reach 1	10221	Max WS	27.79	183.98	186.2		186.31	0.002258	1.8	36.16	61.5	0.41
Reach 1	10093	Max WS	27.77	183.71	186.03		186.07	0.000956	1.2	55.7	70.94	0.28
Reach 1	9951	Max WS	27.74	183.42	185.93		185.95	0.000584	1.02	66.1	68.43	0.22
Reach 1	9814	Max WS	27.68	183.13	185.87		185.88	0.000338	0.78	90.65	77.45	0.16
Reach 1	9599	Max WS	27.58	182.67	185.83		185.83	0.00011	0.51	126.49	78.57	0.1
Reach 1	9362	Max WS	27.45	182.17	185.8		185.81	0.000077	0.47	138.32	72.34	0.08
Reach 1	9239	Max WS	27.41	181.92	185.8		185.8	0.000013	0.2	335.34	148.5	0.03
Reach 1	9104	Max WS	27.32	181.63	185.8		185.8	0.000014	0.23	279.45	104.9	0.04
Reach 1	9028	Max WS	27.31	181.48	185.8		185.8	0.000019	0.25	252.28	102.78	0.04
Reach 1	8891	Max WS	27.29	181.18	185.8		185.8	0.000007	0.17	395.42	146.3	0.03
Reach 1	8813	Max WS	27.25	181.02	185.8		185.8	0.000013	0.24	250.17	76.77	0.04
Reach 1	8671	Max WS	27.26	180.73	185.79		185.79	0.000003	0.13	485.3	136.84	0.02
Reach 1	8591	Max WS	27.27	180.57	185.79		185.79	0.000001	0.09	738.95	203.12	0.01
Reach 1	8323	Max WS	679.69	178.77	185.76		185.77	0.000108	0.89	1902.23	470.64	0.12
Reach 1	7976	Max WS	679.73	178.66	185.72		185.73	0.00012	0.97	1754.45	415.94	0.13
Reach 1	7762	Max WS	679.69	178.62	185.68		185.7	0.000176	1.17	1380.72	316.67	0.15
Reach 1	7619	Max WS	679.71	178.59	185.66		185.68	0.000118	0.97	1728.21	393.93	0.13
Reach 1	7217	Max WS	679.69	178.46	185.61		185.63	0.000132	1.07	1584.83	373.97	0.14
Reach 1	7061	Max WS	679.67	178.42	185.59		185.61	0.000146	1.11	1522.96	346.96	0.14
Reach 1	6824	Max WS	679.65	178.25	185.55		185.58	0.000143	1.13	1463.06	338.41	0.14
Reach 1	6733	Max WS	679.65	178.16	185.54		185.56	0.00012	1.05	1609.88	362.53	0.13
Reach 1	6684	Max WS	679.67	178.11	185.54		185.55	0.000125	1.04	1638.05	392.62	0.13
Reach 1	6646	Max WS	679.65	178.06	185.53	183.53	185.55	0.000116	1.04	1757.84	449.67	0.13
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	679.51	178.05	184.51		184.61	0.000664	2.17	598.44	456.54	0.29
Reach 1	6539	Max WS	679.5	178.07	184.54		184.56	0.000186	1.1	1348.16	363.8	0.15
Reach 1	6496	Max WS	679.51	178.07	184.53		184.55	0.000161	1.03	1164.47	305.82	0.14
Reach 1	6400	Max WS	679.53	178.09	184.5		184.54	0.000277	1.47	980.47	237.41	0.19
Reach 1	6304	Max WS	679.52	178.11	184.49		184.51	0.000155	1.07	1149	297.05	0.14
Reach 1	6214	Max WS	679.55	178.12	184.48		184.5	0.00015	1.01	1329.81	353.31	0.14
Reach 1	5812	Max WS	679.48	178.08	184.4		184.42	0.000238	1.28	1311.95	350.5	0.18
Reach 1	5636	Max WS	679.5	177.99	184.37		184.39	0.000175	1.15	1393.1	328.23	0.15
Reach 1	5444	Max WS	679.5	177.93	184.3		184.34	0.000337	1.58	1025.23	257.13	0.21
Reach 1	5148	Max WS	679.49	177.83	184.16		184.23	0.000509	1.9	806.1	202.96	0.26
Reach 1	4980	Max WS	679.46	177.77	184.09		184.14	0.000413	1.73	925.76	227.74	0.23
Reach 1	4759	Max WS	679.46	177.67	184.01		184.05	0.000407	1.71	966.45	248.92	0.23
Reach 1	4647	Max WS	679.46	177.65	183.96		184.01	0.000411	1.7	942.07	234.02	0.23
Reach 1	4587	Max WS	679.46	177.62	183.94		183.98	0.000401	1.7	945.25	259.79	0.23
Reach 1	4542	Max WS	679.46	177.63	183.91	181.9	183.97	0.000435	1.76	901.08	277.49	0.24
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	679.42	177.56	183.58		183.63	0.000303	1.44	848.48	244.21	0.2
Reach 1	4441	Max WS	679.4	177.54	183.58		183.61	0.000197	1.14	1040.35	270.47	0.16
Reach 1	4399	Max WS	679.41	177.53	183.58		183.6	0.000153	1	1167.39	314.78	0.14
Reach 1	4291	Max WS	679.41	177.47	183.57		183.58	0.000092	0.8	1493.95	380.3	0.11
Reach 1	3909	Max WS	679.4	177.26	183.53		183.54	0.000101	0.86	1453.15	379.84	0.12
Reach 1	3757	Max WS	679.4	177.17	183.49		183.52	0.000232	1.3	934.13	245.65	0.18
Reach 1	3644	Max WS	679.39	177.14	183.47		183.5	0.000233	1.31	936.75	232.91	0.18
Reach 1	3510	Max WS	679.39	177.09	183.38		183.49	0.000739	2.3	683.53	185.53	0.31
Reach 1	3356	Max WS	679.39	177.02	183.26		183.37	0.000808	2.34	653.2	192.33	0.33
Reach 1	3246	Max WS	679.38	176.97	183.22		183.28	0.000472	1.83	912.11	338.52	0.25
Reach 1	3047	Max WS	679.38	176.86	183.13		183.16	0.000577	0.87	980.61	335.93	0.12
Reach 1	2950	Max WS	679.38	176.81	183.04		183.13	0.000537	1.99	840.2	317.37	0.27
Reach 1	2757	Max WS	679.38	176.72	182.94		183.02	0.00054	1.98	796.1	322.6	0.27
Reach 1	2652	Max WS	679.37	176.66	182.83		182.98	0.000871	2.46	633.6	308.61	0.34
Reach 1	2516	Max WS	679.38	176.61	182.7		182.85	0.001109	2.7	622.95	297.6	0.37
Reach 1	2363	Max WS	679.37	176.53	182.58		182.68	0.000843	2.28	756.22	269.65	0.33
Reach 1	2172	Max WS	679.37	176.44	182.44		182.51	0.000744	2.14	856.97	315.3	0.3
Reach 1	2044	Max WS	679.37	176.36	182.38		182.42	0.000429	1.65	1165.62	440.89	0.23
Reach 1	1898	Max WS	679.37	176.29	182.3		182.35	0.000616	1.9	971.24	405.04	0.27
Reach 1	1813	Max WS	679.35	176.25	182.25		182.3	0.000574	1.81	982.51	415.61	0.26
Reach 1	1642	Max WS	679.31	176.17	182.14		182.2	0.000634	2.01	914.2	392.57	0.28
Reach 1	1552	Max WS	679.26	176.13	182.11		182.15	0.000365	1.54	1152.55	495.06	0.22
Reach 1	1510	Max WS	679.35	176.11	182.1		182.13	0.000286	1.42	1265.94	460.97	0.19
Reach 1	1411	Max WS	653.39	176.06	182.05	180.48	182.1	0.00042	1.74	1163.34	445.35	0.24
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	679.38	176.04	181.54		181.58	0.000488	1.77	1143.33	457.25	0.25
Reach 1	1329	Max WS	679.35	176.02	181.52		181.55	0.000495	1.55	1148.97	436.16	0.24
Reach 1	1282	Max WS	679.36	175.99	181.5		181.53	0.000389	1.48	1237.21	436.35	0.22
Reach 1	1207	Max WS	679.37	175.96	181.48		181.5	0.000364	1.41	1294.48	453.19	0.21
Reach 1	1099	Max WS	679.36	175.91	181.44		181.46	0.000363	1.43	1217.26	389.1	0.21
Reach 1	1005	Max WS	679.35	175.86	181.4		181.43	0.000406	1.59	1149.63	408.13	0.22
Reach 1	890	Max WS	679.35	175.81	181.34	179.36	181.38	0.000499	1.68	1007.45	319.55	0.24

1/3 Between 1000-Year and PMF with Dam Breach												
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	28.96	188.13	201.21		201.21	0	0.02	1786.78	209.77	0
Reach 1	11048	Max WS	28.24	190.96	201.21	191.55	201.21	0	0.02	1555.32	355.51	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	395.96	185.59	190.91		191.36	0.005454	2.95	134.27	37.33	0.49
Reach 1	10747	Max WS	394.12	185.06	190.42		190.73	0.002637	3.87	228.69	67.09	0.54
Reach 1	10682	Max WS	393.9	184.94	190.32		190.5	0.002759	3.46	251.06	73.52	0.49
Reach 1	10591	Max WS	392.34	184.75	190		190.28	0.003126	3.99	234.86	79.41	0.58
Reach 1	10453	Max WS	390.75	184.46	189.64		189.9	0.00222	3.55	248.82	75.8	0.51
Reach 1	10316	Max WS	390.02	184.17	189.33		189.54	0.002732	3.46	256.1	84.14	0.5
Reach 1	10221	Max WS	388.83	183.98	189.08		189.29	0.00255	3.5	257.21	82.62	0.51
Reach 1	10093	Max WS	388.29	183.71	188.79		188.98	0.002157	3.26	265.77	81.16	0.48
Reach 1	9951	Max WS	387.71	183.42	188.43		188.66	0.002517	3.53	246.92	76.67	0.52
Reach 1	9814	Max WS	386.23	183.13	188.16		188.31	0.002206	3.11	277.22	85.09	0.46
Reach 1	9599	Max WS	385.24	182.67	187.71		187.87	0.001923	3.02	281.85	85.86	0.45
Reach 1	9362	Max WS	379.49	182.17	187.03		187.28	0.003282	3.81	233.24	81.98	0.57
Reach 1	9239	Max WS	376.28	181.92	186.91		186.95	0.000689	1.74	507.35	161.1	0.26
Reach 1	9104	Max WS	372.12	181.63	186.77		186.84	0.001015	2.22	385.34	112.1	0.32
Reach 1	9028	Max WS	367.35	181.48	186.67		186.75	0.001362	2.4	347.52	113.68	0.35
Reach 1	8891	Max WS	358.17	181.18	186.57		186.61	0.00055	1.72	511.6	152.89	0.25
Reach 1	8813	Max WS	338.36	181.02	186.46		186.56	0.001117	2.48	301.77	79.19	0.35
Reach 1	8671	Max WS	323.52	180.73	186.41		186.43	0.000288	1.28	570.74	140.44	0.18
Reach 1	8591	Max WS	320.14	180.57	186.41		186.41	0.000124	0.87	864.81	208.9	0.12
Reach 1	8323	Max WS	965.93	178.77	186.35		186.36	0.000142	1.1	2180.57	476.6	0.14
Reach 1	7976	Max WS	962.05	178.66	186.29		186.31	0.000159	1.19	1994.81	418.07	0.15
Reach 1	7762	Max WS	958.18	178.62	186.24		186.27	0.000239	1.45	1559.84	320.28	0.18
Reach 1	7619	Max WS	957.13	178.59	186.22		186.24	0.00016	1.2	1949.38	398.24	0.15
Reach 1	7217	Max WS	954.42	178.46	186.15		186.18	0.00018	1.32	1788	378.82	0.16
Reach 1	7061	Max WS	953.78	178.42	186.12		186.15	0.000201	1.37	1708.38	349.84	0.17
Reach 1	6824	Max WS	952.62	178.25	186.07		186.1	0.000198	1.41	1639.34	342.21	0.17
Reach 1	6733	Max WS	952.47	178.16	186.06		186.09	0.000167	1.3	1797.46	365.75	0.16
Reach 1	6684	Max WS	952.07	178.11	186.05		186.08	0.000174	1.29	1846.25	421.91	0.15
Reach 1	6646	Max WS	952.27	178.06	186.05	184.16	186.07	0.000156	1.27	1990.01	458.71	0.15
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	916.56	178.05	185.17		185.18	0.000153	1.12	1932.78	464.02	0.14
Reach 1	6539	Max WS	917.32	178.07	185.15		185.18	0.00021	1.25	1571.9	370.12	0.16
Reach 1	6496	Max WS	916.39	178.07	185.14		185.17	0.000184	1.18	1352.36	311.79	0.15
Reach 1	6400	Max WS	917.71	178.09	185.1		185.16	0.000332	1.72	1123.26	239.63	0.22
Reach 1	6304	Max WS	916.4	178.11	185.09		185.12	0.000184	1.25	1332.03	307.39	0.16
Reach 1	6214	Max WS	917.96	178.12	185.08		185.1	0.000173	1.16	1543.57	360.48	0.15
Reach 1	5812	Max WS	916.19	178.08	184.99		185.02	0.000273	1.48	1519.03	353.91	0.19
Reach 1	5636	Max WS	917.54	177.99	184.95		184.97	0.000213	1.36	1584.9	332.38	0.17
Reach 1	5444	Max WS	915.16	177.93	184.87		184.93	0.000407	1.85	1173.78	265.62	0.24
Reach 1	5148	Max WS	911.77	177.83	184.69		184.79	0.000621	2.24	914.53	205.42	0.3
Reach 1	4980	Max WS	910.66	177.77	184.61		184.68	0.000514	2.05	1044.75	232.01	0.26
Reach 1	4759	Max WS	909.06	177.67	184.5		184.57	0.000515	2.03	1097.71	346.42	0.27
Reach 1	4647	Max WS	907.91	177.65	184.44		184.51	0.000512	2.01	1063.7	321.39	0.26
Reach 1	4587	Max WS	907.79	177.62	184.41		184.48	0.000513	2.03	1080.14	336.87	0.27
Reach 1	4542	Max WS	907.5	177.63	184.38	181.9	184.46	0.000676	2.32	1036.53	375.97	0.31
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	889.64	177.56	184.08		184.14	0.000345	1.63	973.01	261.73	0.22
Reach 1	4441	Max WS	889.67	177.54	184.08		184.12	0.00023	1.31	1177.91	287.87	0.18
Reach 1	4399	Max WS	889.79	177.53	184.08		184.11	0.000182	1.17	1334.61	375	0.16
Reach 1	4291	Max WS	889.36	177.47	184.07		184.09	0.000108	0.92	1698.53	468.26	0.12
Reach 1	3909	Max WS	888.32	177.26	184.02		184.04	0.000118	0.98	1647.54	455.68	0.13
Reach 1	3757	Max WS	887.5	177.17	183.97		184.02	0.000278	1.51	1056.52	294.57	0.2
Reach 1	3644	Max WS	887.3	177.14	183.94		183.99	0.000317	1.61	1056.82	318.6	0.21
Reach 1	3510	Max WS	886.11	177.09	183.82		183.98	0.000978	2.79	778.4	249.06	0.37
Reach 1	3356	Max WS	884.16	177.02	183.67		183.82	0.000996	2.74	751.96	317.44	0.37
Reach 1	3246	Max WS	883.54	176.97	183.63		183.71	0.000558	2.09	1061.41	371.15	0.28
Reach 1	3047	Max WS	882.76	176.86	183.53		183.56	0.000667	0.98	1119.97	358.71	0.13
Reach 1	2950	Max WS	881.81	176.81	183.41		183.55	0.000724	2.42	967.84	353.77	0.32
Reach 1	2757	Max WS	881.22	176.72	183.29		183.4	0.000669	2.3	912.99	341.51	0.3
Reach 1	2652	Max WS	880.38	176.66	183.16		183.34	0.00101	2.76	736.75	313.67	0.37
Reach 1	2516	Max WS	879.72	176.61	183.01		183.18	0.00128	3.01	716.02	303.02	0.41
Reach 1	2363	Max WS	878.63	176.53	182.85		182.99	0.001085	2.68	831.48	275.4	0.37
Reach 1	2172	Max WS	876.24	176.44	182.67		182.78	0.000979	2.52	931.6	321.22	0.35
Reach 1	2044	Max WS	874.72	176.36	182.6		182.66	0.00056	1.94	1262.91	446.47	0.27
Reach 1	1898	Max WS	873.49	176.29	182.49		182.57	0.000821	2.25	1049.17	413.48	0.32
Reach 1	1813	Max WS	872.83	176.25	182.42		182.49	0.000782	2.16	1055.99	428.63	0.3
Reach 1	1642	Max WS	871.4	176.17	182.25		182.36	0.001003	2.56	961.1	414.79	0.35
Reach 1	1552	Max WS	871.11	176.13	182.21		182.27	0.000532	1.89	1202.4	496.26	0.26
Reach 1	1510	Max WS	871.16	176.11	182.2		182.24	0.000424	1.74	1310.74	470.59	0.23
Reach 1	1411	Max WS	870.75	176.06	182.13	180.65	182.2	0.000689	2.24	1196.28	447.79	0.3
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	870.85	176.04	182.08		182.13	0.000447	1.81	1398.19	481.39	0.24
Reach 1	1329	Max WS	870.81	176.02	182.06		182.1	0.000453	1.61	1387.13	442.55	0.23
Reach 1	1282	Max WS	870.73	175.99	182.05		182.08	0.00037	1.55	1475.68	441.27	0.21
Reach 1	1207	Max WS	870.64	175.96	182.02		182.05	0.000343	1.47	1548.29	494.92	0.2
Reach 1	1099	Max WS	870.57	175.91	181.98		182.01	0.000357	1.52	1430.32	391.54	0.21
Reach 1	1005	Max WS	870.62	175.86	181.94		181.98	0.000394	1.68	1375.99	420.36	0.22
Reach 1	890	Max WS	870.58	175.81	181.88	179.56	181.93	0.000499	1.8	1184.3	342.31	0.25

PMF Flood												
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	62.85	188.13	201.98		201.98	0	0.03	1950.52	215.17	0
Reach 1	11048	Max WS	62.81	190.96	201.98	191.85	201.98	0	0.04	1734.84	362.05	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	62.81	185.59	188.22		188.3	0.00122	1.24	50.7	24.83	0.28
Reach 1	10747	Max WS	62.8	185.06	188.02		188.11	0.001287	1.79	79.5	55.99	0.34
Reach 1	10682	Max WS	62.79	184.94	187.95		188	0.001593	1.73	88.78	63.56	0.34
Reach 1	10591	Max WS	62.79	184.75	187.79		187.88	0.001481	1.84	81.4	60.86	0.36
Reach 1	10453	Max WS	62.78	184.46	187.66		187.71	0.000658	1.37	106.88	67.94	0.26
Reach 1	10316	Max WS	62.76	184.17	187.57		187.61	0.000787	1.37	112.84	78.17	0.25
Reach 1	10221	Max WS	62.74	183.98	187.52		187.54	0.000474	1.16	132.33	77.01	0.21
Reach 1	10093	Max WS	62.72	183.71	187.48		187.49	0.000246	0.88	162.36	76.36	0.15
Reach 1	9951	Max WS	62.68	183.42	187.45		187.46	0.000188	0.82	173.34	72.71	0.14
Reach 1	9814	Max WS	62.65	183.13	187.43		187.44	0.000126	0.66	215.72	83.05	0.11
Reach 1	9599	Max WS	62.54	182.67	187.41		187.41	0.000068	0.54	256.11	84.77	0.08
Reach 1	9362	Max WS	62.44	182.17	187.39		187.4	0.000063	0.55	263.05	84.21	0.08
Reach 1	9239	Max WS	62.37	181.92	187.39		187.39	0.000012	0.25	586.34	166.72	0.04
Reach 1	9104	Max WS	62.41	181.63	187.39		187.39	0.000017	0.31	455.58	115.72	0.04
Reach 1	9028	Max WS	62.4	181.48	187.39		187.39	0.000021	0.33	431.04	119.2	0.04
Reach 1	8891	Max WS	62.35	181.18	187.39		187.39	0.000008	0.24	638.61	158.62	0.03
Reach 1	8813	Max WS	62.3	181.02	187.38		187.39	0.00002	0.37	376.85	82.85	0.05
Reach 1	8671	Max WS	62.28	180.73	187.38		187.38	0.000005	0.2	709.96	146.06	0.03
Reach 1	8591	Max WS	62.33	180.57	187.38		187.38	0.000002	0.13	1072.18	215.26	0.02
Reach 1	8323	Max WS	1471.17	178.77	187.31		187.33	0.000178	1.35	2641.06	480.65	0.16
Reach 1	7976	Max WS	1471.24	178.66	187.24		187.27	0.000209	1.49	2391.73	423.18	0.17
Reach 1	7762	Max WS	1471.18	178.62	187.17		187.22	0.000325	1.85	1858.48	327.16	0.22
Reach 1	7619	Max WS	1471.15	178.59	187.14		187.17	0.000218	1.53	2317.38	404.1	0.18
Reach 1	7217	Max WS	1471.06	178.46	187.04		187.08	0.000251	1.7	2128.24	388.28	0.19
Reach 1	7061	Max WS	1471.1	178.42	187		187.04	0.000284	1.76	2015.93	353.85	0.2
Reach 1	6824	Max WS	1471.11	178.25	186.92		186.98	0.000289	1.83	1960.86	407.28	0.21
Reach 1	6733	Max WS	1471.08	178.16	186.9		186.95	0.00024	1.68	2161.51	454.72	0.19
Reach 1	6684	Max WS	1471.05	178.11	186.9		186.93	0.000254	1.67	2221.35	451.63	0.19
Reach 1	6646	Max WS	1471.1	178.06	186.89	184.55	186.92	0.000216	1.6	2379.33	465.34	0.18
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	1470.51	178.05	186.47		186.5	0.000166	1.32	2548.35	478.37	0.15
Reach 1	6539	Max WS	1470.56	178.07	186.46		186.49	0.000232	1.5	2062.77	383.51	0.17
Reach 1	6496	Max WS	1470.5	178.07	186.44		186.48	0.000209	1.43	1765.18	324.43	0.17
Reach 1	6400	Max WS	1470.52	178.09	186.38		186.47	0.000402	2.13	1433.92	245.31	0.24
Reach 1	6304	Max WS	1470.53	178.11	186.38		186.42	0.000214	1.52	1733.09	317.76	0.18
Reach 1	6214	Max WS	1470.53	178.12	186.37		186.4	0.000198	1.42	2021.6	395.7	0.17
Reach 1	5812	Max WS	1470.42	178.08	186.26		186.31	0.000308	1.79	1974.08	361.63	0.21
Reach 1	5636	Max WS	1470.38	177.99	186.21		186.26	0.000259	1.69	2023.55	363.24	0.2
Reach 1	5444	Max WS	1470.2	177.93	186.12		186.2	0.000479	2.27	1565.85	389.34	0.26
Reach 1	5148	Max WS	1470.12	177.83	185.9		186.04	0.000733	2.76	1275.95	366.9	0.33
Reach 1	4980	Max WS	1470.07	177.77	185.78		185.92	0.000873	2.99	1421.6	404.31	0.35
Reach 1	4759	Max WS	1469.94	177.67	185.61		185.72	0.0008	2.84	1706.77	634.73	0.34
Reach 1	4647	Max WS	1469.82	177.65	185.53		185.63	0.000643	2.53	1624.9	555.41	0.31
Reach 1	4587	Max WS	1469.85	177.62	185.49		185.59	0.000649	2.56	1651.31	574.16	0.31
Reach 1	4542	Max WS	1469.79	177.63	185.47	182.16	185.56	0.000668	2.59	1704.63	669.38	0.31
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	1469.79	177.56	185.43		185.49	0.000326	1.83	1655.08	602.21	0.22
Reach 1	4441	Max WS	1469.79	177.54	185.42		185.47	0.000241	1.55	1873.66	600.92	0.19
Reach 1	4399	Max WS	1469.79	177.53	185.42		185.46	0.000185	1.36	2061.06	633.69	0.17
Reach 1	4291	Max WS	1469.75	177.47	185.41		185.44	0.000136	1.19	2547.86	719.24	0.14
Reach 1	3909	Max WS	1469.7	177.26	185.34		185.37	0.000176	1.37	2471.89	713.08	0.16
Reach 1	3757	Max WS	1469.67	177.17	185.26		185.34	0.000433	2.15	1664.68	562.79	0.25
Reach 1	3644	Max WS	1469.67	177.14	185.22		185.3	0.000449	2.19	1638.76	540.6	0.26
Reach 1	3510	Max WS	1469.65	177.09	185.07		185.32	0.001329	3.69	1208.22	401.16	0.44
Reach 1	3356	Max WS	1469.61	177.02	184.92		185.08	0.000944	3.04	1198.31	375.31	0.37
Reach 1	3246	Max WS	1469.57	176.97	184.88		184.97	0.00056	2.38	1533.37	383.64	0.29
Reach 1	3047	Max WS	1469.58	176.86	184.78		184.83	0.00064	1.1	1590.95	384.92	0.13
Reach 1	2950	Max WS	1469.56	176.81	184.67		184.82	0.000711	2.73	1432.63	380.18	0.33
Reach 1	2757	Max WS	1469.47	176.72	184.54		184.67	0.000666	2.61	1356.28	367.16	0.31
Reach 1	2652	Max WS	1469.47	176.66	184.43		184.61	0.000877	2.95	1154.65	353.68	0.36
Reach 1	2516	Max WS	1469.43	176.61	184.31		184.47	0.001008	3.07	1130.28	336.01	0.37
Reach 1	2363	Max WS	1469.39	176.53	184.15		184.32	0.001082	3.1	1204.18	301.31	0.39
Reach 1	2172	Max WS	1469.39	176.44	183.99		184.1	0.000941	2.86	1388.89	386.83	0.35
Reach 1	2044	Max WS	1469.38	176.36	183.93		183.99	0.000477	2.08	1888.57	488.6	0.26
Reach 1	1898	Max WS	1469.34	176.29	183.84		183.91	0.000643	2.33	1632.25	453.35	0.29
Reach 1	1813	Max WS	1469.38	176.25	183.79		183.85	0.000616	2.24	1673.99	490.74	0.28
Reach 1	1642	Max WS	1469.35	176.17	183.66		183.75	0.000717	2.53	1593.45	491.83	0.31
Reach 1	1552	Max WS	1469.34	176.13	183.63		183.69	0.000384	1.88	1936.44	548.82	0.23
Reach 1	1510	Max WS	1469.33	176.11	183.62		183.67	0.000334	1.8	2062.19	568.19	0.22
Reach 1	1411	Max WS	1469.33	176.06	183.57	181.13	183.64	0.000579	2.39	1931.29	611.39	0.29
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	1469.34	176.04	183.53		183.58	0.000371	1.92	2132.77	541.31	0.23
Reach 1	1329	Max WS	1469.34	176.02	183.52		183.56	0.000403	1.81	2097.66	573.92	0.23
Reach 1	1282	Max WS	1469.33	175.99	183.5		183.54	0.000356	1.79	2198.57	559.86	0.22
Reach 1	1207	Max WS	1469.34	175.96	183.48		183.51	0.000305	1.63	2307.48	531.73	0.2
Reach 1	1099	Max WS	1469.33	175.91	183.44		183.48	0.000351	1.77	2095.07	529.85	0.22
Reach 1	1005	Max WS	1469.34	175.86	183.4		183.45	0.00037	1.89	2051	533.66	0.23
Reach 1	890	Max WS	1469.32	175.81	183.34	180.02	183.41	0.000499	2.11	1784.41	499.57	0.26

PMF Flood with Dam Breach												
Reach	**River Sta**	**Profile**	**Q Total**	**Min Ch El**	**W.S. Elev**	**Crit W.S.**	**E.G. Elev**	**E.G. Slope**	**Vel Chnl**	**Flow Area**	**Top Width**	**Froude # Chl**
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach 1	11254	Max WS	62.89	188.13	201.98		201.98	0	0.03	1950.25	215.16	0
Reach 1	11048	Max WS	62.76	190.96	201.98	191.85	201.98	0	0.04	1734.56	362.04	0
Reach 1	10994 Deer Creek Dam		Inl Struct									
Reach 1	10910	Max WS	482.55	185.59	191.4		191.91	0.00586	3.16	153.18	40.65	0.51
Reach 1	10747	Max WS	479.99	185.06	190.91		191.25	0.002632	4.1	261.86	68.54	0.55
Reach 1	10682	Max WS	479.76	184.94	190.82		191.02	0.002706	3.65	288.15	75.42	0.49
Reach 1	10591	Max WS	478.81	184.75	190.52		190.81	0.002917	4.12	276.73	82.52	0.57
Reach 1	10453	Max WS	477.35	184.46	190.18		190.45	0.002127	3.72	289.95	78.27	0.51
Reach 1	10316	Max WS	476.18	184.17	189.91		190.11	0.002413	3.51	304.8	86.09	0.48
Reach 1	10221	Max WS	475.51	183.98	189.68		189.89	0.002229	3.53	307.8	84.82	0.48
Reach 1	10093	Max WS	474.64	183.71	189.43		189.63	0.001877	3.31	318.81	83.84	0.46
Reach 1	9951	Max WS	473.39	183.42	189.13		189.36	0.002072	3.51	301.61	79.54	0.48
Reach 1	9814	Max WS	472.5	183.13	188.93		189.07	0.001711	3.03	343.2	87.34	0.42
Reach 1	9599	Max WS	470.81	182.67	188.6		188.74	0.001374	2.87	359.21	89.05	0.39
Reach 1	9362	Max WS	463.57	182.17	188.21		188.37	0.001746	3.23	334.14	90.13	0.43
Reach 1	9239	Max WS	461.86	181.92	188.16		188.19	0.000368	1.49	717.74	174.39	0.2
Reach 1	9104	Max WS	458.71	181.63	188.08		188.13	0.00057	1.95	537.25	120.3	0.25
Reach 1	9028	Max WS	455.72	181.48	188.03		188.09	0.000686	2.01	509.16	123.7	0.26
Reach 1	8891	Max WS	450.78	181.18	187.99		188.01	0.000291	1.48	735.01	163.86	0.19
Reach 1	8813	Max WS	439.98	181.02	187.9		187.99	0.000702	2.32	420.44	84.66	0.29
Reach 1	8671	Max WS	434.1	180.73	187.89		187.91	0.000197	1.25	784.25	149.21	0.15
Reach 1	8591	Max WS	434.08	180.57	187.88		187.89	0.000087	0.85	1181.03	218.7	0.1
Reach 1	8323	Max WS	1835.42	178.77	187.79		187.82	0.000212	1.54	2871.92	483.11	0.18
Reach 1	7976	Max WS	1831.28	178.66	187.7		187.74	0.000253	1.7	2589.45	426.45	0.19
Reach 1	7762	Max WS	1827.13	178.62	187.61		187.68	0.000396	2.12	2005.95	330.07	0.24
Reach 1	7619	Max WS	1826.56	178.59	187.59		187.63	0.000265	1.75	2497.84	406.91	0.2
Reach 1	7217	Max WS	1823.1	178.46	187.46		187.52	0.000307	1.94	2293.22	392.38	0.22
Reach 1	7061	Max WS	1821.43	178.42	187.41		187.46	0.00035	2.02	2162.5	356.98	0.23
Reach 1	6824	Max WS	1819.88	178.25	187.32		187.39	0.000353	2.09	2123.75	413.46	0.23
Reach 1	6733	Max WS	1819.51	178.16	187.3		187.35	0.000293	1.92	2341.78	460.23	0.21
Reach 1	6684	Max WS	1819.48	178.11	187.29		187.34	0.00031	1.91	2399.36	455.77	0.21
Reach 1	6646	Max WS	1819.17	178.06	187.28	184.55	187.32	0.000264	1.82	2561.76	468.35	0.2
Reach 1	6620 Townline Rd		Bridge									
Reach 1	6587	Max WS	1794.08	178.05	186.94		186.97	0.00019	1.47	2773.65	483.61	0.16
Reach 1	6539	Max WS	1793.41	178.07	186.92		186.96	0.000267	1.67	2242.28	388.9	0.19
Reach 1	6496	Max WS	1791.48	178.07	186.9		186.95	0.000241	1.61	1915.98	329.76	0.18
Reach 1	6400	Max WS	1786.3	178.09	186.82		186.94	0.000473	2.4	1544.31	247.81	0.27
Reach 1	6304	Max WS	1786.92	178.11	186.83		186.89	0.000253	1.72	1882.39	342.21	0.19
Reach 1	6214	Max WS	1785.65	178.12	186.82		186.86	0.000238	1.62	2204.41	416.15	0.19
Reach 1	5812	Max WS	1779.77	178.08	186.69		186.75	0.000355	1.99	2130.86	364.13	0.23
Reach 1	5636	Max WS	1775.88	177.99	186.64		186.69	0.000303	1.9	2179.26	370.77	0.22
Reach 1	5444	Max WS	1770.33	177.93	186.53		186.62	0.000545	2.5	1728.65	401.28	0.28
Reach 1	5148	Max WS	1755.76	177.83	186.29		186.45	0.000803	3	1420.06	375.53	0.35
Reach 1	4980	Max WS	1744.85	177.77	186.16		186.31	0.000914	3.17	1576.92	406.43	0.37
Reach 1	4759	Max WS	1731.75	177.67	186		186.1	0.000757	2.87	1958.62	651.29	0.34
Reach 1	4647	Max WS	1724.09	177.65	185.92		186.02	0.000649	2.63	1844.23	566.7	0.31
Reach 1	4587	Max WS	1719.43	177.62	185.88		185.99	0.000651	2.66	1877.27	581.76	0.31
Reach 1	4542	Max WS	1714.77	177.63	185.86	182.36	185.95	0.000632	2.61	1970.29	677.24	0.31
Reach 1	4516 6th Conession		Bridge									
Reach 1	4489	Max WS	1714.77	177.56	185.82		185.89	0.000324	1.89	1893.97	615.34	0.22
Reach 1	4441	Max WS	1714.1	177.54	185.81		185.87	0.000248	1.63	2112.64	627.85	0.19
Reach 1	4399	Max WS	1714.72	177.53	185.81		185.85	0.000189	1.42	2309.11	637.05	0.17
Reach 1	4291	Max WS	1712.18	177.47	185.8		185.83	0.000137	1.23	2829.82	722.18	0.14
Reach 1	3909	Max WS	1709.32	177.26	185.74		185.76	0.00017	1.4	2753.13	716.88	0.16
Reach 1	3757	Max WS	1707.7	177.17	185.66		185.73	0.000401	2.14	1889.47	567.78	0.25
Reach 1	3644	Max WS	1706.68	177.14	185.62		185.7	0.00042	2.19	1855.76	545.47	0.25
Reach 1	3510	Max WS	1703.9	177.09	185.47		185.71	0.001266	3.74	1371.6	407.42	0.43
Reach 1	3356	Max WS	1701.22	177.02	185.33		185.49	0.000915	3.11	1352.17	379.31	0.37
Reach 1	3246	Max WS	1700.35	176.97	185.29		185.38	0.000564	2.48	1690.88	390.79	0.29
Reach 1	3047	Max WS	1698.98	176.86	185.19		185.24	0.000638	1.14	1749.12	389.62	0.13
Reach 1	2950	Max WS	1696.91	176.81	185.08		185.23	0.000704	2.82	1588.14	381.31	0.33
Reach 1	2757	Max WS	1694.81	176.72	184.95		185.08	0.000666	2.71	1511.03	390.85	0.32
Reach 1	2652	Max WS	1693.01	176.66	184.84		185.02	0.000834	2.99	1303.92	366.35	0.35
Reach 1	2516	Max WS	1690.94	176.61	184.73		184.89	0.000973	3.14	1275.13	356.49	0.37
Reach 1	2363	Max WS	1687.31	176.53	184.54		184.75	0.001275	3.5	1330.03	357.48	0.42
Reach 1	2172	Max WS	1684.38	176.44	184.35		184.49	0.001061	3.15	1539.87	425.27	0.38
Reach 1	2044	Max WS	1683.36	176.36	184.3		184.36	0.000477	2.15	2069.81	502.89	0.26
Reach 1	1898	Max WS	1682.17	176.29	184.2		184.28	0.000645	2.42	1804.74	485.7	0.3
Reach 1	1813	Max WS	1681.53	176.25	184.16		184.22	0.000601	2.3	1858.3	509.93	0.28
Reach 1	1642	Max WS	1680.66	176.17	184.03		184.12	0.000685	2.56	1780.67	511.46	0.31
Reach 1	1552	Max WS	1680.13	176.13	184.01		184.06	0.00037	1.92	2143.41	555.5	0.23
Reach 1	1510	Max WS	1680.27	176.11	184		184.04	0.00033	1.85	2277.26	582.45	0.22
Reach 1	1411	Max WS	1679.97	176.06	183.94	181.31	184.01	0.000549	2.41	2165.19	628.62	0.28
Reach 1	1391 County Rd 60		Bridge									
Reach 1	1373	Max WS	1679.97	176.04	183.91		183.97	0.000375	2	2353.22	601.64	0.23
Reach 1	1329	Max WS	1679.83	176.02	183.9		183.94	0.000395	1.87	2319.97	584.94	0.23
Reach 1	1282	Max WS	1679.49	175.99	183.88		183.93	0.000357	1.86	2415.55	574.18	0.22
Reach 1	1207	Max WS	1679.66	175.96	183.86		183.9	0.000308	1.7	2516.56	568.28	0.2
Reach 1	1099	Max WS	1679.49	175.91	183.82		183.87	0.000356	1.85	2301.47	545.48	0.22
Reach 1	1005	Max WS	1679.46	175.86	183.78		183.83	0.000371	1.96	2256.02	538.32	0.23
Reach 1	890	Max WS	1679.44	175.81	183.72	180.17	183.79	0.000499	2.19	1976.13	510.37	0.26





Legend

- Ontario Roads
- Sunny Day
- Sunny Day Dam Break
- Cross Sections
- Watercourse

N W E S

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

1. Modeled in ArcMap 10.7, HEC-RAS Version 6.6.
2. All units are metric and in meters unless otherwise specified.
3. Aerial imagery from South Western Ontario Orthophotography Project (SWOOP) (2015).

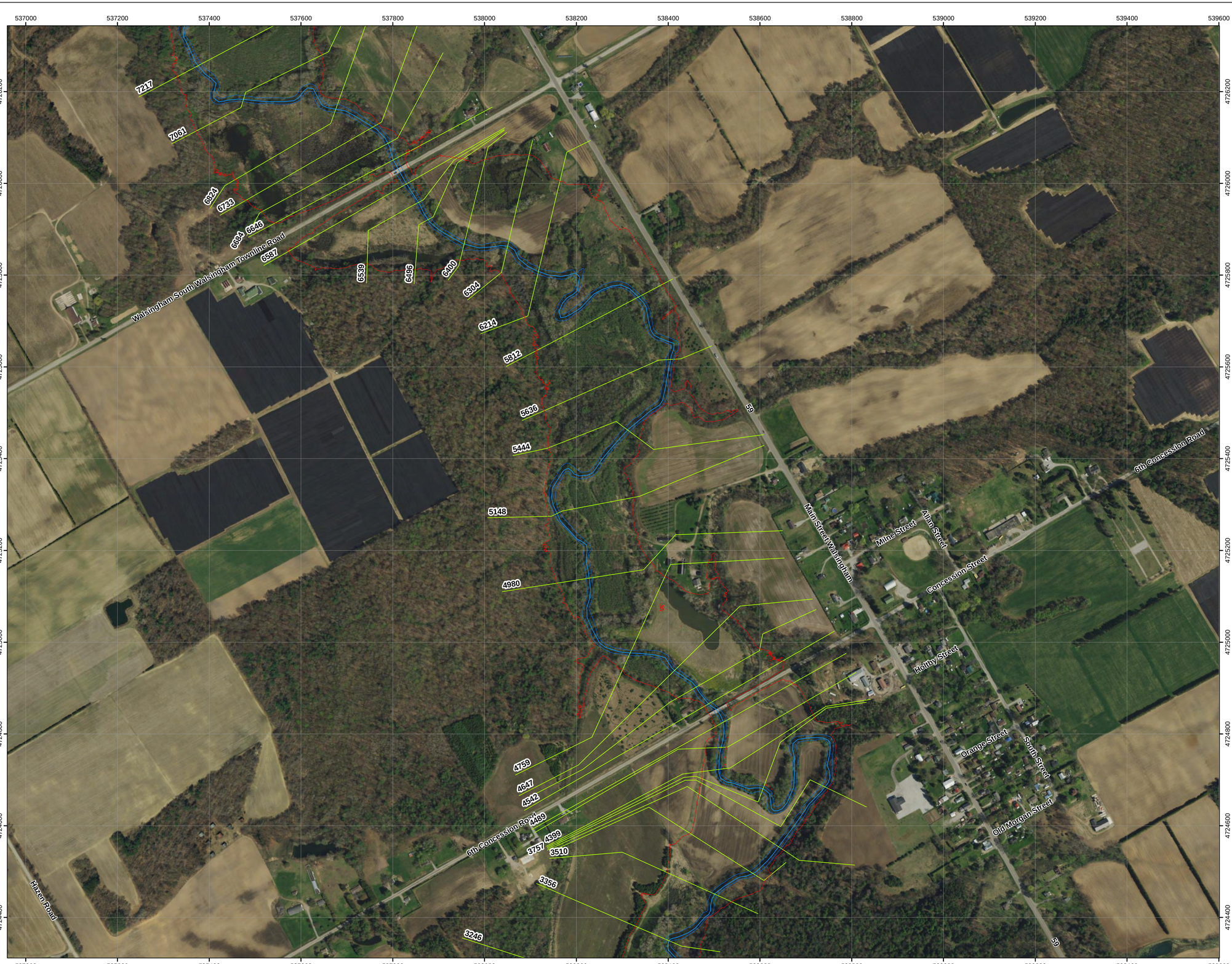
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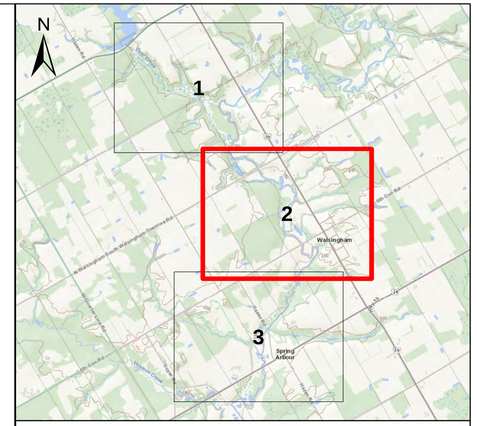
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Engineer: MC
Designed By: MC
Checked By: DG
Date: 02/27/2024
Project No. 24-5684
Map File No. Sunny Day

Deer Creek Dam Safety Review
Long Point Region Conservation Authority

Sunny Day Dam Break
Inundation Map 1





Legend

- Ontario Roads
- Sunny Day
- Sunny Day Dam Break
- Cross Sections
- Watercourse

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

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1:4,000

0 62.5 125 250 375 500
Meters
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Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

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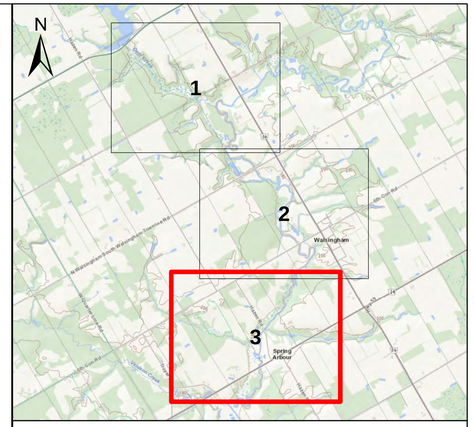
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Engineer: MC	
Designed By: MC	
Checked By: DG	
Date: 02/27/2024	
Project No. 24-5684	
Map File No. Sunny Day	

Deer Creek Dam Safety Review
Long Point Region Conservation Authority

Sunny Day Dam Break
Inundation Map 2





Legend

- Ontario Roads
- Sunny Day
- Sunny Day Dam Break
- Cross Sections
- Watercourse

N W E S

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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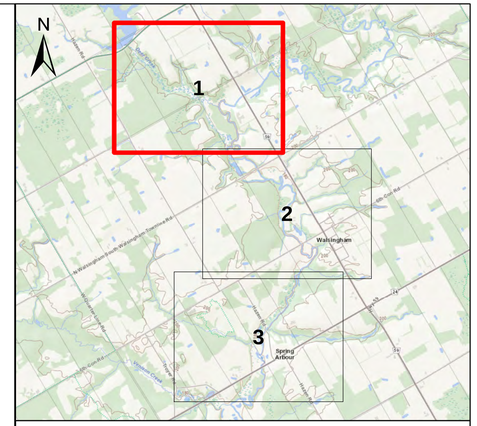
Engineer: MC	
Designed By: MC	
Checked By: DG	
Date: 02/27/2024	
Project No. 24-5684	
Map File No. Sunny Day	

Deer Creek Dam Safety Review

Long Point Region Conservation Authority

Sunny Day Dam Break Inundation Map 3





Legend

- Ontario Roads
- 25-year
- 25-year Dam Break
- Cross Sections
- Watercourse

N
W E
S

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

1. Modeled in ArcMap 10.7, HEC-RAS Version 6.6.
2. All units are metric and in meters unless otherwise specified.
3. Aerial imagery from South Western Ontario Orthophotography Project (SWOOP) (2015).

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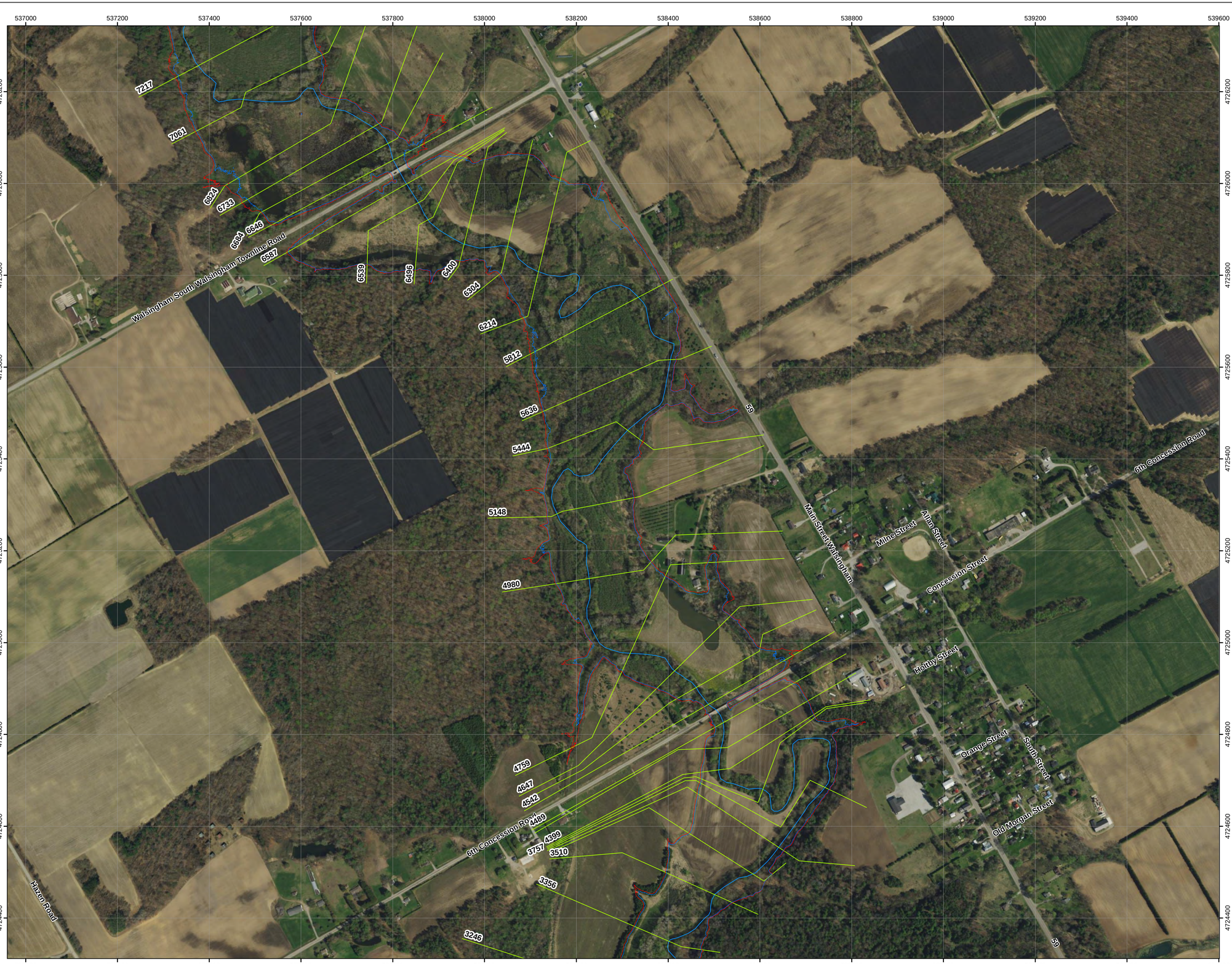
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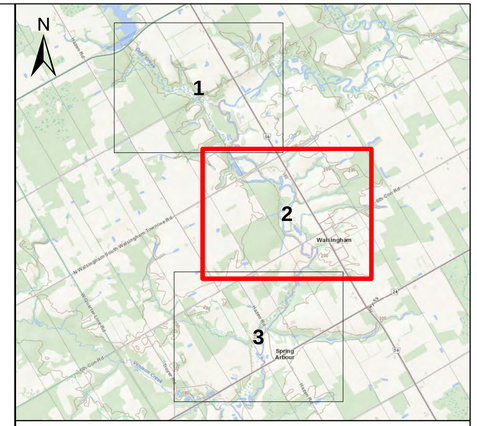
Engineer: MC
Designed By: MC
Checked By: DG
Date: 02/27/2024
Project No. 24-5684
Map File No. 25-Year

Deer Creek Dam Safety Review

Long Point Region Conservation Authority

25-Year Dam Break Inundation Map 1





Legend

- Ontario Roads
- 25-year
- 25-year Dam Break
- Cross Sections
- Watercourse

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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2. All units are metric and in meters unless otherwise specified.
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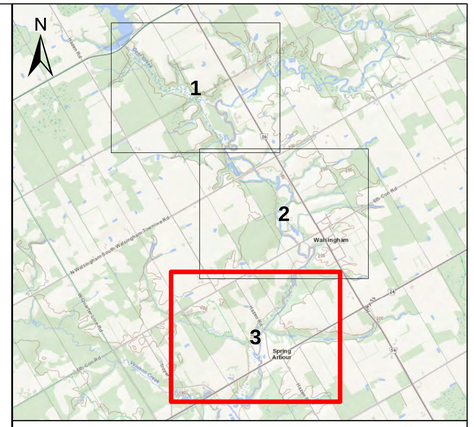
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Designed By: MC	
Checked By: DG	
Date: 02/27/2024	
Project No. 24-5684	
Map File No. 25-Year	

Deer Creek Dam Safety Review
Long Point Region Conservation Authority

25-Year Dam Break
Inundation Map 2





Legend

- Ontario Roads
- 25-year
- 25-year Dam Break
- Cross Sections
- Watercourse

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

1. Modeled in ArcMap 10.7, HEC-RAS Version 6.6.
2. All units are metric and in meters unless otherwise specified.
3. Aerial imagery from South Western Ontario Orthophotography Project (SWOOP) (2015).

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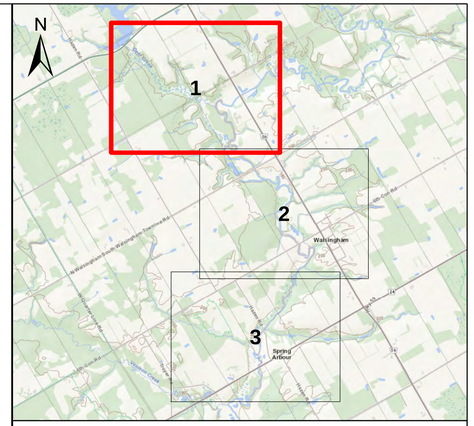
Engineer:	MC
Designed By:	MC
Checked By:	DG
Date:	02/27/2024
Project No.	24-5684
Map File No.	25-Year

Deer Creek Dam Safety Review

Long Point Region Conservation Authority

25-Year Dam Break Inundation Map 3





Legend

- Ontario Roads
- 100-year
- 100-year Dam Break
- Cross Sections
- Watercourse

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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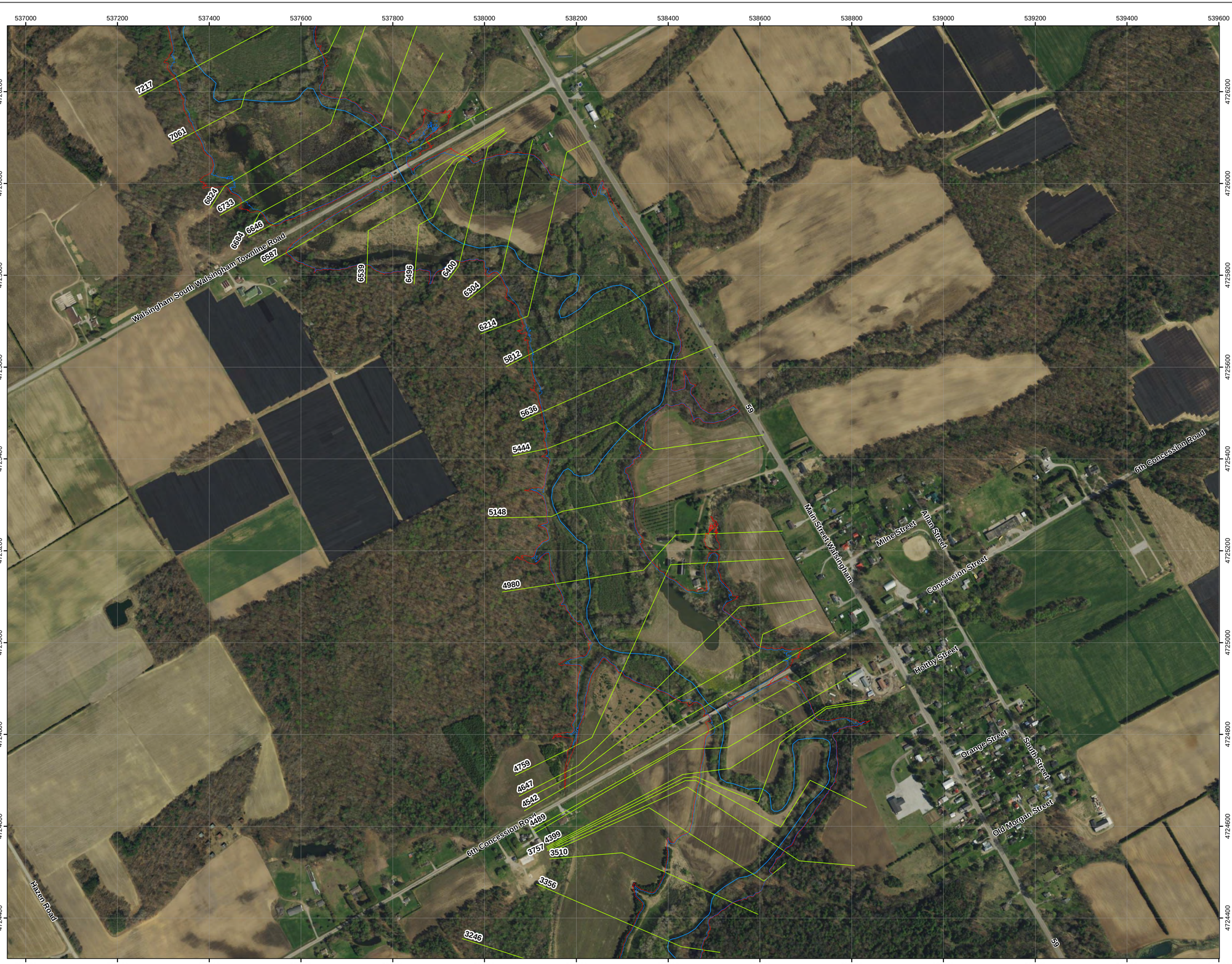
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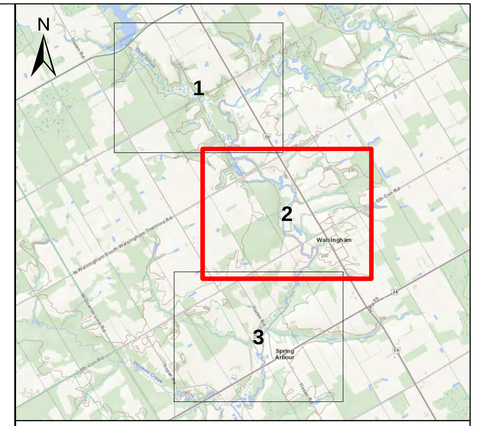
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Engineer: MC
Designed By: MC
Checked By: DG
Date: 02/27/2024
Project No. 24-5684
Map File No. 100-Year

Deer Creek Dam Safety Review
Long Point Region Conservation Authority

100-Year Dam Break Inundation Map 1





Legend

- Ontario Roads
- 100-year
- 100-year Dam Break
- Cross Sections
- Watercourse

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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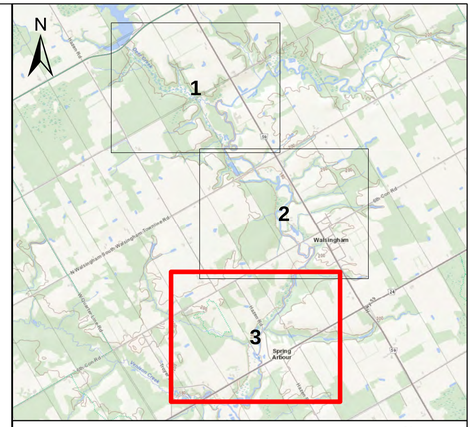
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Engineer: MC	
Designed By: MC	
Checked By: DG	
Date: 02/27/2024	
Project No. 24-5684	
Map File No. 100-Year	

Deer Creek Dam Safety Review
Long Point Region Conservation Authority

100-Year Dam Break
Inundation Map 2





Legend

- Ontario Roads
- 100-year
- 100-year Dam Break
- Cross Sections
- Watercourse

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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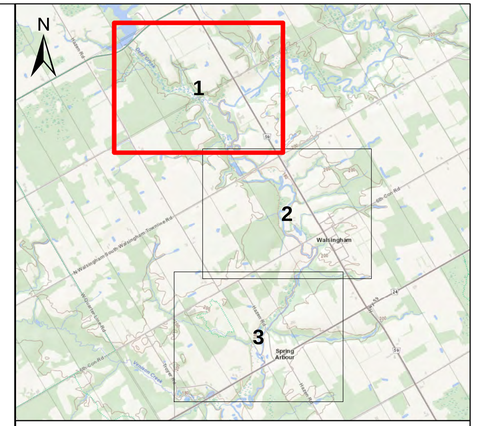
Engineer:	MC
Designed By:	MC
Checked By:	DG
Date:	02/27/2024
Project No.	24-5684
Map File No.	100-Year

Deer Creek Dam Safety Review

Long Point Region Conservation Authority

100-Year Dam Break
Inundation Map 3





Legend

- Ontario Roads
- 1000-year
- 1000-year Dam Break
- Cross Sections
- Watercourse

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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2. All units are metric and in meters unless otherwise specified.
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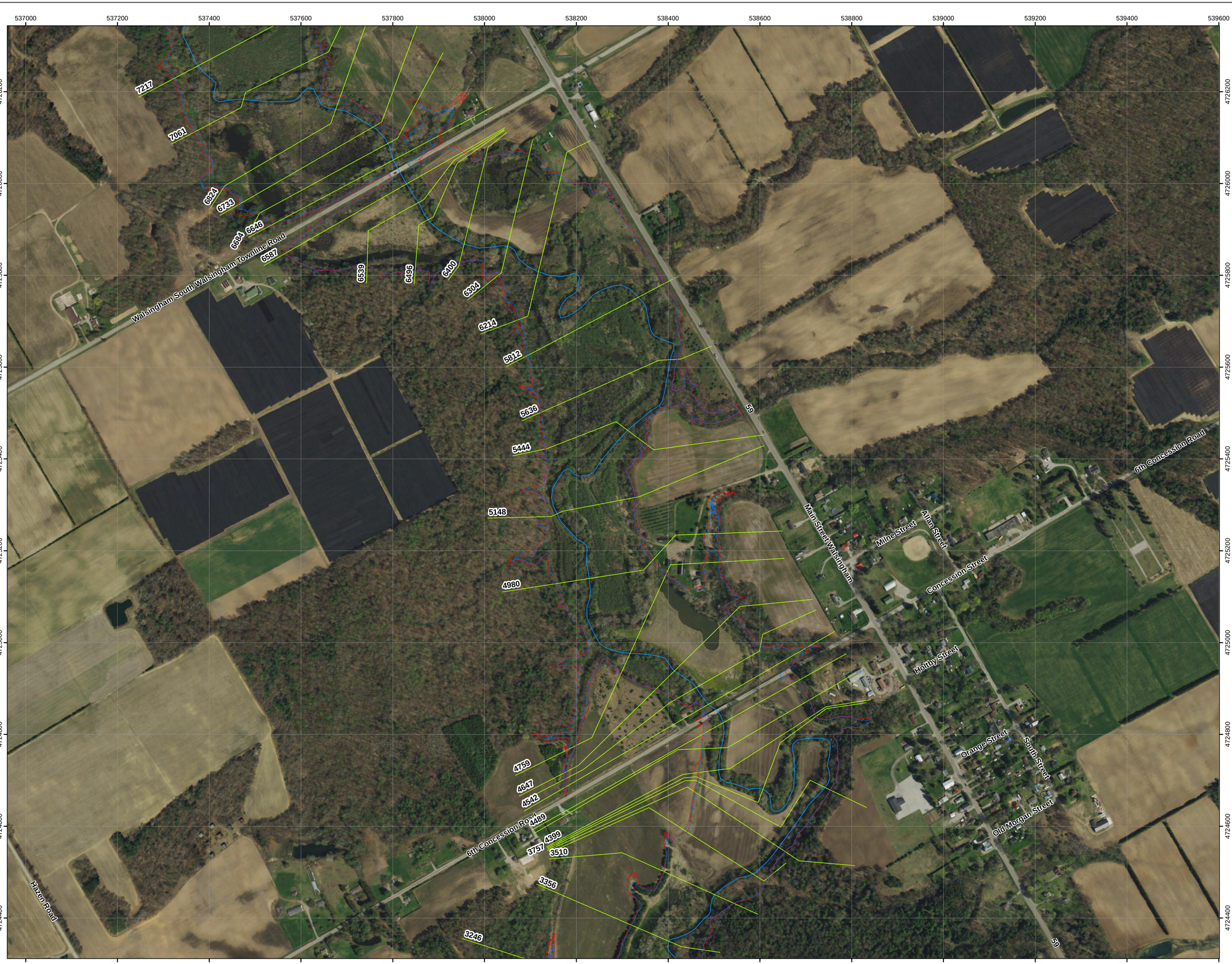
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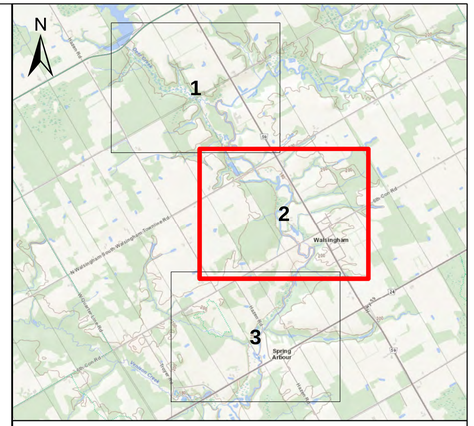
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Engineer: MC
Designed By: MC
Checked By: DG
Date: 02/27/2024
Project No. 24-5684
Map File No. 1000-Year

Deer Creek Dam Safety Review
Long Point Region Conservation Authority

1000-Year Dam Break Inundation Map 1





Legend

- Ontario Roads
- 1000-year
- 1000-year Dam Break
- Cross Sections
- Watercourse

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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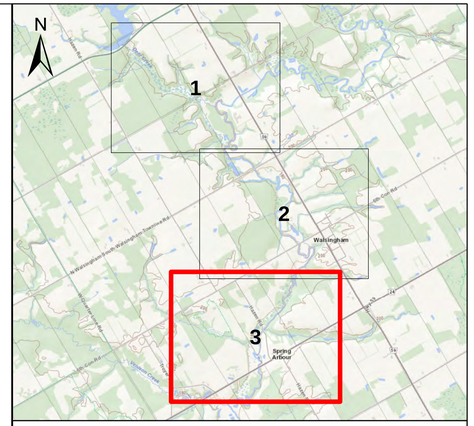
P. 705.742.2297
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Engineer: MC	
Designed By: MC	
Checked By: DG	
Date: 02/27/2024	
Project No. 24-5684	
Map File No. 1000-Year	

Deer Creek Dam Safety Review
Long Point Region Conservation Authority

1000-Year Dam Break
Inundation Map 2





Legend

- Ontario Roads
- 1000-year
- 1000-year Dam Break
- Cross Sections
- Watercourse

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

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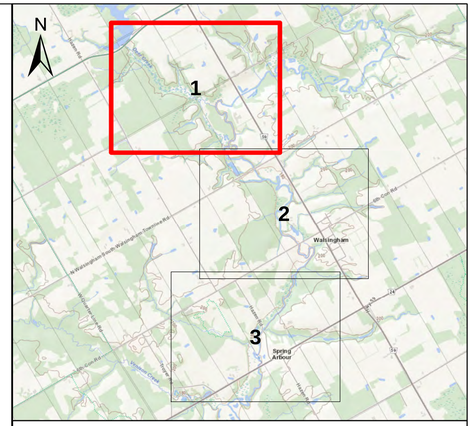
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Designed By: MC
Checked By: DG
Date: 02/27/2024
Project No. 24-5684
Map File No. 1000-Year

Deer Creek Dam Safety Review

Long Point Region Conservation Authority

1000-Year Dam Break Inundation Map 3





Legend

- Ontario Roads
- 1/3 Between 1000-year and PMF
- 1/3 Between 1000-year and PMF Dam Break
- Cross Sections
- Watercourse

N
W E
S

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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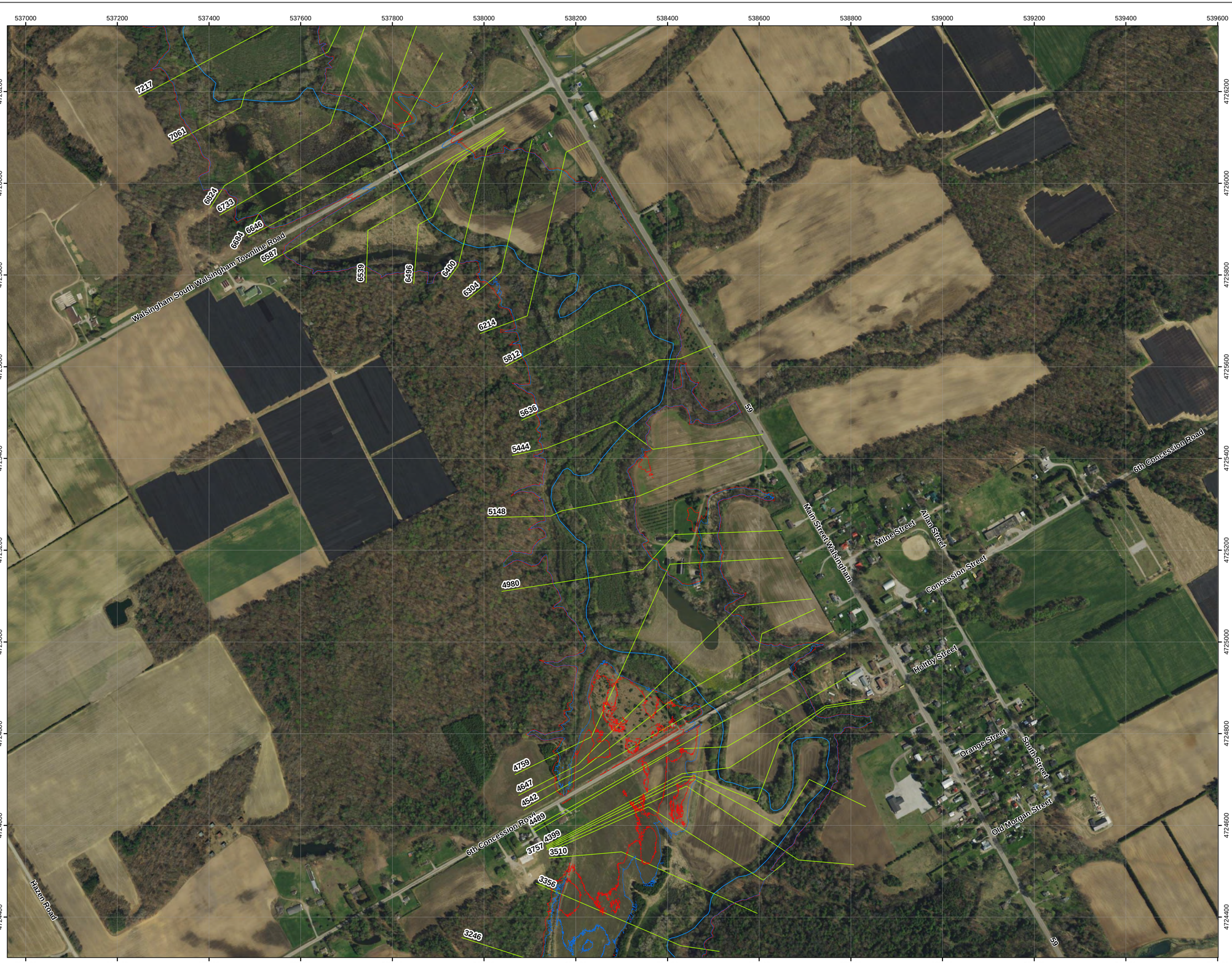
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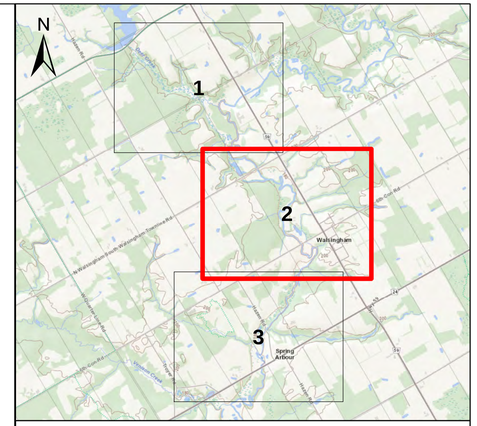
Engineer: MC
Designed By: MC
Checked By: DG
Date: 02/27/2024
Project No. 24-5684
Map File No. IDF

Deer Creek Dam Safety Review

Long Point Region Conservation Authority

IDF (1/3 Between 1000-Year and PMF) Dam Break Inundation Map 1





Legend

- Ontario Roads
- 1/3 Between 1000-year and PMF
- 1/3 Between 1000-year and PMF Dam Break
- Cross Sections
- Watercourse

N
W E
S

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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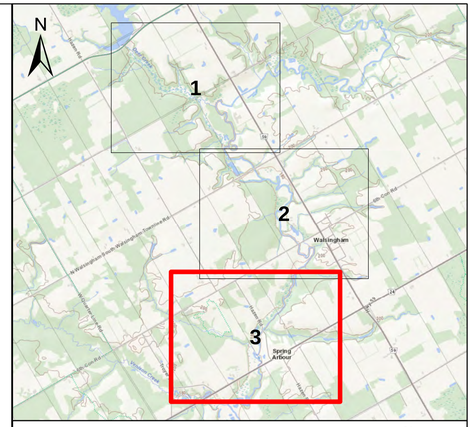
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Designed By: MC	
Checked By: DG	
Date: 02/27/2024	
Project No. 24-5684	
Map File No. IDF	

Deer Creek Dam Safety Review

Long Point Region Conservation Authority

IDF (1/3 Between 1000-Year and PMF) Dam Break Inundation Map 2





Legend

- Ontario Roads
- 1/3 Between 1000-year and PMF
- 1/3 Between 1000-year and PMF Dam Break
- Cross Sections
- Watercourse

N
W E
S

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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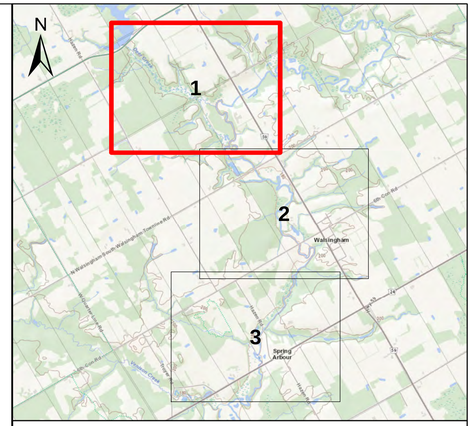
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Project No.	24-5684
Map File No.	IDF

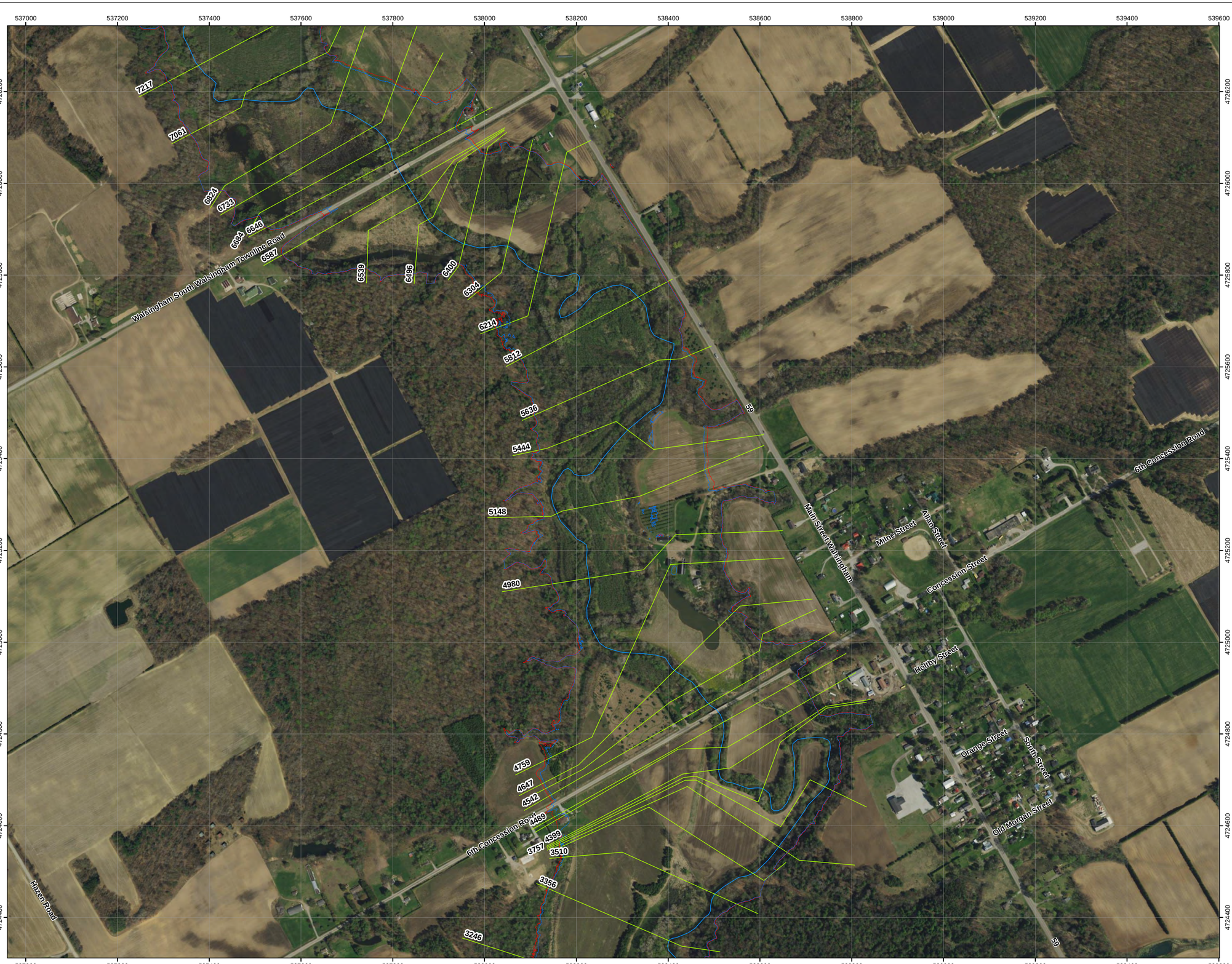
Deer Creek Dam Safety Review

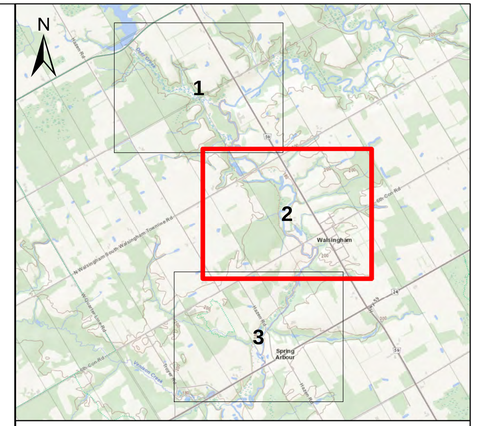
Long Point Region Conservation Authority

IDF (1/3 Between 1000-Year and PMF) Dam Break Inundation Map 3




N
1
2
3
Waterdown
Stoney Creek
Burlington
Hamilton
Dundas
Ancaster
Brantford
Cambridge
Kitchener
Waterloo
Guelph
Milton
Oakville
Mississauga
Toronto
Oshawa
Whitby
Ajax
Pickering
Markham
Vaughan
Richmond Hill
Aurora
Newmarket
Barrie
Orillia
Bradford
Alliston
Collingwood
Wasaga Beach
Midland
Penetanguishene
Tay
Port Severn
Georgina
Uxbridge
Stouffville
Whitchurch-Stouffville
Caledon
Brampton
Orangeville
Shelburne
Mono
Erin
Halton Hills
Acton
Georgetown
Milton
Oakville
Mississauga
Brampton
Caledon





Legend

- Ontario Roads
- PMF
- PMF Dam Break
- Cross Sections
- Watercourse

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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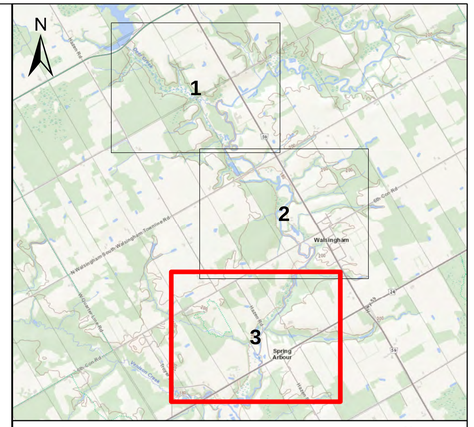
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Engineer: MC	
Designed By: MC	
Checked By: DG	
Date: 02/27/2024	
Project No. 24-5684	
Map File No. PMF	

Deer Creek Dam Safety Review
Long Point Region Conservation Authority

Probable Maximum Flood Dam Break
Inundation Map 2





Legend

- Ontario Roads
- PMF
- PMF Dam Break
- Cross Sections
- Watercourse

NAD83 CSRS
UTM Zone 17 North

CGVD 2013

Scales:

ANSI D (22x34)
1:4,000

0 62.5 125 250 375 500
Meters
1 centimeter = 40 meters

Revisions

No.	Description	By	Date
0	Inundation Mapping	MC	02/27/2025
1			

Notes:

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Engineer: MC
Designed By: MC
Checked By: DG
Date: 02/27/2024
Project No. 24-5684
Map File No. PMF

Deer Creek Dam Safety Review
Long Point Region Conservation Authority

Probable Maximum Flood Dam Break Inundation Map 3

Incremental Property Damages for Sunny Day Event

Project No: 24-5684
Project Name: Deer Creek DSR
Date: 03/03/25

Structure ID	Structure	Area (m^2)	No Dam Breach			Dam Breach			Incremental Damages ($)
			Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	
943 Walsingham Townline Road	Class B - Residential One Storey	263.69	0.00	$ -	$ -	0.00	$ -	$ -	$ -
866 6th Concession Road	Class B - Residential One Storey	285.13	0.00	$ -	$ -	0.00	$ -	$ -	$ -
905 6th Concession Road	Residential Auxiliary Structure	68.64	0.00	$ -	$ -	0.00	$ -	$ -	$ -
905 6th Concession Road	Class B - Residential One Storey	263.51	0.00	$ -	$ -	0.00	$ -	$ -	$ -
730 Norfolk County Road 60	Class B - Residential One Storey	107.42	0.00	$ -	$ -	0.00	$ -	$ -	$ -
730 Norfolk County Road 60	Residential Auxiliary Structure	74.37	0.00	$ -	$ -	0.00	$ -	$ -	$ -
902 6th Concession Road	Industrial / Warehouse	862.18	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Class B - Residential One Storey	158.75	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Industrial / Warehouse	345.44	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Residential Auxiliary Structure	106.22	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Residential Auxiliary Structure	74.57	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Industrial / Warehouse	237.08	0.00	$ -	$ -	0.00	$ -	$ -	$ -
Totals					**$ -**			**$ -**	**$ -**

Incremental Property Damages for 25-Year Event

Project No: 24-5684
Project Name: Deer Creek DSR
Date: 03/03/25

Structure ID	Structure	Area (m^2)	No Dam Breach			Dam Breach			Incremental Damages ($)
			Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	
943 Walsingham Townline Road	Class B - Residential One Storey	263.69	0.00	$ -	$ -	0.00	$ -	$ -	$ -
866 6th Concession Road	Class B - Residential One Storey	285.13	0.00	$ -	$ -	0.00	$ -	$ -	$ -
905 6th Concession Road	Residential Auxiliary Structure	68.64	0.00	$ -	$ -	0.00	$ -	$ -	$ -
905 6th Concession Road	Class B - Residential One Storey	263.51	0.00	$ -	$ -	0.00	$ -	$ -	$ -
730 Norfolk County Road 60	Class B - Residential One Storey	107.42	0.00	$ -	$ -	0.00	$ -	$ -	$ -
730 Norfolk County Road 60	Residential Auxiliary Structure	74.37	0.00	$ -	$ -	0.00	$ -	$ -	$ -
902 6th Concession Road	Industrial / Warehouse	862.18	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Class B - Residential One Storey	158.75	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Industrial / Warehouse	345.44	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Residential Auxiliary Structure	106.22	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Residential Auxiliary Structure	74.57	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Industrial / Warehouse	237.08	0.00	$ -	$ -	0.00	$ -	$ -	$ -
Totals					**$ -**			**$ -**	**$ -**

Incremental Property Damages for 100-Year Event

Project No: 24-5684
Project Name: Deer Creek DSR
Date: 03/03/25

Structure ID	Structure	Area (m^2)	No Dam Breach			Dam Breach			Incremental Damages ($)
			Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	
943 Walsingham Townline Road	Class B - Residential One Storey	263.69	0.00	$ -	$ -	0.00	$ -	$ -	$ -
866 6th Concession Road	Class B - Residential One Storey	285.13	0.00	$ -	$ -	0.00	$ -	$ -	$ -
905 6th Concession Road	Residential Auxiliary Structure	68.64	0.00	$ -	$ -	0.00	$ -	$ -	$ -
905 6th Concession Road	Class B - Residential One Storey	263.51	0.48	$ 1,726	$ 454,873	0.96	$ 1,947	$ 513,161	$ 58,289
730 Norfolk County Road 60	Class B - Residential One Storey	107.42	0.00	$ -	$ -	0.00	$ -	$ -	$ -
730 Norfolk County Road 60	Residential Auxiliary Structure	74.37	0.00	$ -	$ -	0.00	$ -	$ -	$ -
902 6th Concession Road	Industrial / Warehouse	862.18	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Class B - Residential One Storey	158.75	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Industrial / Warehouse	345.44	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Residential Auxiliary Structure	106.22	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Residential Auxiliary Structure	74.57	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Industrial / Warehouse	237.08	0.00	$ -	$ -	0.00	$ -	$ -	$ -
Totals					**$ 454,872.69**			**$ 513,161.32**	**$ 58,288.63**

Incremental Property Damages for 1000-Year Event

Project No: 24-5684
Project Name: Deer Creek DSR
Date: 03/03/25

Structure ID	Structure	Area (m^2)	No Dam Breach			Dam Breach			Incremental Damages ($)
			Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	
943 Walsingham Townline Road	Class B - Residential One Storey	263.69	0.00	$ -	$ -	0.00	$ -	$ -	$ -
866 6th Concession Road	Class B - Residential One Storey	285.13	0.00	$ -	$ -	0.00	$ -	$ -	$ -
905 6th Concession Road	Residential Auxiliary Structure	68.64	0.00	$ -	$ -	0.00	$ -	$ -	$ -
905 6th Concession Road	Class B - Residential One Storey	263.51	0.00	$ -	$ -	0.00	$ -	$ -	$ -
730 Norfolk County Road 60	Class B - Residential One Storey	107.42	0.00	$ -	$ -	0.00	$ -	$ -	$ -
730 Norfolk County Road 60	Residential Auxiliary Structure	74.37	0.00	$ -	$ -	0.00	$ -	$ -	$ -
902 6th Concession Road	Industrial / Warehouse	862.18	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Class B - Residential One Storey	158.75	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Industrial / Warehouse	345.44	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Residential Auxiliary Structure	106.22	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Residential Auxiliary Structure	74.57	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Industrial / Warehouse	237.08	0.00	$ -	$ -	0.00	$ -	$ -	$ -
Totals					**$ -**			**$ -**	**$ -**

Incremental Property Damages for 1/3 Between Event

Project No: 24-5684
Project Name: Deer Creek DSR
Date: 03/03/25

	Structure	Area (m^2)	No Dam Breach			Dam Breach			Incremental Damages ($)
			Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	
943 Walsingham Townline Road	Class B - Residential One Storey	263.69	0.00	$ -	$ -	0.00	$ -	$ -	$ -
866 6th Concession Road	Class B - Residential One Storey	285.13	0.00	$ -	$ -	0.00	$ -	$ -	$ -
905 6th Concession Road	Residential Auxiliary Structure	68.64	0.00	$ -	$ -	0.25	$ 125	$ 8,580	$ 8,580
905 6th Concession Road	Class B - Residential One Storey	263.51	1.62	$ 1,949	$ 513,688	2.09	$ 1,950	$ 513,846	$ 158
730 Norfolk County Road 60	Class B - Residential One Storey	107.42	0.00	$ -	$ -	0.00	$ -	$ -	$ -
730 Norfolk County Road 60	Residential Auxiliary Structure	74.37	0.00	$ -	$ -	0.00	$ -	$ -	$ -
902 6th Concession Road	Industrial / Warehouse	862.18	0.47	$ 570	$ 491,099	0.97	$ 1,068	$ 920,667	$ 429,568
2321 Highway 59	Class B - Residential One Storey	158.75	0.60	$ 1,765	$ 280,197	1.10	$ 1,948	$ 309,302	$ 29,105
2321 Highway 59	Industrial / Warehouse	345.44	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Residential Auxiliary Structure	106.22	0.00	$ -	$ -	0.00	$ -	$ -	$ -
2321 Highway 59	Residential Auxiliary Structure	74.57	0.00	$ -	$ -	0.12	$ 60	$ 4,474	$ 4,474
2321 Highway 59	Industrial / Warehouse	237.08	0.00	$ -	$ -	0.00	$ -	$ -	$ -
Totals					**$ 1,284,984.49**			**$ 1,756,869.40**	**$ 471,884.91**

Incremental Property Damages for PMF Event

Project No: 24-5684
Project Name: Deer Creek DSR
Date: 03/03/25

Address	Structure	Area (m^2)	No Dam Breach			Dam Breach			Incremental Damages ($)
			Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	Depth (m)	Estimated Damages ($/$m^2$)	Estimated Damages ($)	
943 Walsingham Townline Road	Class B - Residential One Storey	263.69	0.00	$ -	$ -	0.22	$ 1,600	$ 421,904	$ 421,904
866 6th Concession Road	Class B - Residential One Storey	285.127	0.05	$ 1,188	$ 338,588	0.45	$ 1,717	$ 489,420	$ 150,832
905 6th Concession Road	Residential Auxiliary Structure	68.643	1.33	$ 467	$ 32,022	1.72	$ 486	$ 33,360	$ 1,339
905 6th Concession Road	Class B - Residential One Storey	263.511	3.18	$ 1,950	$ 513,846	3.57	$ 1,950	$ 513,846	$ -
730 Norfolk County Road 60	Class B - Residential One Storey	107.424	0.87	$ 1,052	$ 112,989	1.25	$ 1,072	$ 115,159	$ 2,170
730 Norfolk County Road 60	Residential Auxiliary Structure	74.368	0.45	$ 225	$ 16,733	0.84	$ 386	$ 28,706	$ 11,973
902 6th Concession Road	Industrial / Warehouse	862.182	2.32	$ 1,352	$ 1,165,306	2.71	$ 1,396	$ 1,203,692	$ 38,386
2321 Highway 59	Class B - Residential One Storey	158.752	2.20	$ 1,950	$ 309,566	2.60	$ 1,950	$ 309,566	$ -
2321 Highway 59	Industrial / Warehouse	345.44	0.80	$ 909	$ 313,890	1.19	$ 1,174	$ 405,604	$ 91,714
2321 Highway 59	Residential Auxiliary Structure	106.215	0.69	$ 326	$ 34,626	1.08	$ 454	$ 48,222	$ 13,596
2321 Highway 59	Residential Auxiliary Structure	74.568	1.22	$ 461	$ 34,376	1.62	$ 481	$ 35,867	$ 1,491
2321 Highway 59	Industrial / Warehouse	237.076	0.53	$ 610	$ 144,711	0.91	$ 1,039	$ 246,282	$ 101,571
Totals					**$ 3,016,653.46**			**$ 3,851,630.11**	**$ 834,976.65**

Canadian Highway Bridge Design Code Wind Data for Deer Creek Freeboard Assessment

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: February 18, 2025

Canadian Highway Bridge Design Code (CSA Group, 2016) Wind Data

Wind Data Location: Tillsonburg

Parameter	Mean Hourly Wind Pressure			
	10-Year	25-Year	50-Year	100-Year
Pressure (Pa)	310	380	435	500
Velocity (m/s)	22.7	25.2	26.9	28.9
Velocity (km/h)	81.8	90.6	96.9	103.9
Velocity (knots)	44.2	48.9	52.3	56.1

Pressure = 0.5 * Air Density * Velocity^2 Equation Source
Velocity = sqrt(Pressure * 2 * 1/Air Density)

Air Density = 1.2 kg/m^3

Regression Calculations

Model Based on Power Function

Return Period	Excedence Probability (P)	Velocity (V) (km/h)	Log P	Log V
10	0.10	81.8	-1.0000	1.9129
25	0.04	90.6	-1.3979	1.9571
50	0.02	96.9	-1.6990	1.9865
100	0.01	103.9	-2.0000	2.0167

Paste Regression Analysis Results Below (Cell G34)

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.999526609
R Square	0.999053443
Adjusted R Squ	0.998580164
Standard Error	0.001666385
Observations	4

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	0.005861686	0.005861686	2110.919798	0.000473391
Residual	2	5.55368E-06	2.77684E-06		
Total	3	0.00586724			

		Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
log (k) >>	Intercept	1.81061156	0.00353187	512.6495473	3.80501E-06	1.795415149	1.82580797	1.795415149	1.82580797
m >>	X Variable 1	-0.103456265	0.002251754	-45.94474723	0.000473391	-0.113144781	-0.09376775	-0.113144781	-0.09376775

RESIDUAL OUTPUT

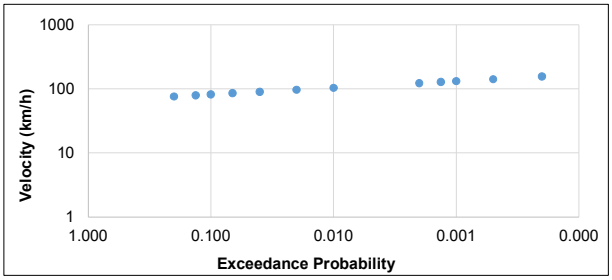
Observation	*Predicted Y*	*Residuals*
1	1.914067825	-0.001160102
2	1.955237212	0.001881462
3	1.986380651	9.0853E-05
4	2.01752409	-0.000812212

Parameters

V = k P^m or log V = m log P + log k			CHECK
log (k)=	1.811	OK	1.809
k=	64.656		
m =	-0.103	OK	-0.104

Regression Results

Return Period	Excedence Probability	Velocity (km/h)	Velocity (knots)
5	0.200	76.4	42.4
8	0.133	79.6	44.2
10	0.100	82.0	45.6
15	0.067	85.6	47.5
25	0.040	90.2	50.1
50	0.020	96.9	53.8
100	**0.010**	**104.1**	**57.8**
500	0.002	123.0	68.3
750	0.001	128.2	71.2
1,000	**0.001**	**132.1**	**73.4**
2,000	0.001	141.9	78.9
5,000	0.000	156.1	86.7



Freeboard Analysis for Deer Creek Dam

Project No: 5684
Project Name: Deer Creek DSR
Designed/Checked By: MC/DG
Date: 18-Feb-25

Wind Speeds

Direction	Fetch Length (km)	1:1000 AEP km/hr	1:100 AEP km/hr
N	0.73	132.1	103.9
NE	Behind Dam	0.0	0.0
E	Behind Dam	0.0	0.0
SE	Behind Dam	0.0	0.0
S	Behind Dam	0.0	0.0
SW	Behind Dam	0.0	0.0
W	Behind Dam	0.0	0.0
NW	Behind Dam	0.0	0.0

Dam Characteristics

Hazard Classification	High
Crest Elevation (m)	203.83
Impervious Core Elevation (m)	203.16
Average Reservoir Depth (m)	9.5
Embankment Slope (H:V)	3:1
Embankment Material	1 layer of rock

	Normal Operating Level	IDF Level
Water Level (m)	200.27	201.21
Available Freeboard (m)	3.56	2.62
Available Freeboard (Top of Impervious Core, m)	2.89	1.95

Freeboard Analysis

	Normal Operating Level	IDF Level
Critical Wind Annual Exceedence Probability	1:1000	1:100
Critical Wind Direction	N	N
Wind Speed (km/hr)	132.1	103.9
Effective Fetch (km)	0.73	0.73
Significant Wave Height (m)	0.82	0.61
Effective Fetch for Wind Setup (km)	0.73	0.73
Wave Runup (5% Exceedence, m)	0.84	0.62
Wind Setup (m)	0.02	0.013
Required Freeboard (m)	0.86	0.63
Required Freeboard (Top of Impervious Core)	0.02	0.01
Adequate Freeboard	***Yes***	***Yes***

Notes:

1. Wind speeds were multiplied by a factor of 1.2 to convert overland wind speeds into overwater wind speeds following the recommendations in USACE (1984) for fetch conditions <16km.
2. Wind data taken from CHBC 2016 for Tillsonburg .
3. The analysis was checked to be Fetch-Limited for the Normal Operating analysis, and Fetch-Limited for the IDF analysis.
4. Wave Runup Caluclations use methods from USACE.

Appendix D

Structural Stability Assessment



Typical Chute Wall and Slab Section - Loading Condition: Usual and Unusual Contitions

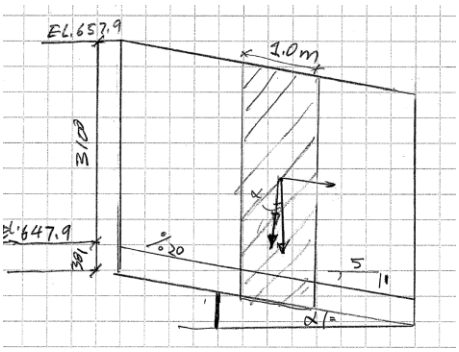
Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/05/25

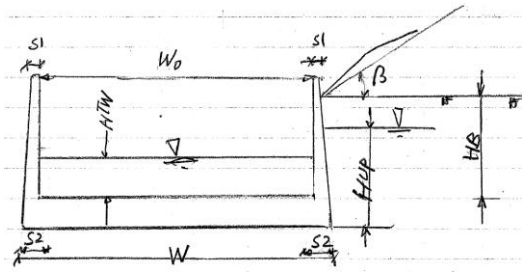
The Chute Slab consists of cantilever retaining wall and slab, shown below, is backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 30°. Assuming that the allowable bearing pressure of the soil is 120 kNm−2, the coefficient of friction is 0.4 and the unit weight of reinforced concrete is 24 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.
3.Due to existance of drainage network system, no uplift is considered for stability analysis.

Chute Slab Walls

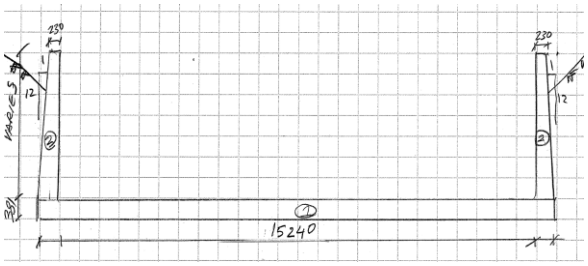
Upstream slab	:rise	1.00		0.2
Upstream slab	:run	5.00		0.2
		11.31		0.92342
Downstream: Incline angle	α =	11.31	Degree	
Earthfill slope at top of the wall	β=	42.72	Degree	
Slab Thickness	T1	380.00	mm	
Slab Thickness	T2	0.00	mm	
Stilling Basin Inside Length	W_0	15240.00	mm	
Stilling Basin Total Length	W	16540.00	mm	= j / tan (a)
Wall Thickess (Top)	s1	230.00	mm	= measured
Wall thickness(Bot)	s2	650.00	mm	
	s3	0.00	mm	
	s4	0.00	mm	
Wall height- U/S	h1	3180.00	mm	= measured
Slab thickness	h2	0.00		= measured
Shear key height	h3	0.00		= measured
Distance from wall Joint to Baffles	L1	1000.00	mm	= measured
Length of baffles	L2	0.00		= measured
Distance from Baffles to End Sills	L3	1000.00		= measured
Distance from End Sills to bottom of toewall	L4	0.00		
	B	1000.00	mm	





Elevations and Depths

Top of stem - Elevation	E1	623.00	ft	
Top of Sill elevation	E2	612.50	ft	
Top of Sill elevation	E3	612.50	ft	
Top of Sill elevation	E4	612.50	ft	
Top of stem to top of water inside the chute	U_W	2000.00	mm	
Top of water elevation inside the chute	E5	VAR	ft	
Tail Water height above top of the sill	HTW1	300.00	mm	
Top of stem to top of water Table @point 1	$U_{WT}1$	3200.40	mm	= E1-E6
Top of water Table elevation @point 1	E6	612.50	ft	
Water Table height above top of the sill @ Point 1	HUP1	0.00	mm	= E6-E2+T2
Top of stem to top of water Table @point 2	$U_{WT}2$	3200.40	mm	= E1-E7
Top of water Table elevation @point 2	E7	612.50	ft	
Water Table height above top of the sill @ Point 2	HUP2	0.00	mm	= E7-E3+T1
Top of stem to top of water Table @point 3	U_{WT3}	0.00	mm	= E1-E8
Top of water Table elevation @point 3	E8	612.50	ft	
Water Table height above top of the sill @ Point 2	HUP3	0.00	mm	= E8-E4+T1
Top of stem to soil - @ point 1	U_S1	914.40	mm	= E1-E9
Top of soil elevation - @ point 1	E9	620.00	ft	
Eartfill Height above top of sill @ point 1	HB1	2286.00	mm	= E9-E2+T2
Top of stem to soil - @ point 2	U_S2	914.40	mm	= E1-E9
Top of soil elevation - @ point 2	E10	620.00	ft	
Eartfill Height above top of sill @ point 2	HB2	2286.00	mm	= E10-E3+T1



Typical Chute Wall and Slab Section - Loading Condition: Usual and Unusual Contitions

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/05/25

Material Properties

Concrete unit weight	γ_c	23.5	kN/m3	
Water unit weight	γ_w	9.81	kN/m3	
Saturated unit weight of soil	γ_{sat}	20	kN/m3	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff
Silt layer Cohession	c	10.00	kPa	

Pressures and Forces

width of stilling basin section	W´	16.54	m	
width of stilling basin section	W_0	15.24		
Water pressure on the wall @Point 1	TP	0.000	kPa	= gw HUP1
Water Table Force on the wall per unit width @Point 1	TF´	0.000	kN/m	= 1/2 gw HUP1'2
Water Table force on the Wall @Point 1	TF	0.000	kN	= TF' L2'
Water Table Force on the Wall arm from toe	TF_a	0.000	m	= HUP1'/3
Silt pressure on the Wall	SP	15.240	kPa	= gsub HB1 Ka
Silt Force on the Wall per unit width	SF'	17.419	kN/m	= 1/2 gsub HB1^2 Ka
Silt force on the wall	SF	0.000	kN	= SF' L'
Silt force on the wall arm from toe	SF_a	0.762	m	= HB1/3
Downstream uplift pressure	U1	0.000	kPa	= gw* HUP1
Downstream uplift pressure per unit width	U1	0.000	kN/m	= U1 W'
Downstream uplift pressure at chute	U2	0.000	kPa	= gw* HUP2
Downstream uplift pressure at chute per unit width	U2	0.000	kN/m	= U2 W'
Uplift force	UF	0.000	kN	= ((Uu1 + Uu2)/2) * B
Uplift force arm from toe	UF_a	2.600	m	= Centroid of trapezoid
Downstream uplift pressure at chute	U3	0.000	kPa	= gw* HUP3
Downstream uplift pressure at chute per unit width	U3	0.000	kN/m	= U3 W'
Uplift force	UF	0.000	kN	= ((Uu2 + Uu3)/2) * B
Uplift force arm from toe	UF_a	7.700	m	= Centroid of trapezoid
Footing force per unit width	C1´	147.702	kN/m	= gc (T1.W)
Footing force	C1	147.702	kN	= C1' L3'
Footing force arm from toe	$C1_a$	0.500	m	= L4+L3/2
Wall Section Area	A1-3	1.399	m^2	
Wall Section Area	A1-2	0.000	m^2	
Wall force per unit width	C3´	16.441	kN/m	= gc (A1-3 + A1-2)/2
Wall force	C3	16	kN	= C3' L1'
Wall force arm from toe	C3a	0.50	m	= L1/2+L2

					(kN)	(m)	(kN-m)
Horizontal Forces							
	slab-1 load				-29.54	0.50	-14.77
	Wall Section Area at 1-1				-6.58	1.59	-10.46
			SHf=		0.00	Resisting forces	
			SHg=		-36.12	Sliding forces	
Vertical Forces							
	Uplift				0.00	2.60	0.00
	Uplift				0.00	7.70	0.00
C1	Footing force per unit width				147.70	0.50	73.85
C3	Wall -3 - Two walls				32.88	0.50	16.44
C6	Weight of Water force				0.00	0.00	0.00
			S Vi =		180.58	Downward forces	
			S Vh =		0.00	Uplift forces	

Typical Chute Wall and Slab Section - Loading Condition: Usual and Unusual Contitions

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/05/25

	ΣV	180.58	kN	S Vi + S Vh
	ΣH	36.12	kN	SHf + SHg
Overturning Check				
Restoring moments	$\Sigma M_{toe}R$	90.29	kNm	
Overturning moments	$\Sigma M_{toe}O$	-25.23	kNm	
Overturning Factor of Safety	O_{FOS}	3.58		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturing Factor of Safety	O_{allow}	1.50		
Efficiency	Efficiency	239%		
Sliding Check				
Sliding Factor of Safety	SFOS =	8.08		=(SV * m + C * A) / SH
Allowable Sliding Factor of Safety	Sallow =	1.5		
Efficiency	EFF	539%		
Foundation Pressure Check				
Total vertical loads	**Σ**V	180.58	kN	S V = S Vi + S Vh
Restoring Moment	M_R	90.29	kNm	
Overturning Moment	M_O	25.23	kNm	
Distance of resultant of all forces from toe	X´	0.36	m	=(MR - Mo) / S V
Total gravity wall base width	B	1.00	m	
	B/2	0.50	m	X' < B/2 resultant force is near the Toe
	B/6	0.17	m	
Eccentricity , Absolute (positive) value	e	0.14	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		
Standard Bearing Pressure				

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	1.77	kPa	(S V / B L) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	20.07	kPa	(S V / B) x (1 + 6e / B),

Typical Chute Wall and Slab Section - Loading Condition: Usual and Unusual Contitions

WILLS

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/05/25

Typical Chute Wall and Slab Section - Loading Condition: Extreme Seismic

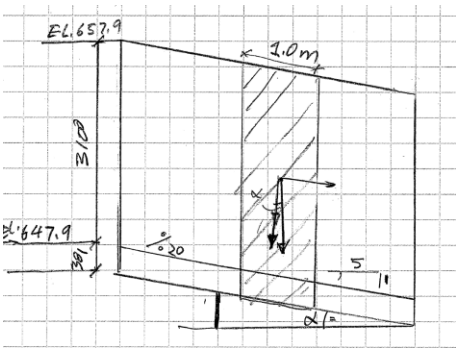
Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/05/25

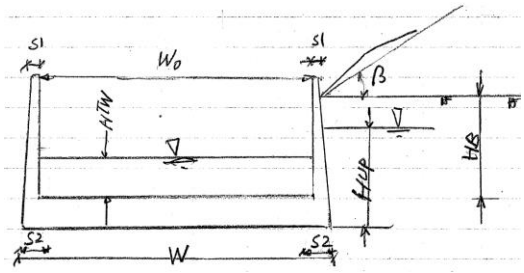
The Chute Slab consists of cantilever retaining wall and slab, shown below, is backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 30°. Assuming that the allowable bearing pressure of the soil is 120 kNm−2, the coefficient of friction is 0.4 and the unit weight of reinforced concrete is 24 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.
3.Due to existance of drainage network system, no uplift is considered for stability analysis.

Chute Slab Walls

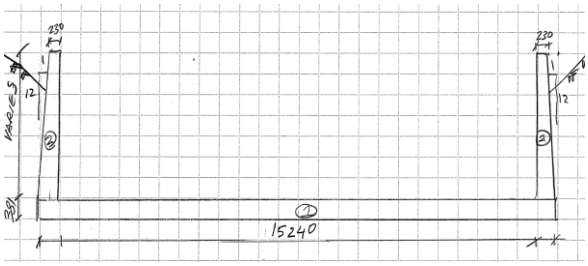
Upstream slab	:rise	1.00		0.2
Upstream slab	:run	5.00		0.2
		11.31		0.92342
Downstream: Incline angle	α =	11.31	Degree	
Earthfill slope at top of the wall	β=	42.72	Degree	
Slab Thickness	T1	380.00	mm	
Slab Thickness	T2	0.00	mm	
Stilling Basin Inside Length	W_0	15240.00	mm	
Stilling Basin Total Length	W	16540.00	mm	= j / tan (a)
Wall Thickess (Top)	s1	230.00	mm	= measured
Wall thickness(Bot)	s2	650.00	mm	
	s3	0.00	mm	
	s4	0.00	mm	
Wall height- U/S	h1	3180.00	mm	= measured
Slab thickness	h2	0.00		= measured
Shear key height	h3	0.00		= measured
Distance from wall Joint to Baffles	L1	1000.00	mm	= measured
Length of baffles	L2	0.00		= measured
Distance from Baffles to End Sills	L3	1000.00		= measured
Distance from End Sills to bottom of toewall	L4	0.00		
	B	1000.00	mm	





Elevations and Depths

Top of stem - Elevation	E1	623.00	ft	
Top of Sill elevation	E2	612.50	ft	
Top of Sill elevation	E3	612.50	ft	
Top of Sill elevation	E4	612.50	ft	
Top of stem to top of water inside the chute	U_W	2000.00	mm	
Top of water elevation inside the chute	E5	VAR	ft	
Tail Water height above top of the sill	HTW1	300.00	mm	
Top of stem to top of water Table @point 1	$U_{WT}1$	3200.40	mm	= E1-E6
Top of water Table elevation @point 1	E6	612.50	ft	
Water Table height above top of the sill @ Point 1	HUP1	0.00	mm	= E6-E2+T2
Top of stem to top of water Table @point 2	$U_{WT}2$	3200.40	mm	= E1-E7
Top of water Table elevation @point 2	E7	612.50	ft	
Water Table height above top of the sill @ Point 2	HUP2	0.00	mm	= E7-E3+T1
Top of stem to top of water Table @point 3	U_{WT3}	0.00	mm	= E1-E8
Top of water Table elevation @point 3	E8	612.50	ft	
Water Table height above top of the sill @ Point 2	HUP3	0.00	mm	= E8-E4+T1
Top of stem to soil - @ point 1	U_S1	914.40	mm	= E1-E9
Top of soil elevation - @ point 1	E9	620.00	ft	
Eartfill Height above top of sill @ point 1	HB1	2286.00	mm	= E9-E2+T2
Top of stem to soil - @ point 2	U_S2	914.40	mm	= E1-E9
Top of soil elevation - @ point 2	E10	620.00	ft	
Eartfill Height above top of sill @ point 2	HB2	2286.00	mm	= E10-E3+T1



Typical Chute Wall and Slab Section - Loading Condition: Extreme Seismic

Project No: 5684
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Material Properties

Concrete unit weight	γ_c	23.5	kN/m3	
Water unit weight	γ_w	9.81	kN/m3	
Saturated unit weight of soil	γ_{sat}	20	kN/m3	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff
Silt layer Cohession	c	0.00	kPa	

Pressures and Forces

width of stilling basin section	W´	16.54	m	
width of stilling basin section	W_0	15.24		
Water pressure on the wall @Point 1	TP	0.000	kPa	= gw HUP1
Water Table Force on the wall per unit width @Point 1	TF´	0.000	kN/m	= 1/2 gw HUP1'2
Water Table force on the Wall @Point 1	TF	0.000	kN	= TF' L2'
Water Table Force on the Wall arm from toe	TF_a	0.000	m	= HUP1'/3
Silt pressure on the Wall	SP	15.240	kPa	= gsub HB1 Ka
Silt Force on the Wall per unit width	SF'	17.419	kN/m	= 1/2 gsub HB1^2 Ka
Silt force on the wall	SF	0.000	kN	= SF' L'
Silt force on the wall arm from toe	SF_a	0.762	m	= HB1/3
Downstream uplift pressure	U1	0.000	kPa	= gw* HUP1
Downstream uplift pressure per unit width	U1	0.000	kN/m	= U1 W'
Downstream uplift pressure at chute	U2	0.000	kPa	= gw* HUP2
Downstream uplift pressure at chute per unit width	U2	0.000	kN/m	= U2 W'
Uplift force	UF	0.000	kN	= ((Uu1 + Uu2)/2) * B
Uplift force arm from toe	UF_a	2.600	m	= Centroid of trapezoid
Downstream uplift pressure at chute	U3	0.000	kPa	= gw* HUP3
Downstream uplift pressure at chute per unit width	U3	0.000	kN/m	= U3 W'
Uplift force	UF	0.000	kN	= ((Uu2 + Uu3)/2) * B
Uplift force arm from toe	UF_a	7.700	m	= Centroid of trapezoid
Footing force per unit width	C1´	147.702	kN/m	= gc (T1.W)
Footing force	C1	147.702	kN	= C1' L3'
Footing force arm from toe	$C1_a$	0.500	m	= L4+L3/2
Wall Section Area	A1-3	1.399	m^2	
Wall Section Area	A1-2	0.000	m^2	
Wall force per unit width	C3´	16.441	kN/m	= gc (A1-3 + A1-2)/2
Wall force	C3	16	kN	= C3' L1'
Wall force arm from toe	C3a	0.50	m	= L1/2+L2

Seismic Parameters

The DE is the 2500-year earthquake.(NBC 2020)

Probability of Exceedance (%) in 50 years	0	1%	2%	5%	10%	40%
1/Probability (Years)	10000	4975	2475	975	475	98
Probability of exceedance per annum	0.0001	0.0002	0.0004	0.001	0.0021	0.0102
Peak Ground Acceleration [PGA (g), 9.81 m/s2]			0.177	0.107	0.0662	0.0156

https://www.seismescanada.rncan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php?code=nbc2020&latitude=42.703&longitude=-80.563&siteDesignation=XS&siteDesignationXS=E

Seismic Parameters

Concrete unit weight	γ_c	2400	kg/m3	= 2400 kg/m3 = assumed for reinforced concrete
Concrete unit weight	γ_c	150	lb/ft3	= 149.8 lb/ft3 = assumed for reinforced concrete

Typical Chute Wall and Slab Section - Loading Condition: Extreme Seismic

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Concrete Horizontal Force	Cross Section Area (m2)	Volume (m3)	Weight (kg)	Weight (kip W	Seismic coefficient a	Horizontal Earthquake Force (kip) W.a	Earthquake Force (kN)	Arm from Toe m
C1x _ Wall			18058.34	39.81	0.12	4.70	20.90	2.00
C2x -Ogee and Foundation				0.00	0.12			
C3x				0.00	0.12			
			18058.34	**39.81**	**2/3 PGA**	**4.70**	**20.90**	

					(kN)	(m)	(kN-m)
Horizontal Forces							
	slab-1 load				-29.54	0.50	-14.77
	Wall Section Area at 1-1				-6.58	1.59	-10.46
	EQ				-20.90	2.00	-41.79
			SHf=		0.00	Resisting forces	
			SHg=		-57.01	Sliding forces	
Vertical Forces							
	Uplift				0.00	2.60	0.00
	Uplift				0.00	7.70	0.00
C1	Footing force per unit width				147.70	0.50	73.85
C3	Wall -3 - Two walls				32.88	0.50	16.44
C6	Weight of Water force				0.00	0.00	0.00
			S Vi =		180.58	Downward forces	
			S Vh =		0.00	Uplift forces	

	ΣV	180.58	kN	S Vi + S Vh
	ΣH	57.01	kN	SHf + SHg

Overturning Check

Restoring moments	**Σ**M_{toe}R	90.29	kNm	
Overturning moments	**Σ**M_{toe}O	-67.02	kNm	
Overturning Factor of Safety	O_{FOS}	1.35		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturning Factor of Safety	O_{allow}	1.50		
Efficiency	Efficiency	90%		

Sliding Check

Sliding Factor of Safety	SFOS =	2.22	=(SV * m + C * A) / SH
Allowable Sliding Factor of Safety	Sallow =	1.5	
Efficiency	EFF	148%	

Foundation Pressure Check

Total vertical loads	**Σ**V	180.58	kN	S V = S Vi + S Vh
Restoring Moment	M_R	90.29	kNm	
Overturning Moment	M_O	67.02	kNm	
Distance of resultant of all forces from toe	X´	0.13	m	=(MR - Mo) / S V
Total gravity wall base width	B	1.00	m	
	B/2	0.50	m	X' < B/2 resultant force is near the Toe
	B/6	0.17	m	
Eccentricity , Absolute (positive) value	e	0.37	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? No
	Therefore	not adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	-13.39	kPa	(S V / B L) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	35.23	kPa	(S V / B) x (1 + 6e / B),

Typical Chute Wall and Slab Section - Loading Condition: Extreme Seismic

WILLS

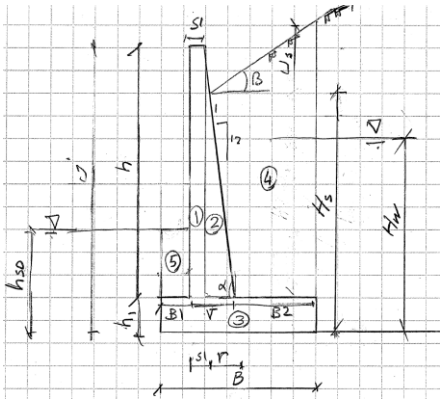
Project No: 5684
Project Name: LPRCA Deer Creek Dam
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Date: 03/05/25

Typical Retaining Wall Section - Loading Condition: Usual Summer

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/04/25

The cantilever retaining wall shown below is backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 30°. Assuming that the allowable bearing pressure of the soil is 120 kNm−2, the coefficient of friction is 0.4 and the unit weight of reinforced concrete is 24 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.

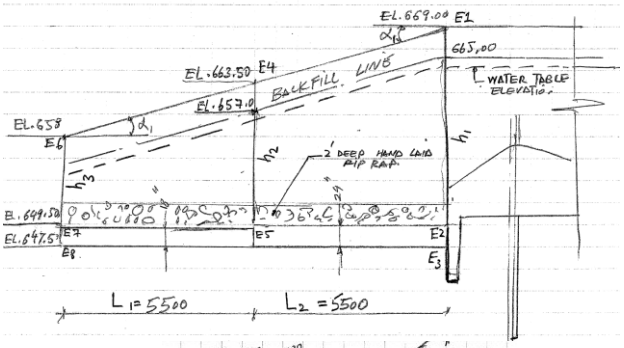


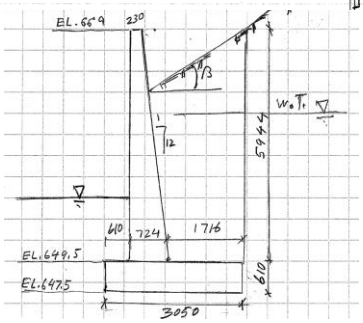
Wall Geometry

Downstream	:rise	12.00		12
Downstream	:run	1.00		
		85.24		
Downstream: Incline angle	α=	85.24	Degree	
Incline face distance: Downstream	r	495.33	mm	= h / tan (a)
Retaining wall height (below stem)	j	6554.00	mm	= h+h1
Stem wall base width	v	725.33	mm	= s1+r
Stem width(Top)	s1	230.00	mm	= measured
Retaining wall base width (Total)	B	3050.00	mm	= measured
Retaining wall base width (U/S)	B1	610.00	mm	= measured
Retaining wall base width (D/S)	B2	1714.67	mm	=B - v - B1
Stem height	h	5944.00	mm	= measured
Slab Thickness	T1	610.00	mm	= measured
Slab Thickness	T2	460.00	mm	

Elevations and Depths

Top of stem - Elevation	E1	669.00	ft	
Top of Sill elevation	E2	649.50	ft	
Bottom of Sill elevation	E3	647.50	ft	
Top of stem to top of water -	U_W	6553.20	mm	
Top of water elevation	VAR	647.50	ft	= E1 - Uw
Water height - Upstream	Hw	0.00	mm	= j + h - Uw
Top of stem to soil - Upstream	U_S	1524.00	mm	
Top of soil elevation - Upstream	E5	664.00	ft	= E1 - Us
Height of soil - Upstream	H_S1	5029.20	mm	= j + h - Us
Top of soil elevation - Upstream	E6	658.00	ft	= E1 - Us
Height of soil - Upstream	H_{S2}	3200.40	mm	= j + h - Us
Top of stem to top of water/silt - Downstream	D_W	4572.00	mm	
Top of water/silt elevation - Downstream	E6	654.00	ft	= E1 - Dw
Water/Silt height - Downstream	Hsd	1981.20	mm	
Top of stem to top of Riprap - Downstream	DR	5334.00	mm	
Top of Riprap elevation - Downstream	E7	651.50	ft	= E1 - Dw
Riprap height - Downstream	Hrd	609.60	mm	





Material Properties

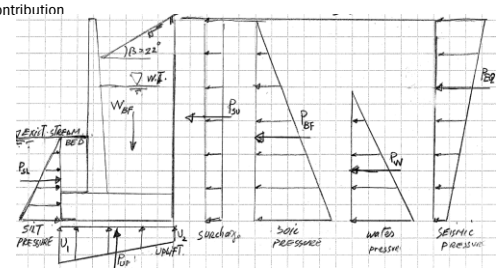
Concrete unit weight	γ_c	23.5	kN/m3	
Water unit weight	γ_w	9.81	kN/m3	
Saturated unit weight of soil	γ_{sat}	20	kN/m3	
Dry unit weight of soil	γ_{dry}	17	kN/m4	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Active earth pressure coefficient - Seismic	K_{AE}	0.57		
Maximum Seismic Soil Coefficient	k_T	0.63		K_{AE} (1-k_v)
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Soil friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff

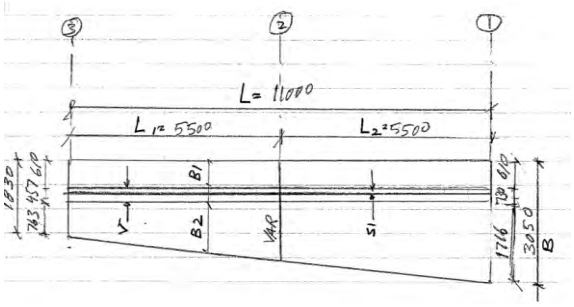
Pressures and Forces

Typical Retaining Wall Section - Loading Condition: Usual Summer

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Unit width of dam section	W´	5.5	m		= width with anchor contribution
Backfill pressure	HP1	28.50	kPa		= gw H
Backfill Force per unit width	HF'1	71.66	kN/m		= 1/2 gw H2
Backfill pressure	HP2	18.14	kPa		= gw H'
Backfill Force per unit width	HF'2	29.02	kN/m		= 1/2 gw H'2
Backfill force	HF	276.88	kN		= TF' W'
Backfill Force arm from toe	Hfa	1.676	m		= H'/3
Silt pressure - @ River side	SP	0.673	kPa		= gsub Hs Ka
Silt Force per unit width @ River side	SF'	6.666	kN/m		= 1/2 gsub Hs^2 Ka
Silt force @ River side	SF	36.664	kN		= SF' W'
Silt force arm from toe @ River side	SF_a	0.660	m		= Hs/3
Backfill uplift pressure	U_U	0.000	kPa		= HP
Backfill uplift pressure per unit width	$U_{U'}$	0.000	kN/m		= Uu W'
Riverbed uplift pressure	D_U	0.673	kPa		= TP
Riverbed uplift pressure per unit width	$D_{U'}$	3.701	kN/m		= Du W'
Uplift force	UF	5.644	kN		= ((Uu' + Du')/2) * B
Uplift force arm from toe	UF_a	1.016	m		= Centroid of trapezoid
Footing force per unit width	C3´	43.722	kN/m		= gc (b h)
Footing force	C3	240.470	kN		= C1' W'
Footing force arm from toe	$C3_a$	1.525	m		= B - b/2
stem rectangle section per unit width	C1´	32.127	kN/m		= gc (j b)
stem rectangle section force	C1	176.700	kN		= C2' W'
stem rectangle section arm from toe	$C1_a$	0.725	m		= B - b/2
stem triangle section per unit width	C2´	34.595	kN/m		= gc (j v) /2
stem triangle section force	C2´	190.273	kN		= C3' W'
Stem triangle section arm from toe	$C2_a$	1.005	m		= (2/3) v
Backfill side vertical soil pressure	QC4	101.05	kPa		= gsat (b h)
Vertical soil pressure per unit width	C4´	555.76	kN/m		= D' W'
Backfill side vertical soil weight	C4	952.95	kN		= D' W' B2
vertical soil force arm from toe	$C4_a$	2.19	m		= B2/2 +v+B1
River side vertical silt pressure	QC5	12.31	kPa		= gsat (b h)
River side vertical silt pressure per unit width	C5´	67.73	kN/m		= D' W'
River side vertical silt pressure weight	C5	41.32	kN		= D' W' B2
River side vertical silt force arm from toe	$C5_a$	0.31	m		= B2/2 +v+B1
Backfill side vertical Water pressure	QC5	0.00	kPa		= gsat (b h)
BackfilVertical Water pressure per unit width	C5´	0.00	kN/m		= D' W'
Backfill side vertical Water weight	C5	0.00	kN		= D' W' B2
BackfilVertical Water force arm from toe	$C5_a$	2.19	m		= B2/2 +v+B1





					Forces	Arm from t	M_{toe}
					(kN)	(m)	(kN-m)
Horizontal Forces							
	Hydrostatic load - Seimic Soil Pressure				-276.88	1.68	-464.16
	Concentrated load - Seismic on structure				-19.27	1.80	-34.68
	Surcharge Load				0.00	2.19	0.00
	Hydrostatic Pressure - Silt Load				36.66	0.66	24.21
			SHf=		36.66	Resisting forces	
			SHg=		-296.15	Sliding forces	
Vertical Forces							
	Uplift				-5.64	1.02	-5.73
	Rectangle Foundation section				240.47	1.53	366.72
	stem wall rectangle section				176.70	0.73	128.11
	stemy wall triangle section				190.27	1.01	191.25
	Soil Weight on Foundation- Backfill side				952.95	2.19	2089.50
	Silt Weight on Foundation- River side				41.32	0.31	12.60
			S Vi =		1601.71	Downward forces	
			S Vh =		-5.64	Uplift forces	

Typical Retaining Wall Section - Loading Condition: Usual Summer

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	ΣV	1596.07	kN	S Vi + S Vh
	ΣH	259.48	kN	SHf + SHg
Overturning Check				
Restoring moments	$\Sigma M_{toe}R$	2812.39	kNm	
Overturning moments	$\Sigma M_{toe}O$	-504.58	kNm	
Overturning Factor of Safety	O_{FOS}	5.57		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturning Factor of Safety	O_{allow}	1.10		
Efficiency	Efficiency	507%		
Sliding Check				
Sliding Factor of Safety	SFOS =	4.31		=SV * m / SH
Allowable Sliding Factor of Safety	Sallow =	1.1		
Efficiency	EFF	392%		
Foundation Pressure Check				
Total vertical loads	**Σ**V	1596.07	kN	S V = S Vi + S Vh
Restoring Moment	M_R	2812.39	kNm	
Overturning Moment	M_O	504.58	kNm	
Distance of resultant of all forces from toe	X´	1.45	m	=(MR - Mo) / S V
Total gravity wall base width	B	3.05	m	
	B/2	1.53	m	X' < B/2 resultant force is near the Toe
	B/6	0.51	m	
Eccentricity , Absolute (positive) value	e	0.08	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		
Standard Bearing Pressure				

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

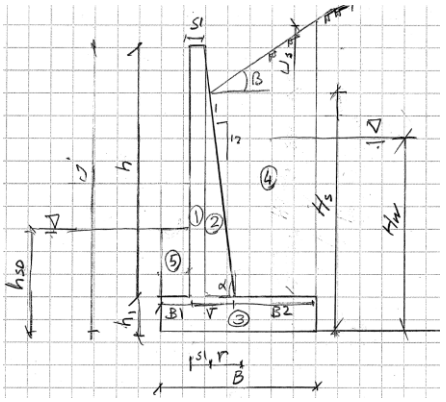
Maximum pressure at Toe	Q_t	80.35	kPa	(S V / B) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	109.94	kPa	(S V / B) x (1 + 6e / B),

Typical Retaining Wall Section - Loading Condition: Usual Water Table - Saturated Soil

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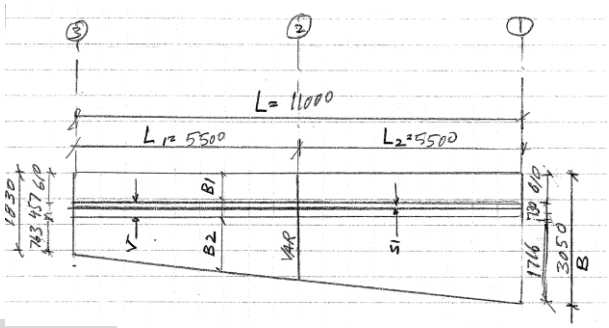
The cantilever retaining wall shown below is backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 30°. Assuming that the allowable bearing pressure of the soil is 120 kNm−2, the coefficient of friction is 0.4 and the unit weight of reinforced concrete is 24 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.



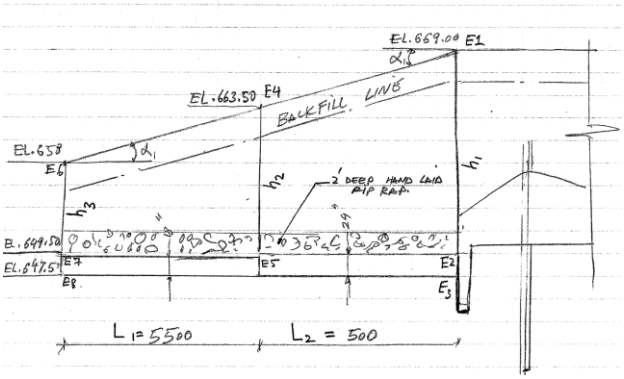
Wall Geometry

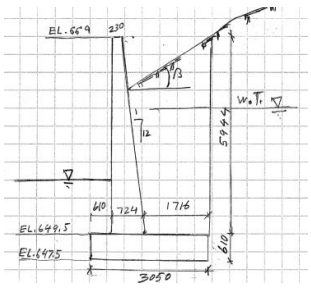
Downstream	:rise	12.00		12
Downstream	:run	1.00		
		85.24		
Downstream: Incline angle	α=	85.24	Degree	
Incline face distance: Downstream	r	495.33	mm	= h / tan (a)
Retaining wall height (below stem)	j	6554.00	mm	= h+h1
Stem wall base width	v	725.33	mm	= s1+r
Stem width(Top)	s1	230.00	mm	= measured
Retaining wall base width (Total)	B	3050.00	mm	= measured
Retaining wall base width (U/S)	B1	610.00	mm	= measured
Retaining wall base width (D/S)	B2	1714.67	mm	=B - v - B1
Stem height	h	5944.00	mm	= measured
Slab Thickness	T1	610.00	mm	= measured
Slab Thickness	T2	460.00	mm	



Elevations and Depths

Top of stem - Elevation	E1	669.00	ft	
Top of Sill elevation	E2	649.50	ft	
Bottom of Sill elevation	E3	647.50	ft	
Top of stem to top of water -	U_w	3352.80	mm	
Top of water elevation	VAR	658.00	ft	= E1 - Uw
Water height - Upstream	Hw	3200.40	mm	= j + h - Uw
Top of stem to soil - Upstream	U_S	609.60	mm	
Top of soil elevation - Upstream	E5	667.00	ft	= E1 - Us
Height of soil - Upstream	H_S	5943.60	mm	= j + h - Us
Top of stem to top of water/silt - Downstream	D_W	5791.20	mm	
Top of water/silt elevation - Downstream	E6	650.00	ft	= E1 - Dw
Water/Silt height - Downstream	Hsd	762.00	mm	
Top of stem to top of Riprap - Downstream	DR	5334.00	mm	
Top of Riprap elevation - Downstream	E7	651.50	ft	= E1 - Dw
Riprap height - Downstream	Hrd	609.60	mm	
	h_1'	2743.20		
	h_2'	3200.40		





Material Properties

Concrete unit weight	γ_c	23.5	kN/m3
Water unit weight	γ_w	9.81	kN/m3
Saturated unit weight of soil	γ_{sat}	20	kN/m3
Dry unit weight of soil	γ_{dry}	17	kN/m4

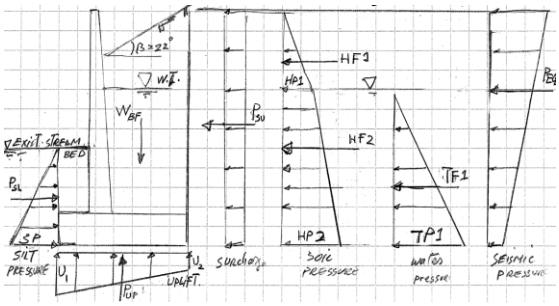
Typical Retaining Wall Section - Loading Condition: Usual Water Table - Saturated Soil

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Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φ s	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff

Pressures and Forces

Unit width of dam section	W´	1	m	= width with anchor contribution
Backfill pressure @ Water Table	HP1	15.54	kPa	= gdry Hs Ka
Backfill Force per unit width	HF'	12.78	kN/m	= 1/2 gdry Hs^2 Ka
Backfill force	HF1	12.78	kN	= HF' W'
Backfill Force arm from toe	Hfa	4.115	m	= H/3
Backfill pressure @ BOT of Footing	HP2	26.42	kPa	= gsub Hs Ka
Backfill Force @ BOT of Footing per unit width	HF'	67.14	kN/m	= 1/2 gsub Hs^2 Ka
Backfill force	HF2	67.14	kN	= HF' W'
Backfill Force arm from toe	Hfa	1.459	m	= H/3
Water Table pressure @ Backfill side	TP1	31.396	kPa	= gw H'
Water Table Force per unit width @ Backfill side	TF´	50.240	kN/m	= 1/2 gw H'2
Water Table force @ Backfill side	TF 1	0.000	kN	= TF' W'
Water Table Force arm from toe @ Backfill side	TF_a	1.067	m	= H'/3
Silt pressure - @ River side	SP	2.588	kPa	= gsub Hs Ka
Silt Force per unit width @ River side	SF'	0.986	kN/m	= 1/2 gsub Hs^2 Ka
Silt force @ River side	SF	0.986	kN	= SF' W'
Silt force arm from toe @ River side	SF_a	0.254	m	= Hs/3
Backfill uplift pressure	U_U	31.396	kPa	= HP
Backfill uplift pressure per unit width	U_U·	31.396	kN/m	= Uu W'
Riverbed uplift pressure	D_U	7.475	kPa	= TP
Riverbed uplift pressure per unit width	D_U·	7.475	kN/m	= Du W'
Uplift force	UF	59.278	kN	= ((Uu' + Du')/2) * B
Uplift force arm from toe	UF_a	1.016	m	= Centroid of trapezoid
Footing force per unit width	C3´	43.722	kN/m	= gc (b h)
Footing force	C3	43.722	kN	= C1' W'
Footing force arm from toe	$C3_a$	1.525	m	= B - b/2
stem rectangle section per unit width	C1´	32.127	kN/m	= gc (j b)
stem rectangle section force	C1	32.127	kN	= C2' W'
stem rectangle section arm from toe	$C1_a$	0.725	m	= B - b/2
stem triangle section per unit width	C2´	34.595	kN/m	= gc (j v) /2
stem triangle section force	C2´	34.595	kN	= C3' W'
Stem triangle section arm from toe	$C2_a$	1.005	m	= (2/3) v
Backfill side vertical soil pressure	QC4	101.05	kPa	= gsat (b h)
Vertical soil pressure per unit width	C4´	101.05	kN/m	= D' W'
Backfill side vertical soil weight	C4	173.26	kN	= D' W' B2
vertical soil force arm from toe	$C4_a$	2.19	m	= B2/2 +v+B1
River side vertical silt pressure	QC5	4.74	kPa	= gsat (b h)
River side vertical silt pressure per unit width	C5´	4.74	kN/m	= D' W'
River side vertical silt pressure weight	C5	2.89	kN	= D' W' B2
River side vertical silt force arm from toe	$C5_a$	0.31	m	= B2/2 +v+B1
Backfill side vertical Water pressure	QC5	31.40	kPa	= gsat (b h)
BackfilVertical Water pressure per unit width	C5´	31.40	kN/m	= D' W'
Backfill side vertical Water weight	C5	53.83	kN	= D' W' B2
BackfilVertical Water force arm from toe	$C5_a$	2.19	m	= B2/2 +v+B1



Typical Retaining Wall Section - Loading Condition: Usual Water Table - Saturated Soil

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					Forces	Arm from t	M_{toe}
					(kN)	(m)	(kN-m)
Horizontal Forces							
	Hydrostatic load - Soil Pressure				-12.78	4.11	-52.59
	Hydrostatic load - Soil Pressure				-67.14	1.46	-97.99
	Hydrostatic load - Water Table Pressure				0.00	1.07	0.00
	Hydrostatic Pressure - Silt Load				0.99	0.25	0.25
			SHf=		0.99	Resisting forces	
			SHg=		-79.93	Sliding forces	
Vertical Forces							
	Uplift				-59.28	1.02	-60.23
	Rectangle Foundation section				43.72	1.53	66.68
	stem wall rectangle section				32.13	0.73	23.29
	stemy wall triangle section				34.60	1.01	34.77
	Soil Weight on Foundation- Backfill side				173.26	2.19	379.91
	Silt Weight on Foundation- River side				2.89	0.31	0.88
	Water Weight on Foundation- Backfill side				53.83	2.19	118.04
			S Vi =		340.43	Downward forces	
			S Vh =		-59.28	Uplift forces	

	ΣV	281.15	kN	S Vi + S Vh
	ΣH	78.94	kN	SHf + SHg

Overturning Check

Restoring moments	**Σ**M_{toe}R	623.82	kNm	
Overturning moments	**Σ**M_{toe}O	-210.80	kNm	
Overturning Factor of Safety	O_{FOS}	2.96		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturning Factor of Safety	O_{allow}	1.50		
Efficiency	Efficiency	197%		

Sliding Check

Sliding Factor of Safety	SFOS =	2.49	=SV * m / SH
Allowable Sliding Factor of Safety	Sallow =	1.5	
Efficiency	EFF	166%	

Foundation Pressure Check

Total vertical loads	**Σ**V	281.15	kN	S V = S Vi + S Vh
Restoring Moment	M_R	623.82	kNm	
Overturning Moment	M_O	210.80	kNm	
Distance of resultant of all forces from toe	X´	1.47	m	=(MR - Mo) / S V
Total gravity wall base width	B	3.05	m	
	B/2	1.53	m	X' < B/2 resultant force is near the Toe
	B/6	0.51	m	
Eccentricity , Absolute (positive) value	e	0.06	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	82.03	(S V / B) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	102.33	(S V / B) x (1 + 6e / B),

Typical Retaining Wall Section - Loading Condition: Usual Water Table - Saturated Soil

WILLS

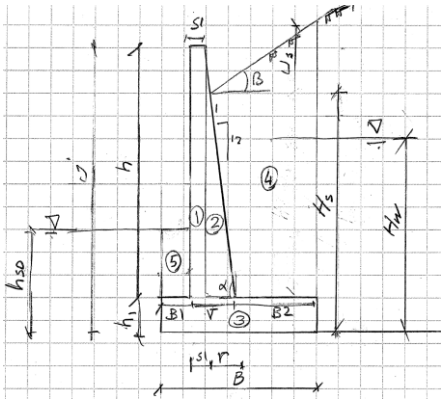
Project No: 5684
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Typical Retaining Wall Section - Loading Condition: Extreme Condition - EQ

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/04/25

The cantilever retaining wall shown below is backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 30°. Assuming that the allowable bearing pressure of the soil is 120 kNm−2, the coefficient of friction is 0.4 and the unit weight of reinforced concrete is 24 kNm−3

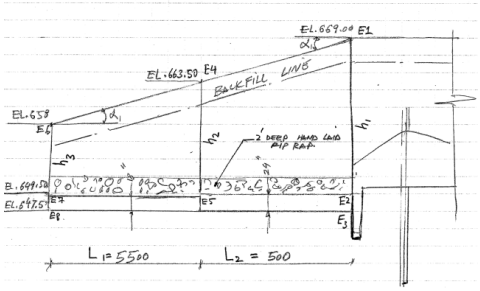
1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.



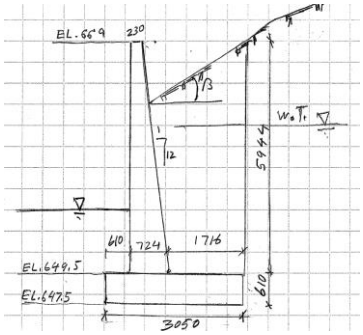
Wall Geometry

Downstream	:rise	12.00		12
Downstream	:run	1.00		
		85.24		
Downstream: Incline angle	α=	85.24	Degree	
Incline face distance: Downstream	r	495.33	mm	= h / tan (a)
Retaining wall height (below stem)	j	6554.00	mm	= h+h1
Stem wall base width	v	725.33	mm	= s1+r
Stem width(Top)	s1	230.00	mm	= measured
Retaining wall base width (Total)	B	3050.00	mm	= measured
Retaining wall base width (U/S)	B1	610.00	mm	= measured
Retaining wall base width (D/S)	B2	1714.67	mm	=B - v - B1
Stem height	h	5944.00	mm	= measured
Slab Thickness	T1	610.00	mm	= measured
Slab Thickness	T2	460.00	mm	

Elevations and Depths



Top of stem - Elevation	E1	669.00	ft	
Top of Sill elevation	E2	649.50	ft	
Bottom of Sill elevation	E3	647.50	ft	
Top of stem to top of water -	U_W	6553.20	mm	
Top of water elevation	VAR	647.50	ft	= E1 - Uw
Water height - Upstream	Hw	0.00	mm	= j + h - Uw
Top of stem to soil - Upstream	U_S	1524.00	mm	
Top of soil elevation - Upstream	E5	664.00	ft	= E1 - Us
Height of soil - Upstream	H_S1	5029.20	mm	= j + h - Us
Top of soil elevation - Upstream	E6	658.00	ft	= E1 - Us
Height of soil - Upstream	H_{S2}	3200.40	mm	= j + h - Us
Top of stem to top of water/silt - Downstream	D_W	4572.00	mm	
Top of water/silt elevation - Downstream	E6	654.00	ft	= E1 - Dw
Water/Silt height - Downstream	Hsd	0.00	mm	
Top of stem to top of Riprap - Downstream	DR	5334.00	mm	
Top of Riprap elevation - Downstream	E7	651.50	ft	= E1 - Dw
Riprap height - Downstream	Hrd	609.60	mm	



Material Properties

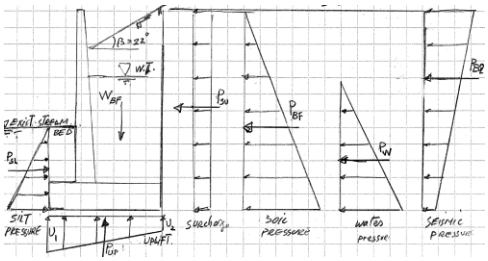
Concrete unit weight	γ_c	23.5	kN/m3	
Water unit weight	γ_w	9.81	kN/m3	
Saturated unit weight of soil	γ_{sat}	20	kN/m3	
Dry unit weight of soil	γ_{dry}	17	kN/m4	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Active earth pressure coefficient - Seismic	K_{AE}	0.57		
Maximum Seismic Soil Coefficient	k_T	0.63		K_{AE} (1-k_v)
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Soil friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff

Pressures and Forces

Typical Retaining Wall Section - Loading Condition: Extreme Condition - EQ

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Unit width of dam section	W´	5.5	m	= width with anchor contribution
Backfill pressure	HP1	53.69	kPa	= gw H
Backfill Force per unit width	HF'1	135.01	kN/m	= 1/2 gw H2
Backfill pressure	HP2	34.17	kPa	= gw H'
Backfill Force per unit width	HF'2	54.67	kN/m	= 1/2 gw H'2
Backfill force	HF	521.64	kN	= TF' W'
Backfill Force arm from toe	Hfa	2.091	m	= H'/3
Silt pressure - @ River side	SP	0.000	kPa	= gsub Hs Ka
Silt Force per unit width @ River side	SF'	0.000	kN/m	= 1/2 gsub Hs^2 Ka
Silt force @ River side	SF	0.000	kN	= SF' W'
Silt force arm from toe @ River side	SF_a	0.000	m	= Hs/3
Backfill uplift pressure	U_U	34.167	kPa	= HP
Backfill uplift pressure per unit width	$U_{U'}$	187.921	kN/m	= Uu W'
Riverbed uplift pressure	D_U	0.000	kPa	= TP
Riverbed uplift pressure per unit width	$D_{U'}$	0.000	kN/m	= Du W'
Uplift force	UF	286.580	kN	= ((Uu' + Du')/2) * B
Uplift force arm from toe	UF_a	1.016	m	= Centroid of trapezoid
Footing force per unit width	C3´	43.722	kN/m	= gc (b h)
Footing force	C3	240.470	kN	= C1' W'
Footing force arm from toe	$C3_a$	1.525	m	= B - b/2
stem rectangle section per unit width	C1´	32.127	kN/m	= gc (j b)
stem rectangle section force	C1	176.700	kN	= C2' W'
stem rectangle section arm from toe	$C1_a$	0.725	m	= B - b/2
stem triangle section per unit width	C2´	34.595	kN/m	= gc (j v) /2
stem triangle section force	C2´	190.273	kN	= C3' W'
Stem triangle section arm from toe	$C2_a$	1.005	m	= (2/3) v
Backfill side vertical soil pressure	QC4	101.05	kPa	= gsat (b h)
Vertical soil pressure per unit width	C4´	555.76	kN/m	= D' W'
Backfill side vertical soil weight	C4	952.95	kN	= D' W' B2
vertical soil force arm from toe	$C4_a$	2.19	m	= B2/2 +v+B1
River side vertical silt pressure	QC5	0.00	kPa	= gsat (b h)
River side vertical silt pressure per unit width	C5´	0.00	kN/m	= D' W'
River side vertical silt pressure weight	C5	0.00	kN	= D' W' B2
River side vertical silt force arm from toe	$C5_a$	0.31	m	= B2/2 +v+B1
Backfill side vertical Water pressure	QC5	0.00	kPa	= gsat (b h)
BackfilVertical Water pressure per unit width	C5´	0.00	kN/m	= D' W'
Backfill side vertical Water weight	C5	0.00	kN	= D' W' B2
BackfilVertical Water force arm from toe	$C5_a$	2.19	m	= B2/2 +v+B1



Seismic Parameters

The DE is the 2500-year earthquake.(NBC 2020)

Probability of Exceedance (%) in 50 years	0	1%	2%	5%	10%	40%
1/Probability (Years)	10000	4975	2475	975	475	98
Probability of exceedance per annum	0.0001	0.0002	0.0004	0.001	0.0021	0.0102
Peak Ground Acceleration [PGA (g), 9.81 m/s2]			0.177	0.0974	0.0604	0.0156

https://www.seismescanada.rncan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php?code=nbc2020&latitude=42.703&longitude=-80.563&siteDesignation=XS&siteDesignationXS=E

Seismic Parameters

Concrete unit weight	γ_c	2400	kg/m3	= 2400 kg/m3 = assumed for reinforced concrete
Concrete unit weight	γ_c	150	lb/ft3	= 149.8 lb/ft3 = assumed for reinforced concrete

Typical Retaining Wall Section - Loading Condition: Extreme Condition - EQ

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Concrete Horizontal Force	Cross Section Area (m2)	Volume (m3)	Weight (kg)/m	Weight (kip W/m	Seismic coefficient a	Horizontal Earthquake Force W.a	Horizontal Earthquake Force Feq/m	Arm from Toe m
C1x _ Wall			11100.00	24.47	0.18	4.33	19.27	1.80
				0.00	0.18			
				0.00	0.18			
			11100.00	**24.47**		**4.33**	**19.27**	

Soil Calss	E		
PGAD= 1.3 x PGA		0.2301	g
k_h = 2/3 PGAD		0.1534	g
k_v = 2/3 k_h		0.102267	g
h		5029.20	mm
Active earth pressure coefficient - upstream	K_{AE}	0.57	

The coefficient K_{ae} for active earth pressure is given by:

$$K_{ae} = \frac{\cos^2(\varphi - \psi - \alpha)}{\cos\psi\cos^2\alpha\cos(\psi+\alpha+\delta)\left(1+\sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi-\psi-\beta)}{\cos(\delta+\psi+\alpha)\cos(-\beta+\alpha)}}\right)^2}$$

Soil dynamic Pressure- Mononobe Okabe				
PGA for 2% , 2500 year probability	PGA	0.1770		
Saturated unit weight of soil	γ_{sat}	20		
Angle of internal friction of soil backfill	ϕ	34	degree	
Angle of internal friction between structute and soil	δ	0	degree	
Slope Inclination	β	20	degree	
Backface Inclination of the wall stem	α	5	degree	
Sesmic Inertia Angle	ψ	7.60	degree	
Seismic Coefficien for verical acceleration	k_v	0.102	g	
Seismic Coefficien for horizontal acceleration	k_h	0.153	g	
Height of soil at the all	h	5.029	m	
Maximum Seismic Soil Coefficient	k_T	0.63		$K_{AE}(1-k_v)$

					Forces (kN)	Arm from t (m)	M_{toe} (kN-m)
Horizontal Forces							
	Hydrostatic load - Seimic Soil Pressure				-521.64	2.09	-1090.89
	Concentrated load - Seismic on structure				-19.27	1.80	-34.68
	Surcharge Load				0.00	2.19	0.00
	Hydrostatic Pressure - Silt Load				0.00	0.00	0.00
			SHf=		0.00	Resisting forces	
			SHg=		-540.91	Sliding forces	
Vertical Forces							
	Uplift				-286.58	1.02	-291.16
	Rectangle Foundation section				240.47	1.53	366.72
	stem wall rectangle section				176.70	0.73	128.11
	stemy wall triangle section				190.27	1.01	191.25
	Soil Weight on Foundation- Backfill side				952.95	2.19	2089.50
	Silt Weight on Foundation- River side				0.00	0.31	0.00
			S Vi =		1560.39	Downward forces	
			S Vh =		-286.58	Uplift forces	

	ΣV	1273.81	kN	S Vi + S Vh
	ΣH	540.91	kN	SHf + SHg

Overturning Check

Restoring moments	$\Sigma M_{toe}R$	2775.57	kNm	
Overturning moments	$\Sigma M_{toe}O$	-1416.74	kNm	
Overturning Factor of Safety	O_{FOS}	1.96		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturing Factor of Safety	O_{allow}	1.10		
Efficiency	Efficiency	178%		

Sliding Check

Sliding Factor of Safety	SFOS =	1.65		=SV * m / SH
Allowable Sliding Factor of Safety	Sallow =	1.1		
Efficiency	EFF	150%		

Typical Retaining Wall Section - Loading Condition: Extreme Condition - EQ

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/04/25

Foundation Pressure Check

Total vertical loads	ΣV	1273.81	kN	S V = S Vi + S Vh
Restoring Moment	M_R	2775.57	kNm	
Overturning Moment	M_O	1416.74	kNm	
Distance of resultant of all forces from toe	X´	1.07	m	=(MR - Mo) / S V
Total gravity wall base width	B	3.05	m	
	B/2	1.53	m	X' < B/2 resultant force is near the Toe
	B/6	0.51	m	
Eccentricity , Absolute (positive) value	e	0.46	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	7.48	kPa	(S V / B) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	144.39	kPa	(S V / B) x (1 + 6e / B),

Typical Ogee and Wall Section - Loading Condition: Usual Summer

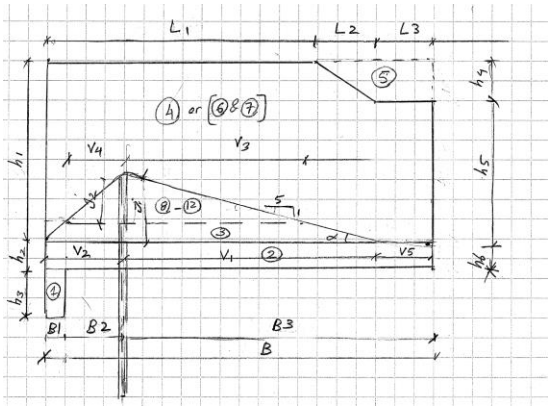
Project No: 5684
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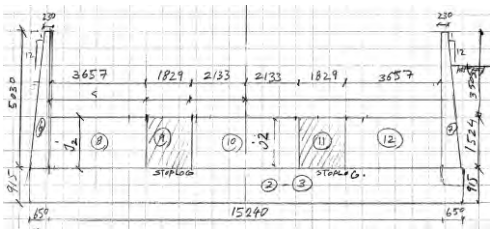
The Ogee section below is a controlled structure with wall backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 35°. Assuming that the allowable bearing pressure of the soil is 150 kNm−2, the coefficient of friction is 0.7 and the unit weight of reinforced concrete is 23.5 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.

Wall Geometry

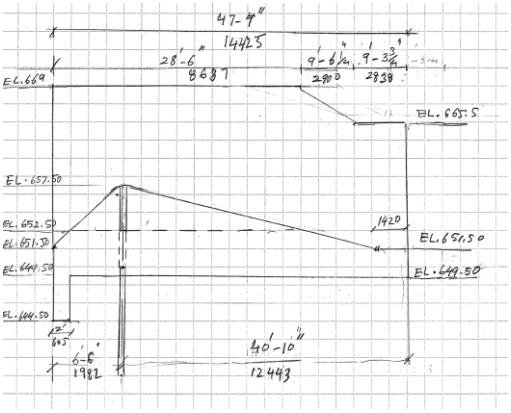
Downstream	:rise	1.00		0.2
Downstream	:run	5.00		0.2
		11.31		0.92342
Downstream: Incline angle	α1=	11.31	Degree	
	α2=	42.72	Degree	
ogee height at weir	J1	1830.00	mm	
ogee height at stoplogs	J2	1524.00	mm	
	v1	9150.00	mm	
Stem wall base width	v2	1981.76	mm	
	v3	7620.00	mm	
	v4	1650.39	mm	
	v5	3293.24	mm	
Stem width(Top)	s1	230.00	mm	= measured
Stem width(Top)	s2	660.00	mm	
shear key base width (Total)	B1	610.00	mm	= measured
base width (U/S)	B2	1372.00	mm	= measured
base width (D/S)	B3	12443.00	mm	= measured
Totam Base width	B	14425.00	mm	
Wall height- U/S	h1	5334.00	mm	= measured
Slab thickness	h2	610.00	mm	= measured
Shear key height	h3	1524.00	mm	= measured
top wall drop height	h4	1067.00	mm	= measured
Wall height- D/S	h5	4267.00	mm	= measured
Slab thickness	h6	610.00	mm	= measured
Wall Top Length	L1	8687.00	mm	= measured
Wall Top Length	L2	2900.00	mm	= measured
Wall Top Length	L3	2838.00	mm	= measured
	L	14425.00	mm	





Elevations and Depths

Top of stem - Elevation	E1	669.00	ft	
Bottom of Sherar Key - Elevation	E2	644.50	ft	= E1 - j
Top of Sill elevation	E3	651.50	ft	
Top of stem to top of water - Upstream	U_w	3657.60	mm	
Top of water elevation - Upstream	E4	657.00	ft	= E1 - Uw
Water height - Upstream	Hw	3810.00	mm	= j + h - Uw
Top of stem to soil - Upstream	U_s	5029.20	mm	
Top of soil elevation - Upstream	E5	652.50	ft	= E1 - Us
Height of soil - Upstream	H_s	2438.40	mm	= j + h - Us
Top of stem to top of water - Downstream	D_w	5334.00	mm	
Top of water elevation - Downstream	E6	651.50	ft	= E1 - Dw
Water height - Downstream	Hsd	0.00	mm	



Material Properties

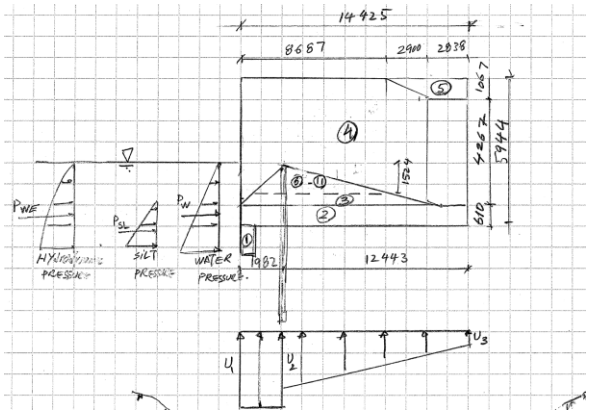
Concrete unit weight	γ_c	23.5	kN/m3
Water unit weight	γ_w	9.81	kN/m3
Saturated unit weight of soil	γ_{sat}	20	kN/m3

Typical Ogee and Wall Section - Loading Condition: Usual Summer

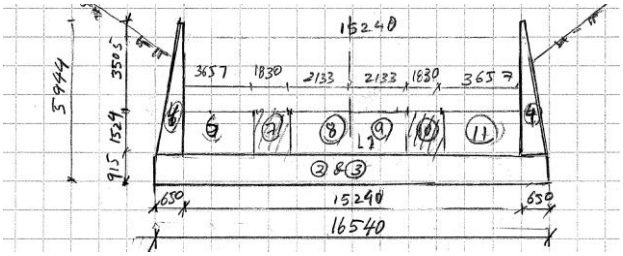
Project No: 5684
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Dry unit weight of soil	γ_{dry}	17	kN/m4	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff

Pressures and Forces



Unit width of dam section	W´	8.27	m	
Water pressure	SP	37.376	kPa	= gw H'
Water Force per unit width	SF'	71.201	kN/m	= 1/2 gw H'2
Water force	SF	588.836	kN	= TF' W'
Water Force arm from toe	SF_a	0.254	m	= H'/3
Silt pressure	TP	16.256	kPa	= gsub Hs Ka
Silt Force per unit width	TF´	19.819	kN/m	= 1/2 gsub Hs^2 Ka
Silt force	TF	163.906	kN	= SF' W'
Silt force arm from toe	TF_a	0.711	m	= Hs/3
Upstream uplift pressure	U1	37.376	kPa	= HP
Upstream uplift pressure per unit width	U1	309.100	kN/m	= Uu W'
Upstream uplift pressure	U2	28.032	kPa	U1-0.25(U1-U3)
Upstream uplift pressure per unit width	U2	231.825	kN/m	= Uu W'
Downstream uplift pressure	U3	0.000	kPa	= TP
Downstream uplift pressure per unit width	U3	0.000	kN/m	= Du W'
Uplift force	UF	2054.938	kN	= ((Uu' + Du')/2) * B
Uplift force arm from toe	UF_a1	13.434	m	= Centroid of Rectangular
Uplift force arm from toe	UF_a2	8.295	m	= Centroid of Trapezoid



Shear key rectangle section per unit width	C1´	21.847	kN/m	= gc (j b)
Shear key rectangle section force	C1	180.671	kN	= C2' W'
Shear key rectangle section arm from toe	$C1_a$	14.120	m	= B - b/2
Footing force per unit width	C2´	206.782	kN/m	= gc (j v) /2
Footing force	C2´	1710.090	kN	= C3' W'
Footing force arm from toe	$C2_a$	7.213	m	= (2/3) v
Footing force per unit width	C3´	73.116	kN/m	= gc (b h)
Footing force	C3	604.671	kN	= C1' W'
Footing force arm from toe	$C3_a$	7.928	m	= B - b/2
wall	C4	55.78	kN/m	= gsat (b h)
wall	C4´	804.63	kN	= D' W'
wall	$C4_a$	7.21	m	= B2/2 +v+B1
wall	C5	107.52	kN/m	= gsat (b h)
wall	C5´	24.73	kN	= D' W'
wall	C5a	1.42	m	= D' W' B2
Ogee Weir	C6	239.36	kN/m	= gsat (b h)
Ogee Weir	C6´	875.34	kN	= D' W'
Ogee Weir	C6a	9.16	m	= D' W' B2
Stoplog	C7	0.00	kN/m	= gsat (b h)
Stoplog	C7´	0.00	kN	= D' W'
Stoplog	C7a	10.10	m	= D' W' B2
Ogee Weir	C8	239.36	kN/m	= gsat (b h)
Ogee Weir	C8´	510.56	kN	= D' W'
Ogee Weir	C8a	9.16	m	= D' W' B2
LRIA (Note Ice loading on concrete per unit width	IF´	0.000		= Acting on the face of the dam
Ice force	IF	0.000		
LRIA (Note Ice force arm from toe	Ifa	1.986		= acting 0.3m below the normal winter water level

Typical Ogee and Wall Section - Loading Condition: Usual Summer

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/04/25

					(kN)	(m)	(kN-m)
Horizontal Forces							
	Hydrostatic load - Water Pressure				-588.84	0.25	-149.56
	Hydrostatic Pressure - Silt Load				-163.91	0.71	-116.57
	ice load				0.00	1.99	0.00
	Shear key			35.50	293.59		
			SHf=		293.59	Resisting forces	
			SHg=		-752.74	Sliding forces	
Vertical Forces							
	Uplift 1				-612.64	13.43	-8230.16
	Uplift 2				-1442.30	8.30	-11964.37
	Shear key rectangle section force				180.67	14.12	2551.07
	Footing force				1710.09	7.21	12334.03
	Footing force				604.67	7.93	4794.09
	wall				804.63	7.21	5803.40
	wall				-24.73	1.42	-35.09
	Ogee Weir				875.34	9.16	8018.13
	Stoplog				0.00	10.10	0.00
	Ogee Weir				510.56	9.16	4676.70
			S Vi =		4661.23	Downward forces	
			S Vh =		-2054.94	Uplift forces	

	ΣV	2606.29	kN	S Vi + S Vh
	ΣH	459.15	kN	SHf + SHg

Overturning Check

Restoring moments	$\Sigma M_{toe}R$	38142.33	kNm	
Overturning moments	$\Sigma M_{toe}O$	-20460.66	kNm	
Overturning Factor of Safety	O_{FOS}	1.86		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturning Factor of Safety	O_{allow}	1.50		
Efficiency	Efficiency	124%		

Sliding Check

Sliding Factor of Safety	SFOS =	3.97	=SV * m / SH
Allowable Sliding Factor of Safety	Sallow =	1.5	
Efficiency	EFF	265%	

Foundation Pressure Check

Total vertical loads	ΣV	2606.29	kN	S V = S Vi + S Vh
Restoring Moment	M_R	38142.33	kNm	
Overturning Moment	M_O	20460.66	kNm	
Distance of resultant of all forces from toe	X´	6.78	m	=(MR - Mo) / S V
Total gravity wall base width	B	14.43	m	
	B/2	7.21	m	X' < B/2 resultant force is near the Toe
	B/6	2.40	m	
Eccentricity , Absolute (positive) value	e	0.43	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	17.96	kPa	(S V / B L) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	25.74	kPa	(S V / B L) x (1 + 6e / B),

Typical Ogee and Wall Section - Loading Condition: Usual Summer

WILLS

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/04/25

Typical Ogee and Wall Section - Loading Condition: Usual Winter

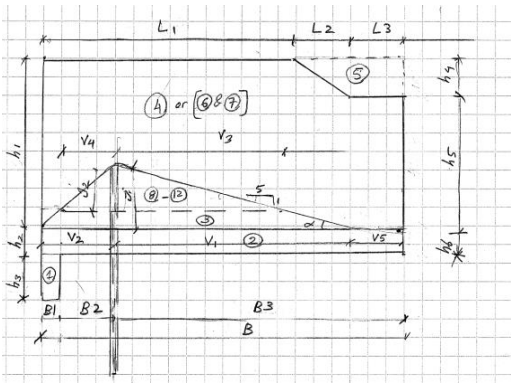
Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/04/25

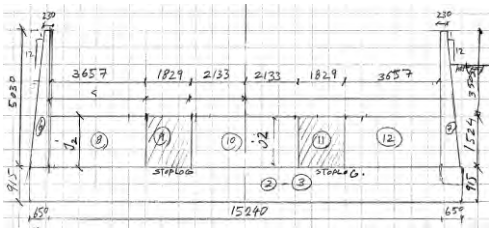
The Ogee section below is a controlled structure with wall backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 35°. Assuming that the allowable bearing pressure of the soil is 150 kNm−2, the coefficient of friction is 0.7 and the unit weight of reinforced concrete is 23.5 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.

Wall Geometry

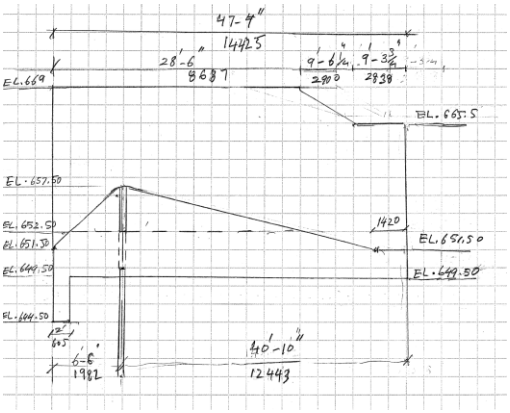
Downstream	:rise	1.00		0.2
Downstream	:run	5.00		0.2
		11.31		0.92342
Downstream: Incline angle	α1=	11.31	Degree	
	α2=	42.72	Degree	
ogee height at weir	J1	1830.00	mm	
ogee height at stoplogs	J2	1524.00	mm	
	v1	9150.00	mm	
Stem wall base width	v2	1981.76	mm	
	v3	7620.00	mm	
	v4	1650.39	mm	
	v5	3293.24	mm	
Stem width(Top)	s1	230.00	mm	= measured
Stem width(Top)	s2	660.00	mm	
shear key base width (Total)	B1	610.00	mm	= measured
base width (U/S)	B2	1372.00	mm	= measured
base width (D/S)	B3	12443.00	mm	= measured
Totam Base width	B	14425.00	mm	
Wall height- U/S	h1	5334.00	mm	= measured
Slab thickness	h2	610.00	mm	= measured
Shear key height	h3	1524.00	mm	= measured
top wall drop height	h4	1067.00	mm	= measured
Wall height- D/S	h5	4267.00	mm	= measured
Slab thickness	h6	610.00	mm	= measured
Wall Top Length	L1	8687.00	mm	= measured
Wall Top Length	L2	2900.00	mm	= measured
Wall Top Length	L3	2838.00	mm	= measured
	L	14425.00	mm	





Elevations and Depths

Top of stem - Elevation	E1	669.00	ft	
Bottom of Sherar Key - Elevation	E2	644.50	ft	= E1 - j
Top of Sill elevation	E3	651.50	ft	
Top of stem to top of water - Upstream	U_w	3657.60	mm	
Top of water elevation - Upstream	E4	657.00	ft	= E1 - Uw
Water height - Upstream	Hw	3810.00	mm	= j + h - Uw
Top of stem to soil - Upstream	U_s	5029.20	mm	
Top of soil elevation - Upstream	E5	652.50	ft	= E1 - Us
Height of soil - Upstream	H_s	2438.40	mm	= j + h - Us
Top of stem to top of water - Downstream	D_w	5334.00	mm	
Top of water elevation - Downstream	E6	651.50	ft	= E1 - Dw
Water height - Downstream	Hsd	0.00	mm	



Material Properties

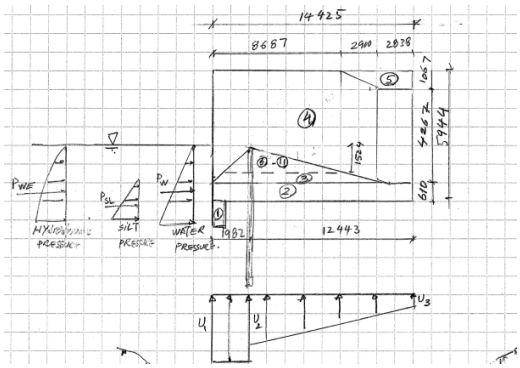
Concrete unit weight	γ_c	23.5	kN/m3
Water unit weight	γ_w	9.81	kN/m3
Saturated unit weight of soil	γ_{sat}	20	kN/m3

Typical Ogee and Wall Section - Loading Condition: Usual Winter

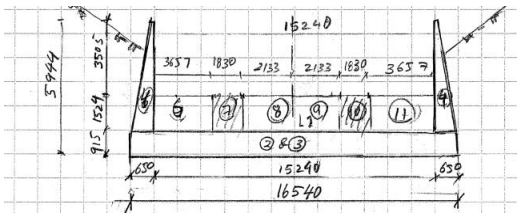
Project No: 5684
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Designed/Checked By: M D Gilani
Date: 03/04/25

	Dry unit weight of soil	γ_{dry}	17	kN/m4	
	Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
	Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
	Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
	Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
	Bedrock friction angle	φ_f	35	°	
	Friction coefficient	μ	0.70		= tan ff

Pressures and Forces



	Unit width of dam section	W´	8.27	m	
	Water pressure	SP	37.376	kPa	= gw H'
	Water Table Force per unit width	SF'	71.201	kN/m	= 1/2 gw H'2
	Water Table force	SF	588.836	kN	= TF' W'
	Water Table Force arm from toe	SF_a	0.254	m	= H'/3
	Silt pressure	TP	16.256	kPa	= gsub Hs Ka
	Silt Force per unit width	TF´	19.819	kN/m	= 1/2 gsub Hs^2 Ka
	Silt force	TF	163.906	kN	= SF' W'
	Silt force arm from toe	TF_a	0.711	m	= Hs/3
	Upstream uplift pressure	U1	37.376	kPa	= HP
	Upstream uplift pressure per unit width	U1	309.100	kN/m	= Uu W'
25% of Cut-off wall Efficiency	Upstream uplift pressure	U2	28.032	kPa	U1-0.25(U1-U3)
	Upstream uplift pressure per unit width	U2	231.825	kN/m	= Uu W'
	Downstream uplift pressure	U3	0.000	kPa	= TP
	Downstream uplift pressure per unit width	U3	0.000	kN/m	= Du W'
	Uplift force	UF	2054.938	kN	= ((Uu' + Du')/2) * B
	Uplift force arm from toe	UF_a1	13.434	m	= Centroid of Rectangular
	Uplift force arm from toe	UF_a2	8.295	m	= Centroid of Trapezoid



	Shear key rectangle section per unit width	C1´	21.847	kN/m	= gc (j b)
	Shear key rectangle section force	C1	180.671	kN	= C2' W'
	Shear key rectangle section arm from toe	$C1_a$	14.120	m	= B - b/2
	Footing force per unit width	C2´	206.782	kN/m	= gc (j v) /2
	Footing force	C2´	1710.090	kN	= C3' W'
	Footing force arm from toe	$C2_a$	7.213	m	= (2/3) v
	Footing force per unit width	C3´	73.116	kN/m	= gc (b h)
	Footing force	C3	604.671	kN	= C1' W'
	Footing force arm from toe	$C3_a$	7.928	m	= B - b/2
	wall	C4	55.78	kN/m	= gsat (b h)
	wall	C4´	804.63	kN	= D' W'
	wall	$C4_a$	7.21	m	= B2/2 +v+B1
	wall	C5	107.52	kN/m	= gsat (b h)
	wall	C5´	24.73	kN	= D' W'
	wall	C5a	1.42	m	= D' W' B2
	Ogee Weir	C6	239.36	kN/m	= gsat (b h)
	Ogee Weir	C6´	875.34	kN	= D' W'
	Ogee Weir	C6a	9.16	m	= D' W' B2
	Stoplog	C7	0.00	kN/m	= gsat (b h)
	Stoplog	C7´	0.00	kN	= D' W'
	Stoplog	C7a	10.10	m	= D' W' B2
	Ogee Weir	C8	239.36	kN/m	= gsat (b h)
	Ogee Weir	C8´	510.56	kN	= D' W'
	Ogee Weir	C8a	9.16	m	= D' W' B2
LRIA (Note	Ice loading on concrete per unit width	IF´	75.000		= Acting on the face of the dam
	Ice force	IF	620.250		
LRIA (Note	Ice force arm from toe	Ifa	1.986		= acting 0.3m below the normal winter water level

Typical Ogee and Wall Section - Loading Condition: Usual Winter

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Designed/Checked By: M D Gilani
Date: 03/04/25

					(kN)	(m)	(kN-m)
Horizontal Forces							
	Hydrostatic load - Water Pressure				-588.84	0.25	-149.56
	Hydrostatic Pressure - Silt Load				-163.91	0.71	-116.57
	ice load				-620.25	1.99	-1231.82
	Shear key			35.50	293.59		
			SHf=		293.59	Resisting forces	
			SHg=		-1372.99	Sliding forces	
Vertical Forces							
	Uplift 1				-612.64	13.43	-8230.16
	Uplift 2				-1442.30	8.30	-11964.37
	Shear key rectangle section force				180.67	14.12	2551.07
	Footing force				1710.09	7.21	12334.03
	Footing force				604.67	7.93	4794.09
	wall				804.63	7.21	5803.40
	wall				-24.73	1.42	-35.09
	Ogee Weir				875.34	9.16	8018.13
	Stoplog				0.00	10.10	0.00
	Ogee Weir				510.56	9.16	4676.70
			S Vi =		4661.23	Downward forces	
			S Vh =		-2054.94	Uplift forces	

	ΣV	2606.29	kN	S Vi + S Vh
	ΣH	1079.40	kN	SHf + SHg

Overturning Check

Restoring moments	**Σ**M_{toe}R	38142.33	kNm	
Overturning moments	**Σ**M_{toe}O	-21692.48	kNm	
Overturning Factor of Safety	O_{FOS}	1.76		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturing Factor of Safety	O_{allow}	1.50		
Efficiency	Efficiency	117%		

Sliding Check

Sliding Factor of Safety	SFOS =	1.69	=SV * m / SH
Allowable Sliding Factor of Safety	Sallow =	1.5	
Efficiency	EFF	113%	

Foundation Pressure Check

Total vertical loads	**Σ**V	2606.29	kN	S V = S Vi + S Vh
Restoring Moment	M_R	38142.33	kNm	
Overturning Moment	M_O	21692.48	kNm	
Distance of resultant of all forces from toe	X´	6.31	m	=(MR - Mo) / S V
Total gravity wall base width	B	14.43	m	
	B/2	7.21	m	X' < B/2 resultant force is near the Toe
	B/6	2.40	m	
Eccentricity , Absolute (positive) value	e	0.90	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	13.66	kPa	(S V / B L) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	30.03	kPa	(S V / B L) x (1 + 6e / B),

Typical Ogee and Wall Section - Loading Condition: Usual Winter

WILLS

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/04/25

Typical Ogee and Wall Section - Loading Condition: UnUsual Flood

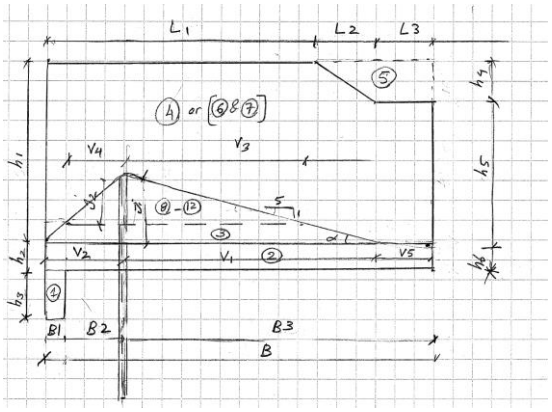
Project No: 5684
Project Name: LPRCA Deer Creek Dam
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Date: 03/04/25

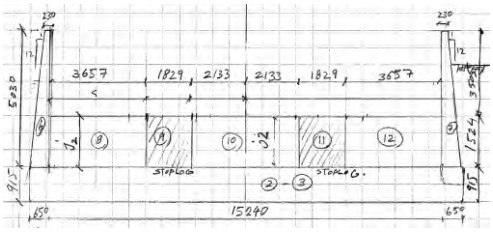
The Ogee section below is a controlled structure with wall backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 35°. Assuming that the allowable bearing pressure of the soil is 150 kNm−2, the coefficient of friction is 0.7 and the unit weight of reinforced concrete is 23.5 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.

Wall Geometry

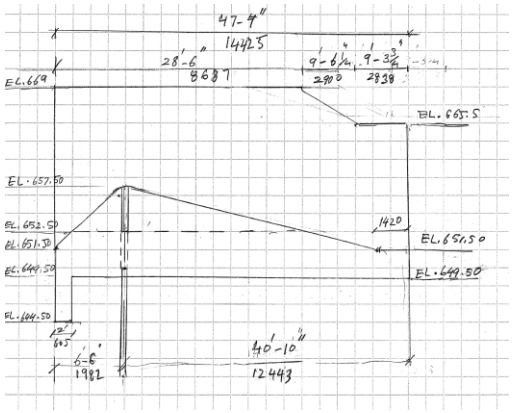
Downstream	:rise	1.00		0.2
Downstream	:run	5.00		0.2
		11.31		0.92342
Downstream: Incline angle	α1=	11.31	Degree	
	α2=	42.72	Degree	
ogee height at weir	J1	1830.00	mm	
ogee height at stoplogs	J2	1524.00	mm	
	v1	9150.00	mm	
Stem wall base width	v2	1981.76	mm	
	v3	7620.00	mm	
	v4	1650.39	mm	
	v5	3293.24	mm	
Stem width(Top)	s1	230.00	mm	= measured
Stem width(Top)	s2	660.00	mm	
shear key base width (Total)	B1	610.00	mm	= measured
base width (U/S)	B2	1372.00	mm	= measured
base width (D/S)	B3	12443.00	mm	= measured
Totam Base width	B	14425.00	mm	
Wall height- U/S	h1	5334.00	mm	= measured
Slab thickness	h2	610.00	mm	= measured
Shear key height	h3	1524.00	mm	= measured
top wall drop height	h4	1067.00	mm	= measured
Wall height- D/S	h5	4267.00	mm	= measured
Slab thickness	h6	610.00	mm	= measured
Wall Top Length	L1	8687.00	mm	= measured
Wall Top Length	L2	2900.00	mm	= measured
Wall Top Length	L3	2838.00	mm	= measured
	L	14425.00	mm	





Elevations and Depths

Top of stem - Elevation	E1	669.00	ft	
Bottom of Sherar Key - Elevation	E2	644.50	ft	= E1 - j
Top of Sill elevation	E3	651.50	ft	
Top of stem to top of water - Upstream	U_w	3505.20	mm	
Top of water elevation - Upstream	E4	657.50	ft	= E1 - Uw
Water height - Upstream	Hw	3962.40	mm	= j + h - Uw
Top of stem to soil - Upstream	U_s	5029.20	mm	
Top of soil elevation - Upstream	E5	652.50	ft	= E1 - Us
Height of soil - Upstream	H_s	2438.40	mm	= j + h - Us
Top of stem to top of water - Downstream	D_w	5334.00	mm	
Top of water elevation - Downstream	E6	651.50	ft	= E1 - Dw
Water height - Downstream	Hsd	0.00	mm	



Material Properties

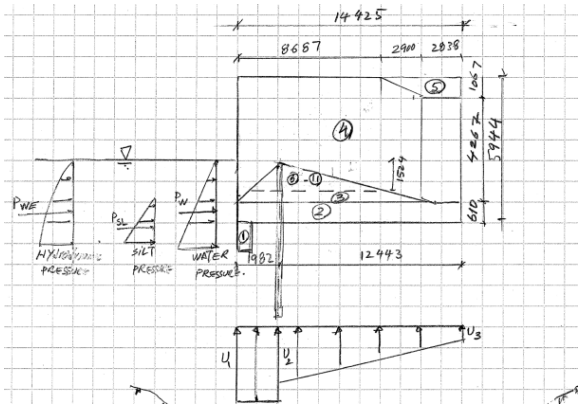
Concrete unit weight	γ_c	23.5	kN/m3
Water unit weight	γ_w	9.81	kN/m3
Saturated unit weight of soil	γ_{sat}	20	kN/m3

Typical Ogee and Wall Section - Loading Condition: UnUsual Flood

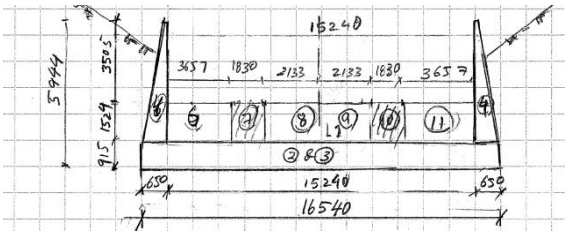
Project No: 5684
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Date: 03/04/25

Dry unit weight of soil	γ_{dry}	17	kN/m4	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff

Pressures and Forces



Unit width of dam section	W´	8.27	m	
Water pressure	SP	38.871	kPa	= gw H'
Water Table Force per unit width	SF'	77.012	kN/m	= 1/2 gw H'2
Water Table force	SF	636.885	kN	= TF' W'
Water Table Force arm from toe	SF_a	1.321	m	= H'/3
Silt pressure	TP	16.256	kPa	= gsub Hs Ka
Silt Force per unit width	TF´	19.819	kN/m	= 1/2 gsub Hs^2 Ka
Silt force	TF	163.906	kN	= SF' W'
Silt force arm from toe	TF_a	0.711	m	= Hs/3
Upstream uplift pressure	U1	38.871	kPa	= HP
Upstream uplift pressure per unit width	U1´	321.464	kN/m	= Uu W'
Upstream uplift pressure	U2	29.153	kPa	U1-0.25(U1-U3)
Upstream uplift pressure per unit width	U2	241.098	kN/m	= Uu W'
Downstream uplift pressure	U3	0.000	kPa	= TP
Downstream uplift pressure per unit width	U3	0.000	kN/m	= Du W'
Uplift force	UF	2137.135	kN	= ((Uu' + Du')/2) * B
Uplift force arm from toe	UF_a1	13.434	m	= Centroid of Rectangular
Uplift force arm from toe	UF_a2	8.295	m	= Centroid of Trapezoid



Shear key rectangle section per unit width	C1´	21.847	kN/m	= gc (j b)
Shear key rectangle section force	C1	180.671	kN	= C2' W'
Shear key rectangle section arm from toe	$C1_a$	14.120	m	= B - b/2
Footing force per unit width	C2´	206.782	kN/m	= gc (j v) /2
Footing force	C2´	1710.090	kN	= C3' W'
Footing force arm from toe	$C2_a$	7.213	m	= (2/3) v
Footing force per unit width	C3´	73.116	kN/m	= gc (b h)
Footing force	C3	604.671	kN	= C1' W'
Footing force arm from toe	$C3_a$	7.928	m	= B - b/2
wall	C4	55.78	kN/m	= gsat (b h)
wall	C4´	804.63	kN	= D' W'
wall	$C4_a$	7.21	m	= B2/2 +v+B1
wall	C5	107.52	kN/m	= gsat (b h)
wall	C5´	24.73	kN	= D' W'
wall	C5a	1.42	m	= D' W' B2
Ogee Weir	C6	239.36	kN/m	= gsat (b h)
Ogee Weir	C6´	875.34	kN	= D' W'
Ogee Weir	C6a	9.16	m	= D' W' B2
Stoplog	C7	0.00	kN/m	= gsat (b h)
Stoplog	C7´	0.00	kN	= D' W'
Stoplog	C7a	10.10	m	= D' W' B2
Ogee Weir	C8	239.36	kN/m	= gsat (b h)
Ogee Weir	C8´	510.56	kN	= D' W'
Ogee Weir	C8a	9.16	m	= D' W' B2
LRIA (Note Ice loading on concrete per unit width	IF´	0.000		= Acting on the face of the dam
Ice force	IF	0.000		
LRIA (Note Ice force arm from toe	Ifa	2.138		= acting 0.3m below the normal winter water level

Typical Ogee and Wall Section - Loading Condition: UnUsual Flood

Project No: 5684
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					(kN)	(m)	(kN-m)
Horizontal Forces							
	Hydrostatic load - Water Pressure				-636.89	1.32	-841.20
	Hydrostatic Pressure - Silt Load				-163.91	0.71	-116.57
	ice load				0.00	2.14	0.00
	Shear key			35.50	293.59		
			SHf=		293.59	Resisting forces	
			SHg=		-800.79	Sliding forces	
Vertical Forces							
	Uplift 1				-637.14	13.43	-8559.37
	Uplift 2				-1499.99	8.30	-12442.94
	Shear key rectangle section force				180.67	14.12	2551.07
	Footing force				1710.09	7.21	12334.03
	Footing force				604.67	7.93	4794.09
	wall				804.63	7.21	5803.40
	wall				-24.73	1.42	-35.09
	Ogee Weir				875.34	9.16	8018.13
	Stoplog				0.00	10.10	0.00
	Ogee Weir				510.56	9.16	4676.70
			S Vi =		4661.23	Downward forces	
			S Vh =		-2137.14	Uplift forces	

ΣV	2524.10	kN	S Vi + S Vh
ΣH	507.20	kN	SHf + SHg

Overturning Check

Restoring moments	$\Sigma M_{toe}R$	38142.33	kNm	
Overturning moments	$\Sigma M_{toe}O$	-21960.08	kNm	
Overturning Factor of Safety	O_{FOS}	1.74		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturning Factor of Safety	O_{allow}	1.30		
Efficiency	Efficiency	134%		

Sliding Check

Sliding Factor of Safety	SFOS =	3.48	=SV * m / SH
Allowable Sliding Factor of Safety	Sallow =	1.3	
Efficiency	EFF	268%	

Foundation Pressure Check

Total vertical loads	ΣV	2524.10	kN	S V = S Vi + S Vh
Restoring Moment	M_R	38142.33	kNm	
Overturning Moment	M_O	21960.08	kNm	
Distance of resultant of all forces from toe	X´	6.41	m	=(MR - Mo) / S V
Total gravity wall base width	B	14.43	m	
	B/2	7.21	m	X' < B/2 resultant force is near the Toe
	B/6	2.40	m	
Eccentricity , Absolute (positive) value	e	0.80	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	14.11	kPa	(S V / B L) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	28.21	kPa	(S V / B L) x (1 + 6e / B),

Typical Ogee and Wall Section - Loading Condition: UnUsual Flood

WILLS

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/04/25

Typical Ogee and Wall Section - Loading Condition: Unusual Winter

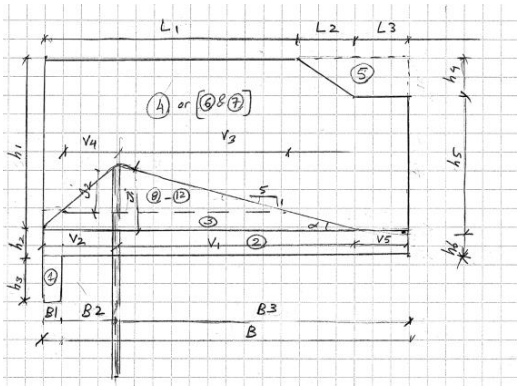
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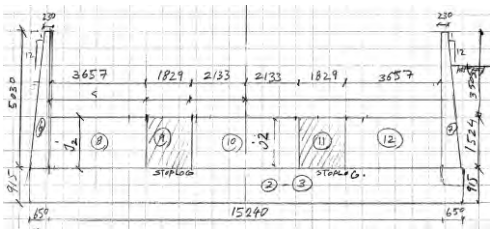
The Ogee section below is a controlled structure with wall backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 35°. Assuming that the allowable bearing pressure of the soil is 150 kNm−2, the coefficient of friction is 0.7 and the unit weight of reinforced concrete is 23.5 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.

Wall Geometry

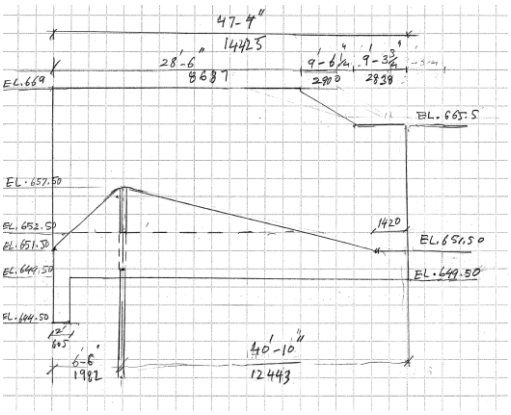
Downstream	:rise	1.00		0.2
Downstream	:run	5.00		0.2
		11.31		0.92342
Downstream: Incline angle	α1=	11.31	Degree	
	α2=	42.72	Degree	
ogee height at weir	J1	1830.00	mm	
ogee height at stoplogs	J2	1524.00	mm	
	v1	9150.00	mm	
Stem wall base width	v2	1981.76	mm	
	v3	7620.00	mm	
	v4	1650.39	mm	
	v5	3293.24	mm	
Stem width(Top)	s1	230.00	mm	= measured
Stem width(Top)	s2	660.00	mm	
shear key base width (Total)	B1	610.00	mm	= measured
base width (U/S)	B2	1372.00	mm	= measured
base width (D/S)	B3	12443.00	mm	= measured
Totam Base width	B	14425.00	mm	
Wall height- U/S	h1	5334.00	mm	= measured
Slab thickness	h2	610.00	mm	= measured
Shear key height	h3	1524.00	mm	= measured
top wall drop height	h4	1067.00	mm	= measured
Wall height- D/S	h5	4267.00	mm	= measured
Slab thickness	h6	610.00	mm	= measured
Wall Top Length	L1	8687.00	mm	= measured
Wall Top Length	L2	2900.00	mm	= measured
Wall Top Length	L3	2838.00	mm	= measured
	L	14425.00	mm	





Elevations and Depths

Top of stem - Elevation	E1	669.00	ft	
Bottom of Sherar Key - Elevation	E2	644.50	ft	= E1 - j
Top of Sill elevation	E3	651.50	ft	
Top of stem to top of water - Upstream	U_w	3657.60	mm	
Top of water elevation - Upstream	E4	657.00	ft	= E1 - Uw
Water height - Upstream	Hw	3810.00	mm	= j + h - Uw
Top of stem to soil - Upstream	U_s	5029.20	mm	
Top of soil elevation - Upstream	E5	652.50	ft	= E1 - Us
Height of soil - Upstream	H_s	2438.40	mm	= j + h - Us
Top of stem to top of water - Downstream	D_w	5334.00	mm	
Top of water elevation - Downstream	E6	651.50	ft	= E1 - Dw
Water height - Downstream	Hsd	0.00	mm	



Material Properties

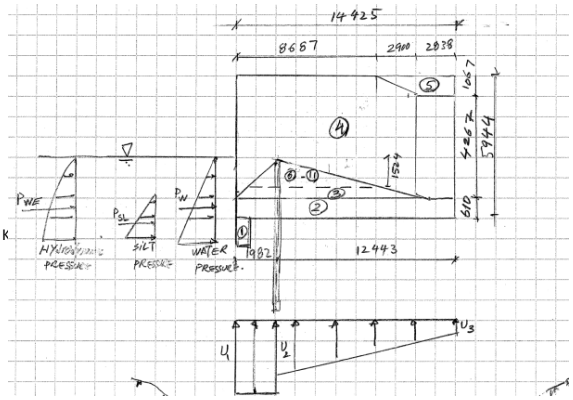
Concrete unit weight	γ_c	23.5	kN/m3
Water unit weight	γ_w	9.81	kN/m3
Saturated unit weight of soil	γ_{sat}	20	kN/m3

Typical Ogee and Wall Section - Loading Condition: Unusual Winter

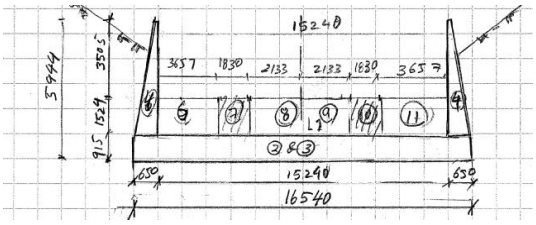
Project No: 5684
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Dry unit weight of soil	γ_{dry}	17	kN/m4	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff

Pressures and Forces



Unit width of dam section	W´	8.27	m	
Water pressure	SP	37.376	kPa	= gw H'
Water Table Force per unit width	SF'	71.201	kN/m	= 1/2 gw H'2
Water Table force	SF	588.836	kN	= TF' W'
Water Table Force arm from toe	SF_a	0.254	m	= H'/3
Silt pressure	TP	16.256	kPa	= gsub Hs Ka
Silt Force per unit width	TF´	19.819	kN/m	= 1/2 gsub Hs^2 K
Silt force	TF	163.906	kN	= SF' W'
Silt force arm from toe	TF_a	0.711	m	= Hs/3
Upstream uplift pressure	U1	37.376	kPa	= HP
Upstream uplift pressure per unit width	U1	309.100	kN/m	= Uu W'
Upstream uplift pressure	U2	28.032	kPa	U1-0.25(U1-U3)
Upstream uplift pressure per unit width	U2	231.825	kN/m	= Uu W'
Downstream uplift pressure	U3	0.000	kPa	= TP
Downstream uplift pressure per unit width	U3	0.000	kN/m	= Du W'
Uplift force	UF	2054.938	kN	= ((Uu' + Du')/2) * B
Uplift force arm from toe	UF_a1	13.434	m	= Centroid of Rectangular
Uplift force arm from toe	UF_a2	8.295	m	= Centroid of Trapezoid



Shear key rectangle section per unit width	C1´	21.847	kN/m	= gc (j b)
Shear key rectangle section force	C1	180.671	kN	= C2' W'
Shear key rectangle section arm from toe	$C1_a$	14.120	m	= B - b/2
Footing force per unit width	C2´	206.782	kN/m	= gc (j v) /2
Footing force	C2´	1710.090	kN	= C3' W'
Footing force arm from toe	$C2_a$	7.213	m	= (2/3) v
Footing force per unit width	C3´	73.116	kN/m	= gc (b h)
Footing force	C3	604.671	kN	= C1' W'
Footing force arm from toe	$C3_a$	7.928	m	= B - b/2
wall	C4	55.78	kN/m	= gsat (b h)
wall	C4´	804.63	kN	= D' W'
wall	$C4_a$	7.21	m	= B2/2 +v+B1
wall	C5	107.52	kN/m	= gsat (b h)
wall	C5´	24.73	kN	= D' W'
wall	C5a	1.42	m	= D' W' B2
Ogee Weir	C6	239.36	kN/m	= gsat (b h)
Ogee Weir	C6´	875.34	kN	= D' W'
Ogee Weir	C6a	9.16	m	= D' W' B2
Stoplog	C7	0.00	kN/m	= gsat (b h)
Stoplog	C7´	0.00	kN	= D' W'
Stoplog	C7a	10.10	m	= D' W' B2
Ogee Weir	C8	239.36	kN/m	= gsat (b h)
Ogee Weir	C8´	510.56	kN	= D' W'
Ogee Weir	C8a	9.16	m	= D' W' B2
LRIA (Note Ice loading on concrete per unit width	IF´	113.000		= Acting on the face of the dam
Ice force	IF	934.510		
LRIA (Note Ice force arm from toe	Ifa	1.986		= acting 0.3m below the normal winter water level

Typical Ogee and Wall Section - Loading Condition: Unusual Winter

Project No: 5684
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					(kN)	(m)	(kN-m)
Horizontal Forces							
	Hydrostatic load - Water Pressure				-588.84	0.25	-149.56
	Hydrostatic Pressure - Silt Load				-163.91	0.71	-116.57
	ice load				-934.51	1.99	-1855.94
	Shear key			35.50	293.59		
			SHf=		293.59	Resisting forces	
			SHg=		-1687.25	Sliding forces	
Vertical Forces							
	Uplift 1				-612.64	13.43	-8230.16
	Uplift 2				-1442.30	8.30	-11964.37
	Shear key rectangle section force				180.67	14.12	2551.07
	Footing force				1710.09	7.21	12334.03
	Footing force				604.67	7.93	4794.09
	wall				804.63	7.21	5803.40
	wall				-24.73	1.42	-35.09
	Ogee Weir				875.34	9.16	8018.13
	Stoplog				0.00	10.10	0.00
	Ogee Weir				510.56	9.16	4676.70
			S Vi =		4661.23	Downward forces	
			S Vh =		-2054.94	Uplift forces	

	ΣV	2606.29	kN	S Vi + S Vh
	ΣH	1393.66	kN	SHf + SHg

Overturning Check

Restoring moments	$\Sigma M_{toe}R$	38142.33	kNm	
Overturning moments	$\Sigma M_{toe}O$	-22316.60	kNm	
Overturning Factor of Safety	O_{FOS}	1.71		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturing Factor of Safety	O_{allow}	1.50		
Efficiency	Efficiency	114%		

Sliding Check

Sliding Factor of Safety	SFOS =	1.31	=SV * m / SH
Allowable Sliding Factor of Safety	Sallow =	1.3	
Efficiency	EFF	101%	

Foundation Pressure Check

Total vertical loads	ΣV	2606.29	kN	S V = S Vi + S Vh
Restoring Moment	M_R	38142.33	kNm	
Overturning Moment	M_O	22316.60	kNm	
Distance of resultant of all forces from toe	X´	6.07	m	=(MR - Mo) / S V
Total gravity wall base width	B	14.43	m	
	B/2	7.21	m	X' < B/2 resultant force is near the Toe
	B/6	2.40	m	
Eccentricity , Absolute (positive) value	e	1.14	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	11.48	kPa	(S V / B L) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	32.21	kPa	(S V / B L) x (1 + 6e / B),

Typical Ogee and Wall Section - Loading Condition: Unusual Winter

WILLS

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Typical Ogee and Wall Section - Loading Condition: Unusual Winter

WILLS

Project No: 5684
Project Name: LPRCA Deer Creek Dam
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Typical Ogee and Wall Section - Loading Condition: Extreme Seismic Load (EQ)

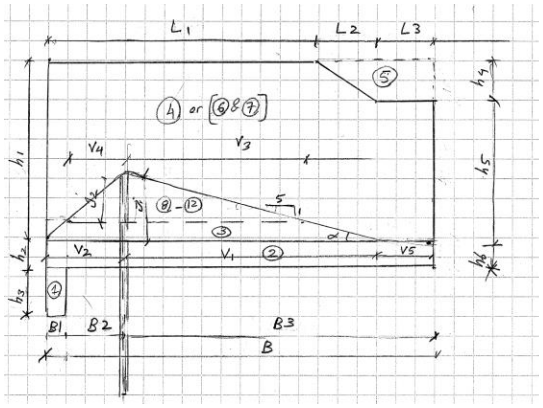
Project No: 5684
Project Name: LPRCA Deer Creek Dam
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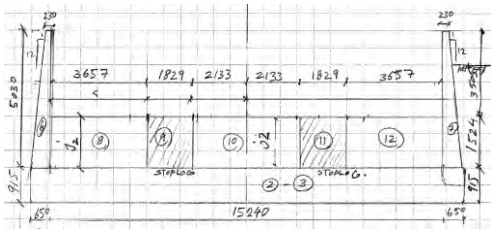
The Ogee section below is a controlled structure with wall backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 35°. Assuming that the allowable bearing pressure of the soil is 150 kNm−2, the coefficient of friction is 0.7 and the unit weight of reinforced concrete is 23.5 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.

Wall Geometry

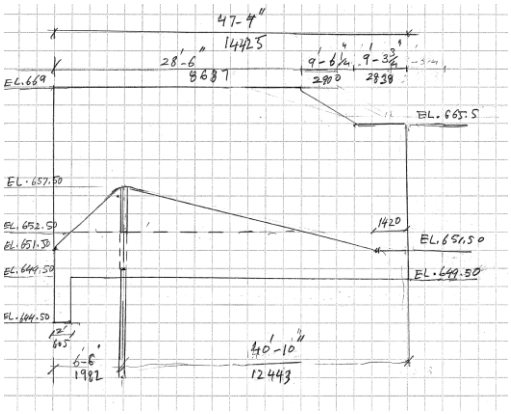
Downstream	:rise	1.00		0.2
Downstream	:run	5.00		0.2
		11.31		0.923419947
Downstream: Incline angle	α1=	11.31	Degree	
	α2=	42.72	Degree	
ogee height at weir	J1	1830.00	mm	= sin(a) r
ogee height at stoplogs	J2	1524.00		
	v1	9150.00		
Stem wall base width	v2	1981.76	mm	= j / tan (a)
	v3	7620.00		
	v4	1650.39		
	v5	3293.24		
Stem width(Top)	s1	230.00	mm	= measured
Stem width(Top)	s2	660.00		
shear key base width (Total)	B1	610.00	mm	= measured
base width (U/S)	B2	1372.00		= measured
base width (D/S)	B3	12443.00		= measured
Totam Base width	B	14425.00		
Wall height- U/S	h1	5334.00	mm	= measured
Slab thickness	h2	610.00		= measured
Shear key height	h3	1524.00		= measured
top wall drop height	h4	1067.00		= measured
Wall height- D/S	h5	4267.00		= measured
Slab thickness	h6	610.00		= measured
Wall Top Length	L1	8687.00	mm	= measured
Wall Top Length	L2	2900.00		= measured
Wall Top Length	L3	2838.00		= measured
	L	14425.00		





Elevations and Depths

Top of stem - Elevation	E1	669.00	ft	
Bottom of Sherar Key - Elevation	E2	644.50	ft	= E1 - j
Top of Sill elevation	E3	651.50	ft	
Top of stem to top of water -	U_w	3657.60	mm	
Top of water elevation	E4	657.00	ft	= E1 - Uw
Water height - Upstream	Hw	3810.00	mm	= j + h - Uw
Top of stem to soil - Upstream	U_s	5029.20	mm	
Top of soil elevation - Upstream	E5	652.50	ft	= E1 - Us
Height of soil - Upstream	H_s	2438.40	mm	= j + h - Us
Top of stem to top of water - Downstream	D_w	5334.00	mm	
Top of water elevation - Downstream	E6	651.50	ft	= E1 - Dw
Water height - Downstream	Hsd	0.00	mm	



Material Properties

Concrete unit weight	γ_c	23.5	kN/m3
Water unit weight	γ_w	9.81	kN/m3
Saturated unit weight of soil	γ_{sat}	20	kN/m3

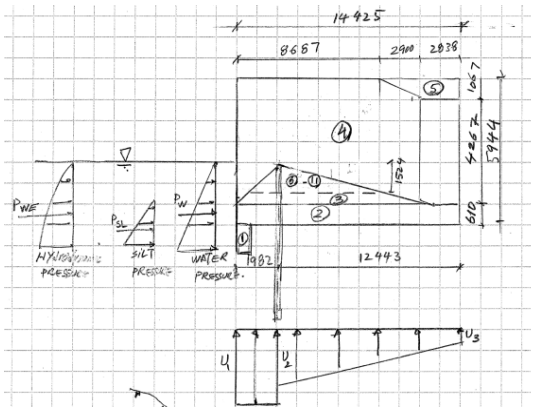
Typical Ogee and Wall Section - Loading Condition: Extreme Seismic Load (EQ)

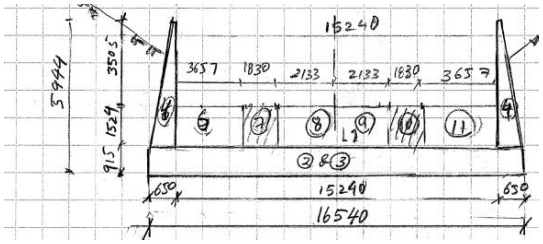
Project No: 5684
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Dry unit weight of soil	γ_{dry}	17	kN/m4	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff

Pressures and Forces

Unit width of dam section	W´	8.27	m	
Water pressure	SP	37.376	kPa	= gw H'
Water Table Force per unit width	SF'	71.201	kN/m	= 1/2 gw H'2
Water Table force	SF	588.836	kN	= TF' W'
Water Table Force arm from toe	SF_a	0.254	m	= H'/3
Silt pressure	TP	16.256	kPa	= gsub Hs Ka
Silt Force per unit width	TF´	19.819	kN/m	= 1/2 gsub Hs^2 Ka
Silt force	TF	163.906	kN	= SF' W'
Silt force arm from toe	TF_a	0.711	m	= Hs/3
Upstream uplift pressure	U1	37.376	kPa	= HP
Upstream uplift pressure per unit width	U1´	309.100	kN/m	= Uu W'
Upstream uplift pressure	U2	28.032	kPa	U1-0.25(U1-U3)
Upstream uplift pressure per unit width	U2	231.825	kN/m	= Uu W'
Downstream uplift pressure	U3	0.000	kPa	= TP
Downstream uplift pressure per unit width	U3	0.000	kN/m	= Du W'
Uplift force	UF	2054.938	kN	= ((Uu' + Du')/2) * B
Uplift force arm from toe	UF_a1	13.434	m	= Centroid of Rectangular
Uplift force arm from toe	UF_a2	8.295	m	= Centroid of Trapezoid
Shear key rectangle section per unit width	C1´	21.847	kN/m	= gc (j b)
Shear key rectangle section force	C1	180.671	kN	= C2' W'
Shear key rectangle section arm from toe	$C1_a$	14.120	m	= B - b/2
Footing force per unit width	C2´	206.782	kN/m	= gc (j v) /2
Footing force	C2´	1710.090	kN	= C3' W'
Footing force arm from toe	$C2_a$	7.213	m	= (2/3) v
Footing force per unit width	C3´	73.116	kN/m	= gc (b h)
Footing force	C3	604.671	kN	= C1' W'
Footing force arm from toe	$C3_a$	7.928	m	= B - b/2
wall	C4	55.78	kN/m	= gsat (b h)
wall	C4´	804.63	kN	= D' W'
wall	$C4_a$	7.21	m	= B2/2 +v+B1
wall	C5	107.52	kN/m	= gsat (b h)
wall	C5´	24.73	kN	= D' W'
wall	C5a	1.42	m	= D' W' B2
Ogee Weir	C6	166.00	kN/m	= gsat (b h)
Ogee Weir	C6´	607.08	kN	= D' W'
Ogee Weir	C6a	9.16	m	= D' W' B2
Stoplog	C7	0.00	kN/m	= gsat (b h)
Stoplog	C7´	0.00	kN	= D' W'
Stoplog	C7a	10.10	m	= D' W' B2
Ogee Weir	C8	239.36	kN/m	= gsat (b h)
Ogee Weir	C8´	510.56	kN	= D' W'
Ogee Weir	C8a	9.16	m	= D' W' B2
LRIA (Note Ice loading on concrete per unit width	IF´	0.000		= Acting on the face of the dam
Ice force	IF	0.000		
LRIA (Note Ice force arm from toe	Ifa	1.986		= acting 0.3m below the normal winter water level





Typical Ogee and Wall Section - Loading Condition: Extreme Seismic Load (EQ)

Project No: 5684
Project Name: LPRCA Deer Creek Dam
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Seismic Parameters

The DE is the 2500-year earthquake.(NBC 2020)

Probability of Exceedance (%) in 50 years	0	1%	2%	5%	10%	40%
1/Probability (Years)	10000	4975	2475	975	475	98
Probability of exceedance per annum	0.0001	0.0002	0.0004	0.001	0.0021	0.0102
Peak Ground Acceleration [PGA (g), 9.81 m/s2]			0.177	0.107	0.0662	0.0156

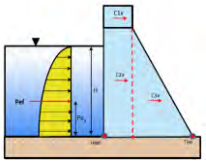
https://www.seismescanada.rncan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php?code=nbc2020&latitude=42.703&longitude=-80.563&siteDesignation=XS&siteDesignationXS=E

Seismic Parameters

Concrete unit weight	γ_c	2400	kg/m3	= 2400 kg/m3 = assumed for reinforced concrete
Concrete unit weight	γ_c	150	lb/ft3	= 149.8 lb/ft3 = assumed for reinforced concrete

Concrete Horizontal Force	Cross Section Area	Volume (m3)	Weight	Weight (kip)	Seismic coefficient	Horizontal Earthquake Force (kip)	Earthquake Force (kN)	Arm from Toe
	(m2)		(kg)	W	a	W.a		m
C1x _ Wall			444242.86	979.38	0.18	173.35	771.13	1.09
C2x -Ogee and Foundation				0.00	0.18			
C3x				0.00	0.18			
			444242.86	**979.38**		**173.35**	**771.13**	

Period of Vibration	t_e	0.50		= assumed
Total height of reservoir	H	17.50	ft	
Unit width of dam section	W´	28.00	ft	
Factor depending Principally on depth of water and t earthquake vibration period, te in seconds.	C_e	51.00	lb sf	= 51 / sqrt(1 - 0.72 (H /1000 te) 2)
PGA for 1% , 4975 year probability	α	0.1770		a = 0.1740
Additional total water load down to depth y	P_{ew}	1843.06	lb/ft	= 2/3 Ce a H2 , Westergaard formula
Hydrodynamic force	P_{ef}	51605.77	lb	=W' Pew
Hydrodynamic force	P_{ef}	229.62	kN	=W' Pew
Hydrostatic water force arm from toe	P_{ea}	0.61	m	= (2/5) H = 0.4H



					(kN)	(m)	(kN-m)
Horizontal Forces							
	Hydrostatic load - Upstream				-588.84	0.25	-149.56
	Hydrostatic Pressure - Silt Load				-163.91	0.71	-116.57
	Hydrostatic load - Downstream				0.00	0.00	0.00
	Hydrodynamic load - Upstream				-229.62	0.61	-139.98
	Shear key			35.50	293.59		0.00
	EQ- C1X- Wall &ogee				-771.13	1.09	-840.53
			SHf=		293.59	Resisting forces	
			SHg=		-1753.49	Sliding forces	
Vertical Forces							
	Uplift				-612.64	13.43	-8230.16
					-1442.30	8.30	-11964.37
	Shear key rectangle section force				180.67	14.12	2551.07
	Footing force				1710.09	7.21	12334.03
	Footing force				604.67	7.93	4794.09
	wall				804.63	7.21	5803.40
	wall				-24.73	1.42	-35.09
	Ogee Weir				607.08	9.16	5560.85
	Stoplog				0.00	10.10	0.00
	Ogee Weir				510.56	9.16	4676.70
			S Vi =		4392.97	Downward forces	
			S Vh =		-2054.94	Uplift forces	

Typical Ogee and Wall Section - Loading Condition: Extreme Seismic Load (EQ)

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	ΣV	2338.03	kN	S Vi + S Vh
	ΣH	1459.90	kN	SHf + SHg

Overturning Check

Restoring moments	$\Sigma M_{toe}R$	35685.05	kNm	
Overturning moments	$\Sigma M_{toe}O$	-21441.17	kNm	
Overturning Factor of Safety	O_{FOS}	1.66		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturing Factor of Safety	O_{allow}	1.10		
Efficiency	Efficiency	151%		

Sliding Check

Sliding Factor of Safety	SFOS =	1.12		=SV * m / SH
Allowable Sliding Factor of Safety	Sallow =	1.1		
Efficiency	EFF	102%		

Foundation Pressure Check

Total vertical loads	ΣV	2338.03	kN	S V = S Vi + S Vh
Restoring Moment	M_R	35685.05	kNm	
Overturning Moment	M_O	21441.17	kNm	
Distance of resultant of all forces from toe	X´	6.09	m	=(MR - Mo) / S V
Total gravity wall base width	B	14.43	m	
	B/2	7.21	m	X' < B/2 resultant force is near the Toe
	B/6	2.40	m	
Eccentricity , Absolute (positive) value	e	1.12	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

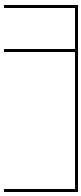
Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	10.47	(S V / B) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	28.73	(S V / B) x (1 + 6e / B),

Typical Ogee and Wall Section - Loading Condition: Extreme Seismic Load (EQ)

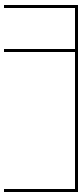
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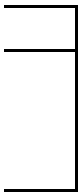
Project No: 5684
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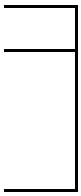












Typical Trasition Wall Section - Loading Condition: Usual Summer

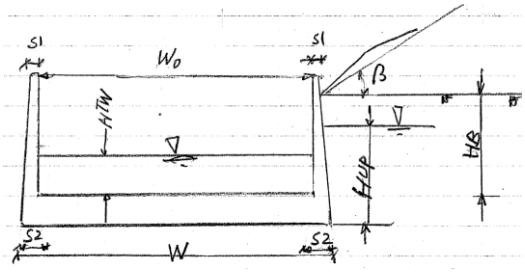
Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/05/25

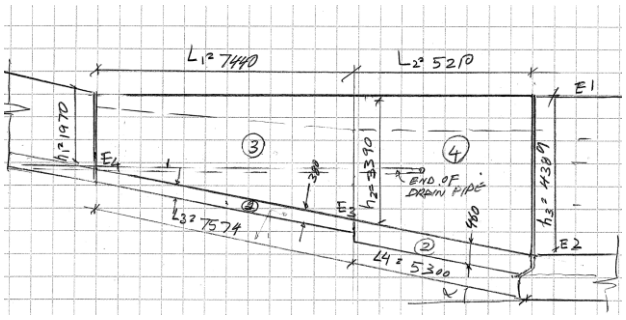
The Transition section from Chute to Stilling Basin have walls and varable slab with walls backfilled with granular material having a unit weight, ρ, of 20 kNm−3 and an internal angle of friction, ϕ, of 35°. Assuming that the allowable bearing pressure of the soil is 150 kNm−2, the coefficient of friction is 0.7 and the unit weight of reinforced concrete is 23.5 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.

Stilling Basin Geometry

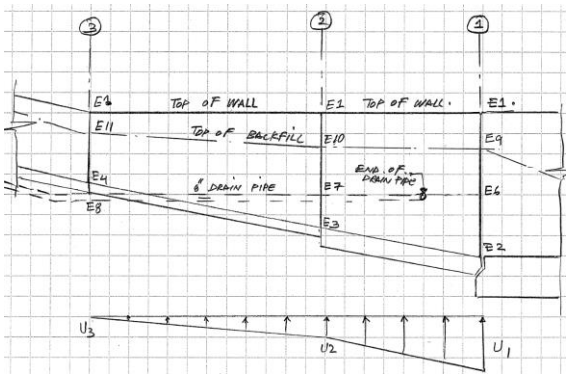
Upstream slab	:rise	1.00		0.2
Upstream slab	:run	5.00		0.2
		11.31		0.92342
Downstream: Incline angle	α =	11.31	Degree	
Earthfill slope at top of the wall	β=	42.72	Degree	
Slab Thickness	T1	380.00	mm	
Slab Thickness	T2	460.00	mm	
Stilling Basin Inside Length	W_0	15240.00	mm	
Stilling Basin Total Length	W	16012.00	mm	= j / tan (a)
Wall Thickess (Top)	s1	230.00	mm	= measured
Wall thickness(Bot)	s2	386.00	mm	
	s3	508.00	mm	
	s4	596.00	mm	
Wall height- U/S	h1	1969.33	mm	= measured
Slab thickness	h2	3344.20		= measured
Shear key height	h3	4386.79		= measured
Distance from wall Joint to Baffles	L1	7440.00	mm	= measured
Length of baffles	L2	5210.00		= measured
Distance from Baffles to End Sills	L3	7574.00		= measured
Distance from End Sills to bottom of toewall	L4	5300.00		
	B	12874.00	mm	

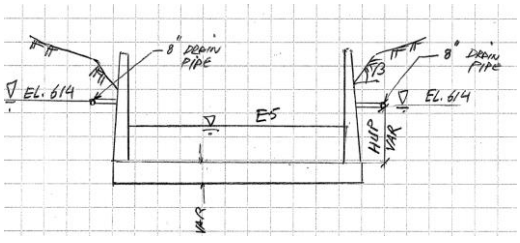




Elevations and Depths

Top of stem - Elevation	E1	623.00	ft	
Top of Sill elevation	E2	608.61	ft	
Top of Sill elevation	E3	612.03	ft	
Top of Sill elevation	E4	616.54	ft	
Top of stem to top of water inside the chute	U_W	2000.00	mm	
Top of water elevation inside the chute	E5	VAR	ft	
Tail Water height above top of the sill	HTW1	300.00	mm	
Top of stem to top of water Table @point 1	$U_{WT}1$	2743.20	mm	= E1-E6
Top of water Table elevation @point 1	E6	614.00	ft	
Water Table height above top of the sill @ Point 1	HUP1	2102.87	mm	= E6-E2+T2
Top of stem to top of water Table @point 2	$U_{WT}2$	2743.20	mm	= E1-E7
Top of water Table elevation @point 2	E7	614.00	ft	
Water Table height above top of the sill @ Point 2	HUP2	980.46	mm	= E7-E3+T1
Top of stem to top of water Table @point 3	U_{WT3}	0.00	mm	= E1-E8
Top of water Table elevation @point 3	E8	614.00	ft	
Water Table height above top of the sill @ Point 2	HUP3	0.00	mm	= E8-E4+T1
Top of stem to soil - @ point 1	U_S1	914.40	mm	= E1-E9
Top of soil elevation - @ point 1	E9	620.00	ft	
Eartfill Height above top of sill @ point 1	HB1	3471.67	mm	= E9-E2+T2
Top of stem to soil - @ point 2	U_S2	914.40	mm	= E1-E9
Top of soil elevation - @ point 2	E10	620.00	ft	
Eartfill Height above top of sill @ point 2	HB2	2429.26	mm	= E10-E3+T1





Typical Trasition Wall Section - Loading Condition: Usual Summer

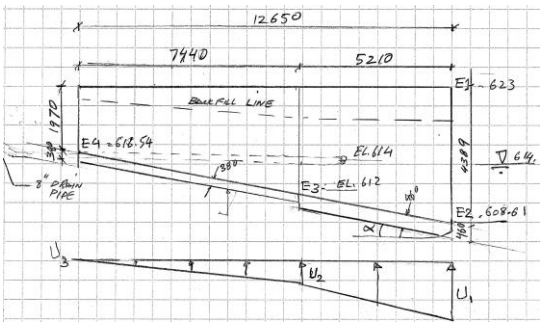
Project No: 5684
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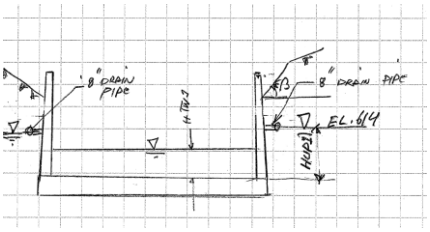
Material Properties

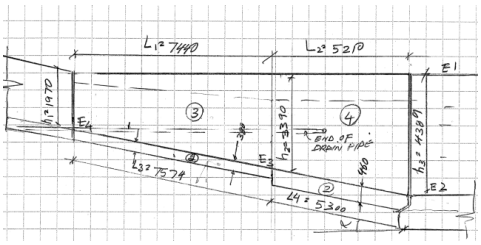
Concrete unit weight	γ_c	23.5	kN/m3	
Water unit weight	γ_w	9.81	kN/m3	
Saturated unit weight of soil	γ_{sat}	20	kN/m3	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff
Silt layer Cohession	c	0.00	kPa	

Pressures and Forces

width of stilling basin section	W´	16.012	m	
width of stilling basin section	W_0	15.24		
Water pressure on the wall @Point 1	TP	20.629	kPa	= gw HUP1
Water Table Force on the wall per unit width @Point 1	TF´	21.690	kN/m	= 1/2 gw HUP1'2
Water Table force on the Wall @Point 1	TF	113.006	kN	= TF' L2'
Water Table Force on the Wall arm from toe	TF_a	0.701	m	= HUP1'/3
Silt pressure on the Wall	SP	23.144	kPa	= gsub HB1 Ka
Silt Force on the Wall per unit width	SF'	40.175	kN/m	= 1/2 gsub HB1^2 Ka
Silt force on the wall	SF	209.312	kN	= SF' L'
Silt force on the wall arm from toe	SF_a	1.157	m	= HB1/3
Downstream uplift pressure	U1	20.629	kPa	= gw* HUP1
Downstream uplift pressure per unit width	U1´	330.314	kN/m	= U1 W'
Downstream uplift pressure at chute	U2	5.890	kPa	= gw* HUP2
Downstream uplift pressure at chute per unit width	U2	94.318	kN/m	= U2 W'
Uplift force	UF	1106.168	kN	= ((Uu1 + Uu2)/2) * B
Uplift force arm from toe	UF_a	2.600	m	= Centroid of trapezoid
Downstream uplift pressure at chute	U3	0.000	kPa	= gw* HUP3
Downstream uplift pressure at chute per unit width	U3	0.000	kN/m	= U3 W'
Uplift force	UF	350.864	kN	= ((Uu2 + Uu3)/2) * B
Uplift force arm from toe	UF_a	7.700	m	= Centroid of trapezoid
Footing force per unit width	C1´	142.987	kN/m	= gc (T1.W)
Footing force	C1	1082.985	kN	= C1' L3'
Footing force arm from toe	$C1_a$	8.930	m	= L4+L3/2
Footing force per unit width	C2´	173.090	kN/m	= gc (T2.W)
Footing force	C2´	917.376	kN	= C2' L4'
Footing force arm from toe	$C2_a$	2.605	m	= L4/2
Wall Section Area at 1-3	A1-3	0.607	m^2	
Wall Section Area at 1-2	A1-2	1.234	m^2	
Wall force per unit width	C3´	21.627	kN/m	= gc (A1-3 + A1-2)/2
Wall force	C3	161	kN	= C3' L1'
Wall force arm from toe	C3a	8.93	m	= L1/2+L2
Wall Section Area at 1-2	A1-2	1.234	m^2	
Wall Section Area at 1-1	A1-1	1.812	m^2	
Wall force per unit width	C4´	35.788	kN/m	= gc (A1-1 + A1-2)/2
Wall force	C4´	186	kN	= C4' L2'
Wall force arm from toe	C4a	2.61	m	= L2/2







Typical Trasition Wall Section - Loading Condition: Usual Summer

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					(kN)	(m)	(kN-m)
Horizontal Forces							
slab-1 load					-216.60	2.00	-433.19
Slab-2 Load					-183.48	0.70	-128.43
Wall Section Area at 1-1					-64.36	3.80	-244.57
Wall Section Area at 1-2				0.00	-74.58	3.10	-231.20
			SHf=		0.00	Resisting forces	
			SHg=		-539.01	Sliding forces	
Vertical Forces							
U1	Uplift				-1106.17	2.60	-2876.04
U2	Uplift				-350.86	7.70	-2701.65
C1- Footing force per unit width	Footing force per unit width				1082.98	8.93	9671.05
C2- Footing force per unit width	Footing force per unit width				917.38	2.61	2389.76
C3-Wall Section Area at 1-1	Wall Section Area at 1-1				321.80	8.93	2873.71
C4-Wall Section Area at 1-2	Wall Section Area at 1-2				372.91	2.61	971.42
C5							
C6	Weight of Water force				0.00	0.00	0.00
			S Vi =		2695.07	Downward forces	
			S Vh =		-1457.03	Uplift forces	

	ΣV	1238.04	kN	S Vi + S Vh
	ΣH	539.01	kN	SHf + SHg

Overturning Check

Restoring moments	$\Sigma M_{toe}R$	15905.95	kNm	
Overturning moments	$\Sigma M_{toe}O$	-6615.09	kNm	
Overturning Factor of Safety	O_{FOS}	2.40		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturing Factor of Safety	O_{allow}	1.50		
Efficiency	Efficiency	160%		

Sliding Check

Sliding Factor of Safety	SFOS =	1.61	=(SV * m + C * A) / SH
Allowable Sliding Factor of Safety	Sallow =	1.5	
Efficiency	EFF	107%	

Foundation Pressure Check

Total vertical loads	**Σ**V	1238.04	kN	S V = S Vi + S Vh
Restoring Moment	M_R	15905.95	kNm	
Overturning Moment	M_O	6615.09	kNm	
Distance of resultant of all forces from toe	X´	7.50	m	=(MR - Mo) / S V
Total gravity wall base width	B	12.87	m	
	B/2	6.44	m	X' > B/2 resultant force is near the Heel
	B/6	2.15	m	
Eccentricity , Absolute (positive) value	e	1.07	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	3.01	(S V / B L) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	9.00	(S V / B L) x (1 + 6e / B),

Typical Trasition Wall Section - Loading Condition: Usual Summer

WILLS

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Typical Trasition Wall Section - Loading Condition: Extreme Seismic

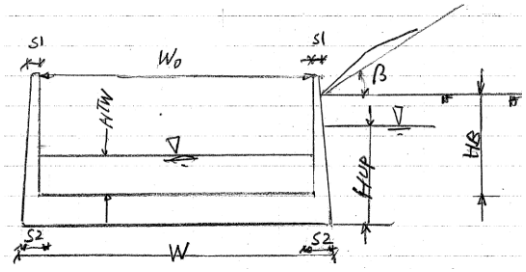
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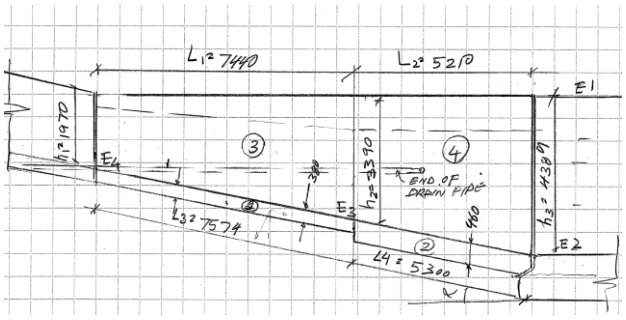
The Transition section from Chute to Stilling Basin have walls and varable slab with walls backfilled with granular material having a unit weight, ρ, of 20 kNm−3 and an internal angle of friction, ϕ, of 35°. Assuming that the allowable bearing pressure of the soil is 150 kNm−2, the coefficient of friction is 0.7 and the unit weight of reinforced concrete is 23.5 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.

Stilling Basin Geometry

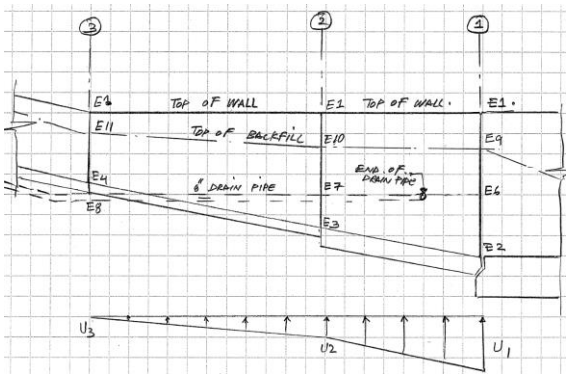
Upstream slab	:rise		1.00		0.2
Upstream slab	:run		5.00		0.2
			11.31		0.92342
Downstream: Incline angle		α =	11.31	Degree	
Earthfill slope at top of the wall		β=	42.72	Degree	
Slab Thickness		T1	380.00	mm	
Slab Thickness		T2	460.00	mm	
Stilling Basin Inside Length		W_0	15240.00	mm	
Stilling Basin Total Length		W	16012.00	mm	= j / tan (a)
Wall Thickess (Top)		s1	230.00	mm	= measured
Wall thickness(Bot)		s2	386.00	mm	
		s3	508.00	mm	
		s4	596.00	mm	
Wall height- U/S		h1	1969.33	mm	= measured
Slab thickness		h2	3344.20		= measured
Shear key height		h3	4386.79		= measured
Distance from wall Joint to Baffles		L1	7440.00	mm	= measured
Length of baffles		L2	5210.00		= measured
Distance from Baffles to End Sills		L3	7574.00		= measured
Distance from End Sills to bottom of toewall		L4	5300.00		
		B	12874.00	mm	

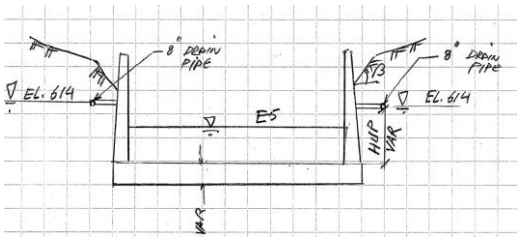




Elevations and Depths

Top of stem - Elevation	E1	623.00	ft	
Top of Sill elevation	E2	608.61	ft	
Top of Sill elevation	E3	612.03	ft	
Top of Sill elevation	E4	616.54	ft	
Top of stem to top of water inside the chute	U_W	2000.00	mm	
Top of water elevation inside the chute	E5	VAR	ft	
Tail Water height above top of the sill	HTW1	300.00	mm	
Top of stem to top of water Table @point 1	$U_{WT}1$	2743.20	mm	= E1-E6
Top of water Table elevation @point 1	E6	614.00	ft	
Water Table height above top of the sill @ Point 1	HUP1	2102.87	mm	= E6-E2+T2
Top of stem to top of water Table @point 2	$U_{WT}2$	2743.20	mm	= E1-E7
Top of water Table elevation @point 2	E7	614.00	ft	
Water Table height above top of the sill @ Point 2	HUP2	980.46	mm	= E7-E3+T1
Top of stem to top of water Table @point 3	U_{WT3}	0.00	mm	= E1-E8
Top of water Table elevation @point 3	E8	614.00	ft	
Water Table height above top of the sill @ Point 2	HUP3	0.00	mm	= E8-E4+T1
Top of stem to soil - @ point 1	U_S1	914.40	mm	= E1-E9
Top of soil elevation - @ point 1	E9	620.00	ft	
Eartfill Height above top of sill @ point 1	HB1	3471.67	mm	= E9-E2+T2
Top of stem to soil - @ point 2	U_S2	914.40	mm	= E1-E9
Top of soil elevation - @ point 2	E10	620.00	ft	
Eartfill Height above top of sill @ point 2	HB2	2429.26	mm	= E10-E3+T1





Typical Trasition Wall Section - Loading Condition: Extreme Seismic

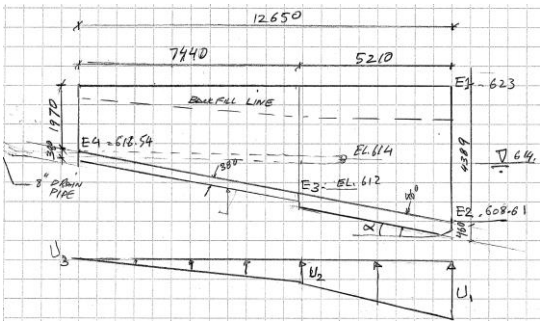
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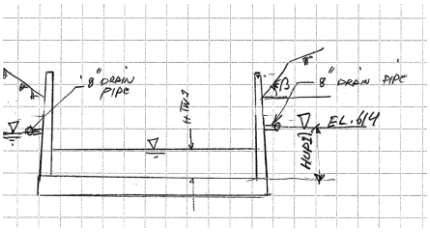
Material Properties

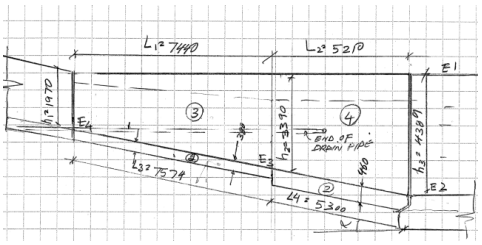
Concrete unit weight	γ_c	23.5	kN/m3	
Water unit weight	γ_w	9.81	kN/m3	
Saturated unit weight of soil	γ_{sat}	20	kN/m3	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff
Silt layer Cohession	c	0.00	kPa	

Pressures and Forces

width of stilling basin section	W´	16.012	m	
width of stilling basin section	W_0	15.24		
Water pressure on the wall @Point 1	TP	20.629	kPa	= gw HUP1
Water Table Force on the wall per unit width @Point 1	TF´	21.690	kN/m	= 1/2 gw HUP1'2
Water Table force on the Wall @Point 1	TF	113.006	kN	= TF' L2'
Water Table Force on the Wall arm from toe	TF_a	0.701	m	= HUP1'/3
Silt pressure on the Wall	SP	23.144	kPa	= gsub HB1 Ka
Silt Force on the Wall per unit width	SF'	40.175	kN/m	= 1/2 gsub HB1^2 Ka
Silt force on the wall	SF	209.312	kN	= SF' L'
Silt force on the wall arm from toe	SF_a	1.157	m	= HB1/3
Downstream uplift pressure	U1	20.629	kPa	= gw* HUP1
Downstream uplift pressure per unit width	U1	330.314	kN/m	= U1 W'
Downstream uplift pressure at chute	U2	5.890	kPa	= gw* HUP2
Downstream uplift pressure at chute per unit width	U2	94.318	kN/m	= U2 W'
Uplift force	UF	1106.168	kN	= ((Uu1 + Uu2)/2) * B
Uplift force arm from toe	UF_a	2.600	m	= Centroid of trapezoid
Downstream uplift pressure at chute	U3	0.000	kPa	= gw* HUP3
Downstream uplift pressure at chute per unit width	U3	0.000	kN/m	= U3 W'
Uplift force	UF	350.864	kN	= ((Uu2 + Uu3)/2) * B
Uplift force arm from toe	UF_a	7.700	m	= Centroid of trapezoid
Footing force per unit width	C1´	142.987	kN/m	= gc (T1.W)
Footing force	C1	1082.985	kN	= C1' L3'
Footing force arm from toe	$C1_a$	8.930	m	= L4+L3/2
Footing force per unit width	C2´	173.090	kN/m	= gc (T2.W)
Footing force	C2´	917.376	kN	= C2' L4'
Footing force arm from toe	$C2_a$	2.605	m	= L4/2
Wall Section Area at 1-3	A1-3	0.607	m^2	
Wall Section Area at 1-2	A1-2	1.234	m^2	
Wall force per unit width	C3´	21.627	kN/m	= gc (A1-3 + A1-2)/2
Wall force	C3	161	kN	= C3' L1'
Wall force arm from toe	C3a	8.93	m	= L1/2+L2
Wall Section Area at 1-2	A1-2	1.234	m^2	
Wall Section Area at 1-1	A1-1	1.812	m^2	
Wall force per unit width	C4´	35.788	kN/m	= gc (A1-1 + A1-2)/2
Wall force	C4´	186	kN	= C4' L2'
Wall force arm from toe	C4a	2.61	m	= L2/2







Seismic Parameters

The DE is the 2500-year earthquake.(NBC 2020)

Typical Trasition Wall Section - Loading Condition: Extreme Seismic

Project No: 5684
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Probability of Exceedance (%) in 50 years	0	1%	2%	5%	10%	40%
1/Probability (Years)	10000	4975	2475	975	475	98
Probability of exceedance per annum	0.0001	0.0002	0.0004	0.001	0.0021	0.0102
Peak Ground Acceleration [PGA (g), 9.81 m/s2]			0.177	0.107	0.0662	0.0156

https://www.seismescanada.rncan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php?code=nbc2020&latitude=42.703&longitude=-80.563&siteDesignation=XS&siteDesignationXS=E

Seismic Parameters

Concrete unit weight	γ_c	2400	kg/m3	= 2400 kg/m3 = assumed for reinforced concrete
Concrete unit weight	γ_c	150	lb/ft3	= 149.8 lb/ft3 = assumed for reinforced concrete

Concrete Horizontal Force	Cross Section Area	Volume (m3)	Weight	Weight (kip	Seismic coefficient	Horizontal Earthquake Force (kip)	Earthquake Force (kN)	Arm from Toe
	(m2)		(kg)	W	a	W.a		m
C1x _ Wall			269507.19	594.16	0.12	70.11	311.88	1.09
C2x -Ogee and Foundation				0.00	0.12			
C3x				0.00	0.12			
			269507.19	**594.16**	**2/3 PGA**	**70.11**	**311.88**	

					(kN)	(m)	(kN-m)
Horizontal Forces							
slab-1 load					-216.60	2.00	-433.19
Slab-2 Load					-183.48	0.70	-128.43
Wall Section Area at 1-1					-64.36	3.80	-244.57
Wall Section Area at 1-2				0.00	-74.58	3.10	-231.20
EQ					-311.88	2.00	-623.76
			SHf=		0.00	Resisting forces	
			SHg=		-850.89	Sliding forces	
Vertical Forces							
U1	Uplift				-1106.17	2.60	-2876.04
U2	Uplift				-350.86	7.70	-2701.65
C1- Footing force per unit width	Footing force per unit width				1082.98	8.93	9671.05
C2- Footing force per unit width	Footing force per unit width				917.38	2.61	2389.76
C3-Wall Section Area at 1-1	Wall Section Area at 1-1				321.80	8.93	2873.71
C4-Wall Section Area at 1-2	Wall Section Area at 1-2				372.91	2.61	971.42
C5							
C6	Weight of Water force				0.00	0.00	0.00
			S Vi =		2695.07	Downward forces	
			S Vh =		-1457.03	Uplift forces	

ΣV	1238.04	kN	S Vi + S Vh
ΣH	850.89	kN	SHf + SHg

Overturning Check

Restoring moments	$\Sigma M_{toe}R$	15905.95	kNm	
Overturning moments	$\Sigma M_{toe}O$	-7238.85	kNm	
Overturning Factor of Safety	O_{FOS}	2.20		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturning Factor of Safety	O_{allow}	1.10		
Efficiency	Efficiency	200%		

Sliding Check

Sliding Factor of Safety	SFOS =	1.02	=(SV * m + C * A) / SH

Typical Trasition Wall Section - Loading Condition: Extreme Seismic

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/05/25

Allowable Sliding Factor of Safety	Sallow =	1.1	
Efficiency	EFF	93%	

Foundation Pressure Check

Total vertical loads	ΣV	1238.04	kN	S V = S Vi + S Vh
Restoring Moment	M_R	15905.95	kNm	
Overturning Moment	M_O	7238.85	kNm	
Distance of resultant of all forces from toe	X´	7.00	m	=(MR - Mo) / S V
Total gravity wall base width	B	12.87	m	
	B/2	6.44	m	X' > B/2 resultant force is near the Heel
	B/6	2.15	m	
Eccentricity , Absolute (positive) value	e	0.57	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	4.42	(S V / B L) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	7.59	(S V / B L) x (1 + 6e / B),

Typical Stilling Basin and Wall Section - Loading Condition: Usual Summer

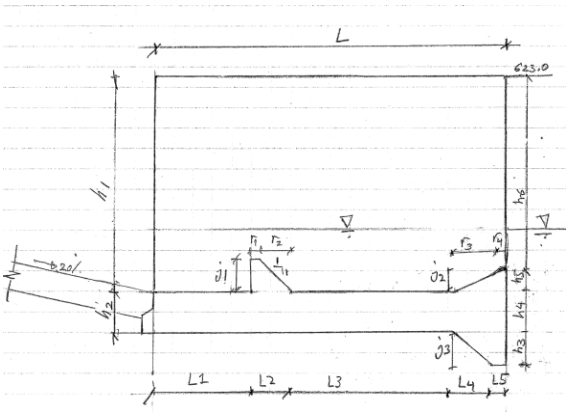
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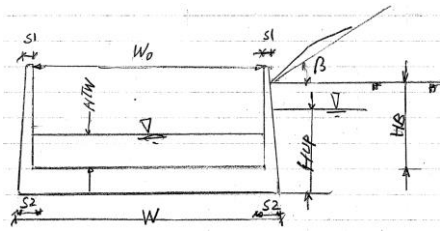
The Stilling Basin below is an energy dissipation structure with wall backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 35°. Assuming that the allowable bearing pressure of the soil is 150 kNm−2, the coefficient of friction is 0.7 and the unit weight of reinforced concrete is 23.5 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.

Stilling Basin Geometry

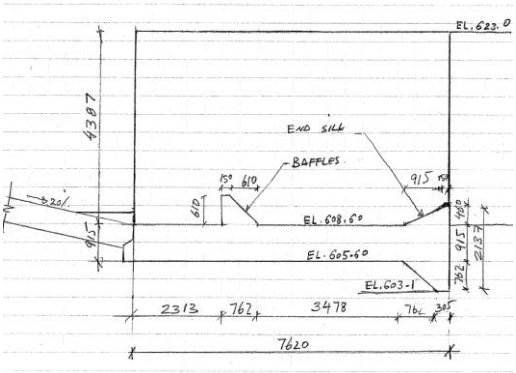
Upstream slab	:rise	1.00		0.2
Upstream slab	:run	5.00		0.2
		11.31		0.92342
Downstream: Incline angle	α =	11.31	Degree	
Earthfill slope at top of the wall	β=	42.72	Degree	
Baffle height at basin	J1	610.00	mm	= sin(a) r
End Sill height at Basin-D/S	J2	460.00		
Stilling Basin Inside Length	W_0	15240.00	mm	
Stilling Basin Total Length	W	16540.00	mm	= j / tan (a)
Wall Thickess (Top)	s1	230.00	mm	= measured
Wall thickness(Bot)	s2	650.00		
Baffle thickness at top	r1	150.00		= measured
Baffle sloped length at the bottom	r2	610.00		= measured
End Sill sloped length at the bottom	r3	915.00		
End Sill thickness	r4	150.00		
Wall height- U/S	h1	4387.00	mm	= measured
Slab thickness	h2	912.00		= measured
Shear key height	h3	762.00		= measured
Slab thickness	h4	912.00		= measured
End sill Height	h5	460.00		= measured
Wall height- D/S	h6	3927.00		= measured
Distance from wall Joint to Baffles	L1	2313.00	mm	= measured
Length of baffles	L2	762.00		= measured
Distance from Baffles to End Sills	L3	3478.00		= measured
Distance from End Sills to bottom of toewall	L4	762.00		
Thickness of Toewall/shearkey	L5	305.00		
	L	7620.00	mm	

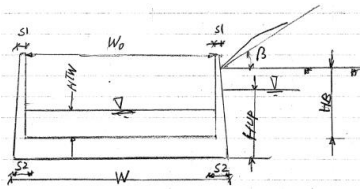




Elevations and Depths

Top of stem - Elevation	E1	623.00	ft	
Bottom of Sherar Key - Elevation	E2	603.00	ft	= E1 - j
Top of Sill elevation	E3	608.60	ft	
Bot of Sill elevation		605.60	ft	
Top of stem to top of water -	U_W	3352.80	mm	
Top of water elevation	E4	612.00	ft	= E1 - Uw
Tail Water height above top of the sil	HTW1	1036.32	mm	= j + h - Uw
Top of stem to top of water Table -	U_{WT}	200.00	mm	
Top of water Table elevation	E5	612.00	ft	= E1 - Uw
Tail Water height above top of the sil	HUP1	1036.32	mm	= j + h - Uw
Top of stem to soil - Upstream	U_S	3352.80	mm	
Top of soil elevation - Upstream	E6	620.00	ft	= E1 - Us
Eartfill Height above top of sill	HB	2000.00	mm	= j + h - Us
Top of stem to top of water - Downstream	D_W	3352.80	mm	
Top of water elevation - Downstream	E7	612.00	ft	= E1 - Dw
Water height - Downstream	HTW2	1036.32	mm	





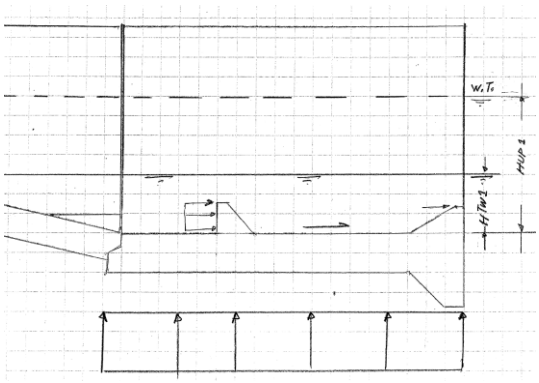
Typical Stilling Basin and Wall Section - Loading Condition: Usual Summer

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/05/25

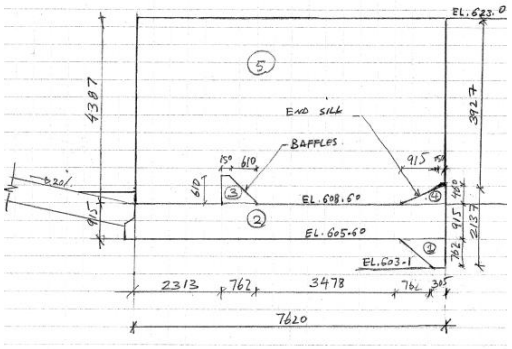
Material Properties

Concrete unit weight	γ_c	23.5	kN/m3	
Water unit weight	γ_w	9.81	kN/m3	
Saturated unit weight of soil	γ_{sat}	20	kN/m3	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff

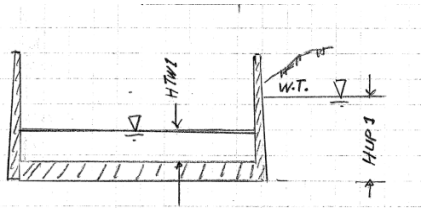
Pressures and Forces



width of stilling basin section	W´	16.54	m	
width of stilling basin section	W_0	15.24		
Water pressure on the wall	SP	10.166	kPa	= gsub Hs Ka
Water Table Force on the wallper unit width	SF'	5.268	kN/m	= 1/2 gsub Hs^2 Ka
Water Table force on the Wall	SF	40.140	kN	= SF' L'
Water Table Force on the Wall arm from toe	SF_a	3.810	m	= Hs/3
Silt pressure on the Wall	TP	13.333	kPa	= gw H'
Silt Force on the Wall per unit width	TF´	13.333	kN/m	= 1/2 gw H'2
Silt force on the wall	TF	220.533	kN	= TF' W'
Silt force on the wall arm from toe	TF_a	0.095	m	= H'/3
Upstream uplift pressure	U1	19.113	kPa	= HP
Upstream uplift pressure per unit width	U1	316.129	kN/m	= Uu W'
Downstream uplift pressure	U3	0.000	kPa	= TP
Downstream uplift pressure per unit width	U3	0.000	kN/m	= Du W'
Uplift force	UF	2408.906	kN	= ((Uu' + Du')/2) * B
Uplift force arm from toe	UF_a	3.810	m	= Centroid of trapezoid



Shear key rectangle section per unit width	C1´	12.284	kN/m	= gc (j b)
Shear key rectangle section force	C1	187.211	kN	= C2' W'
Shear key rectangle section arm from toe	$C1_a$	0.559	m	= B - b/2
Footing force per unit width	C2´	163.312	kN/m	= gc (j v) /2
Footing force	C2´	2701.178	kN	= C3' W'
Footing force arm from toe	$C2_a$	3.810	m	= (2/3) v
Baffles force per unit width	C3´	6.522	kN/m	= gc (b h)
Width of baffles	wb	460.000	mm	
Baffles force	C3	3.000	kN	= C1' W'
Number of Baales	n	17	m	= B - b/2
Baffles force total	C3t	51		
Baffles force arm from toe	$C3_a$	4.952	kN	
End Sill force per unit width	C4	6.57	kN/m	= gsat (b h)
End Sill force	C4´	100.08	kN	= D' W'
End sill force arm from toe	$C4_a$	0.46	m	= B2/2 +v+B1



Wall Section Area	C5A	1.93	m^2	= gsat (b h)
Wall force per unit width	C5´	45.36	kN/m	= D' W'
Wall force	C5	691.31	kN	
Wall force arm from toe	C5a	3.81	m	= D' W' B2
Water Pressure per unit width	C6A	10.17	kPa	= gsat (b h)
Weight of Water force per unit width	C6´	168.15	kN/m	= D' W'
Weight of Water force	C6	1281.31	kN	= D' W' B2
Weight of Water force arm from toe	C6a	3.81	m	
LRIA (Note Ice loading on concrete per unit width	IF´	0.000		= Acting on the face of the dam
Ice force	IF	0.000		
LRIA (Note Ice force arm from toe	Ifa	0.919		= acting 0.3m below the normal winter water level

Typical Stilling Basin and Wall Section - Loading Condition: Usual Summer

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					(kN)	(m)	(kN-m)
Horizontal Forces							
	Hydrostatic load - Water Pressure				0.00	3.81	0.00
	Hydrostatic Pressure - Silt Load				0.00	0.10	0.00
	ice load				0.00	0.92	0.00
	Shear key			8.88	146.79		
			SHf=		146.79	Resisting forces	
			SHg=		0.00	Sliding forces	
Vertical Forces							
	Uplift				-2408.91	3.81	-9177.93
C1	Shear key rectangle section force				187.21	0.56	104.65
C2	Footing force				2701.18	3.81	10291.49
C3	Baffle Weight				51.01	4.95	252.56
C4	End Sill force				100.08	0.46	45.54
C5	Wall force - 2 walls				1382.62	3.81	5267.79
C6	Weight of Water force				1281.31	3.81	4881.78
			S Vi =		5703.41	Downward forces	
			S Vh =		-2408.91	Uplift forces	

ΣV	3294.50 kN	S Vi + S Vh	
ΣH	146.79 kN	SHf + SHg	

Overturning Check

Restoring moments	$\Sigma M_{toe}R$	20843.80 kNm	
Overturning moments	$\Sigma M_{toe}O$	-9177.93 kNm	
Overturning Factor of Safety	O_{FOS}	2.27	= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturning Factor of Safety	O_{allow}	1.50	
Efficiency	Efficiency	151%	

Sliding Check

Sliding Factor of Safety	SFOS =	15.71	=SV * m / SH
Allowable Sliding Factor of Safety	Sallow =	1.5	
Efficiency	EFF	1048%	

Foundation Pressure Check

Total vertical loads	**Σ**V	3294.50	kN	S V = S Vi + S Vh
Restoring Moment	M_R	20843.80	kNm	
Overturning Moment	M_O	9177.93	kNm	
Distance of resultant of all forces from toe	X´	3.54	m	=(MR - Mo) / S V
Total gravity wall base width	B	7.62	m	
	B/2	3.81	m	X' < B/2 resultant force is near the Toe
	B/6	1.27	m	
Eccentricity , Absolute (positive) value	e	0.27	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	20.60	kPa	(S V / B L) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	31.68	kPa	(S V / B L) x (1 + 6e / B),

Typical Stilling Basin and Wall Section - Loading Condition: Unusual Flood

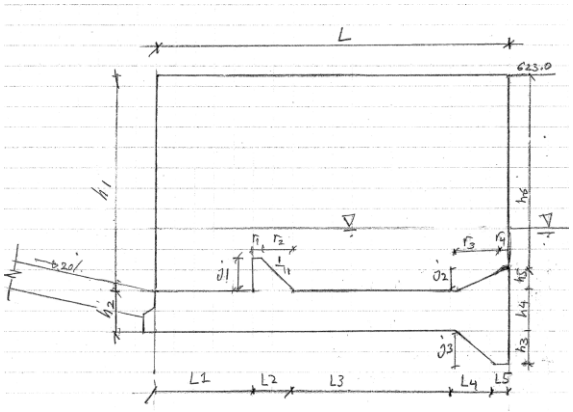
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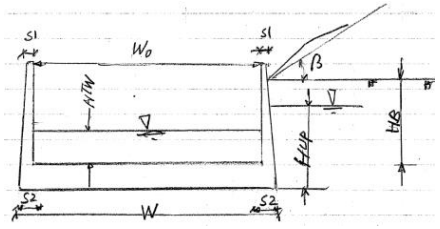
The Stilling Basin below is an energy dissipation structure with wall backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 35°. Assuming that the allowable bearing pressure of the soil is 150 kNm−2, the coefficient of friction is 0.7 and the unit weight of reinforced concrete is 23.5 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.

Stilling Basin Geometry

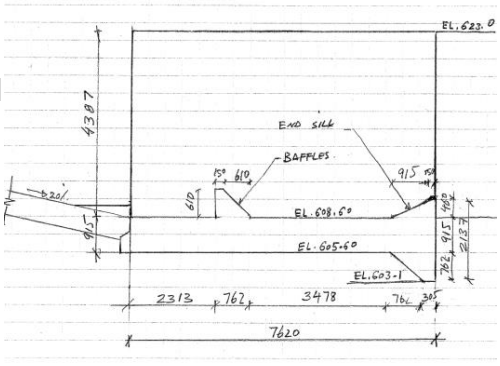
Upstream slab	:rise	1.00		0.2
Upstream slab	:run	5.00		0.2
		11.31		0.92342
Downstream: Incline angle	α =	11.31	Degree	
Earthfill slope at top of the wall	β=	42.72	Degree	
Baffle height at basin	J1	610.00	mm	= sin(a) r
End Sill height at Basin-D/S	J2	460.00		
Stilling Basin Inside Length	W_0	15240.00	mm	
Stilling Basin Total Length	W	16540.00	mm	= j / tan (a)
Wall Thickess (Top)	s1	230.00	mm	= measured
Wall thickness(Bot)	s2	650.00		
Baffle thickness at top	r1	150.00		= measured
Baffle sloped length at the bottom	r2	610.00		= measured
End Sill sloped length at the bottom	r3	915.00		
End Sill thickness	r4	150.00		
Wall height- U/S	h1	4387.00	mm	= measured
Slab thickness	h2	912.00		= measured
Shear key height	h3	762.00		= measured
Slab thickness	h4	912.00		= measured
End sill Height	h5	460.00		= measured
Wall height- D/S	h6	3927.00		= measured
Distance from wall Joint to Baffles	L1	2313.00	mm	= measured
Length of baffles	L2	762.00		= measured
Distance from Baffles to End Sills	L3	3478.00		= measured
Distance from End Sills to bottom of toewall	L4	762.00		
Thickness of Toewall/shearkey	L5	305.00		
	L	7620.00	mm	

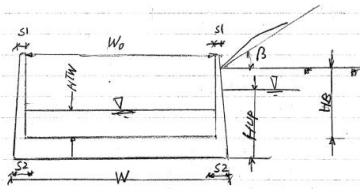




Elevations and Depths

Top of stem - Elevation	E1	623.00	ft	
Bottom of Sherar Key - Elevation	E2	603.00	ft	= E1 - j
Top of Sill elevation	E3	608.60	ft	
Bot of Sill elevation		605.60	ft	
Top of stem to top of water -	U_W	1828.80	mm	
Top of water elevation	E4	617.00	ft	= E1 - Uw
Tail Water height above top of the sil	HTW1	2560.32	mm	= j + h - Uw
Top of stem to top of water Table -	U_{WT}	200.00	mm	
Top of water Table elevation	E5	612.00	ft	= E1 - Uw
Tail Water height above top of the sil	HUP1	1036.32	mm	= j + h - Uw
Top of stem to soil - Upstream	U_S	3352.80	mm	
Top of soil elevation - Upstream	E6	620.00	ft	= E1 - Us
Eartfill Height above top of sill	HB	2000.00	mm	= j + h - Us
Top of stem to top of water - Downstream	D_W	3352.80	mm	
Top of water elevation - Downstream	E7	612.00	ft	= E1 - Dw
Water height - Downstream	HTW2	1036.32	mm	





Material Properties

Concrete unit weight	γ_c	23.5	kN/m3
Water unit weight	γ_w	9.81	kN/m3

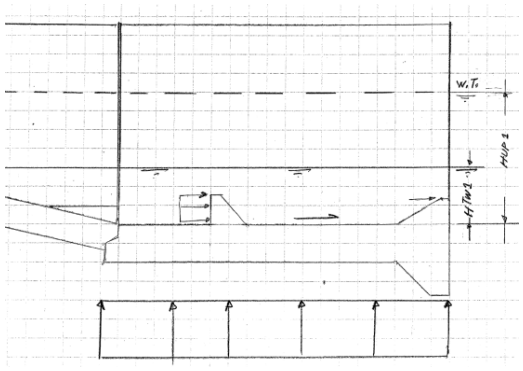
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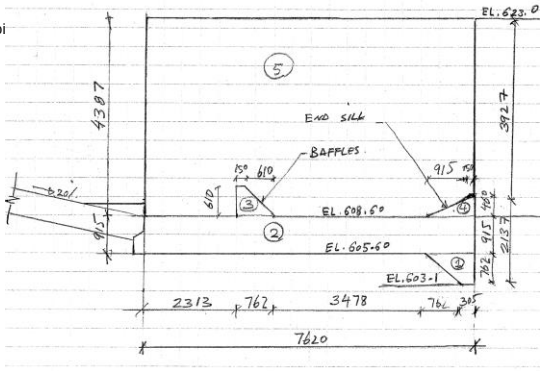
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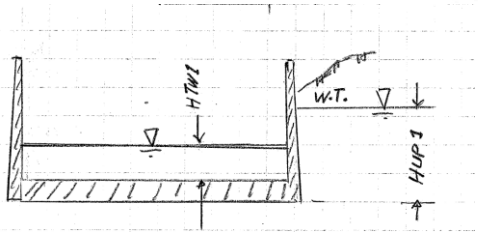
Saturated unit weight of soil	γ_{sat}	20	kN/m3	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff

Pressures and Forces

width of stilling basin section	W´	16.54	m	
width of stilling basin section	W_0	15.24		
Water pressure on the wall	SP	10.166	kPa	= gsub Hs Ka
Water Table Force on the wallper unit width	SF'	5.268	kN/m	= 1/2 gsub Hs^2 Ka
Water Table force on the Wall	SF	40.140	kN	= SF' L'
Water Table Force on the Wall arm from toe	SF_a	3.810	m	= Hs/3
Silt pressure on the Wall	TP	13.333	kPa	= gw H'
Silt Force on the Wall per unit width	TF´	13.333	kN/m	= 1/2 gw H'2
Silt force on the wall	TF	220.533	kN	= TF' W'
Silt force on the wall arm from toe	TF_a	0.095	m	= H'/3
Upstream uplift pressure	U1	34.063	kPa	= HP
Upstream uplift pressure per unit width	U1·	563.410	kN/m	= Uu W'
Downstream uplift pressure	U3	0.000	kPa	= TP
Downstream uplift pressure per unit width	U3	0.000	kN/m	= Du W'
Uplift force	UF	4293.181	kN	= ((Uu' + Du')/2) * B
Uplift force arm from toe	UF_a	3.810	m	= Centroid of trapezoi
Shear key rectangle section per unit width	C1´	7.156	kN/m	= gc (j b)
Shear key rectangle section force	C1	109.061	kN	= C2' W'
Shear key rectangle section arm from toe	$C1_a$	0.559	m	= B - b/2
Footing force per unit width	C2´	95.138	kN/m	= gc (j v) /2
Footing force	C2´	1573.580	kN	= C3' W'
Footing force arm from toe	$C2_a$	3.810	m	= (2/3) v
Baffles force per unit width	C3´	3.800	kN/m	= gc (b h)
Width of baffles	wb	460.000	mm	
Baffles force	C3	1.748	kN	= C1' W'
Number of Baales	n	17	m	= B - b/2
Baffles force total	C3t	30		
Baffles force arm from toe	$C3_a$	4.952	kN	
End Sill force per unit width	C4	3.83	kN/m	= gsat (b h)
End Sill force	C4´	58.30	kN	= D' W'
End sill force arm from toe	$C4_a$	0.46	m	= B2/2 +v+B1
Wall Section Area	C5A	1.93	m^2	= gsat (b h)
Wall force per unit width	C5´	45.36	kN/m	= D' W'
Wall force	C5	691.31	kN	
Wall force arm from toe	C5a	3.81	m	= D' W' B2
Water Pressure per unit width	C6A	25.12	kPa	= gsat (b h)
Weight of Water force per unit width	C6´	415.43	kN/m	= D' W'
Weight of Water force	C6	3165.58	kN	= D' W' B2
Weight of Water force arm from toe	C6a	3.81	m	
LRIA (Note Ice loading on concrete per unit width	IF´	0.000		= Acting on the face of the dam
Ice force	IF	0.000		
LRIA (Note Ice force arm from toe	Ifa	2.443		= acting 0.3m below the normal winter water level







					(kN)	(m)	(kN-m)
Horizontal Forces							
	Hydrostatic load - Water Pressure				0.00	3.81	0.00

Typical Stilling Basin and Wall Section - Loading Condition: Unusual Flood

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Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/05/25

	Hydrostatic Pressure - Silt Load				0.00	0.10	0.00	
	jump Load				-395.00	0.45	-177.75	The jump load extracted from Hydrotechnical report
	Shear key			8.88	146.79			
			SHf=		146.79	Resisting forces		
			SHg=		-395.00	Sliding forces		
Vertical Forces								
	Uplift				-4293.18	3.81	-16357.02	
C1	Shear key rectangle section force				109.06	0.56	60.96	
C2	Footing force				1573.58	3.81	5995.34	
C3	Baffle Weight				29.71	4.95	147.13	
C4	End Sill force				58.30	0.46	26.53	
C5	Wall force - 2 walls				1382.62	3.81	5267.79	
C6	Weight of Water force				3165.58	3.81	12060.87	
			S Vi =		6318.86	Downward forces		
			S Vh =		-4293.18	Uplift forces		

ΣV	2025.68	kN	S Vi + S Vh
ΣH	248.21	kN	SHf + SHg

Overturning Check

Restoring moments	$\Sigma M_{toe}R$	23558.62	kNm	
Overturning moments	$\Sigma M_{toe}O$	-16534.77	kNm	
Overturning Factor of Safety	O_{FOS}	1.42		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturning Factor of Safety	O_{allow}	1.30		
Efficiency	Efficiency	110%		

Sliding Check

Sliding Factor of Safety	SFOS =	5.71	=SV * m / SH
Allowable Sliding Factor of Safety	Sallow =	1.3	
Efficiency	EFF	440%	

Foundation Pressure Check

Total vertical loads	**Σ**V	2025.68	kN	S V = S Vi + S Vh
Restoring Moment	M_R	23558.62	kNm	
Overturning Moment	M_O	16534.77	kNm	
Distance of resultant of all forces from toe	X´	3.47	m	=(MR - Mo) / S V
Total gravity wall base width	B	7.62	m	
	B/2	3.81	m	X' < B/2 resultant force is near the Toe
	B/6	1.27	m	
Eccentricity , Absolute (positive) value	e	0.34	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	11.74	(S V / B) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	20.41	(S V / B) x (1 + 6e / B),

Typical Stilling Basin and Wall Section - Loading Condition: Unusual Flood

WILLS

Project No: 5684
Project Name: LPRCA Deer Creek Dam
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Date: 03/05/25

Typical Stilling Basin and Wall Section - Loading Condition: Extreme Seismic

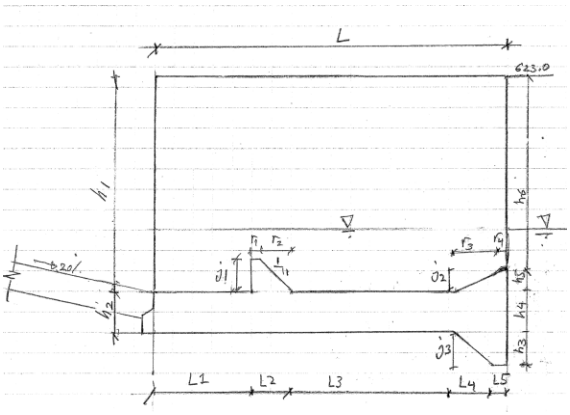
Project No: 5684
Project Name: LPRCA Deer Creek Dam
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Date: 03/05/25

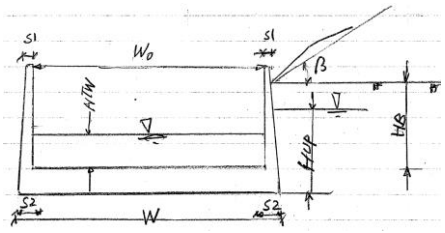
The Stilling Basin below is an energy dissipation structure with wall backfilled with granular material having a unit weight, ρ, of 19 kNm−3 and an internal angle of friction, ϕ, of 35°. Assuming that the allowable bearing pressure of the soil is 150 kNm−2, the coefficient of friction is 0.7 and the unit weight of reinforced concrete is 23.5 kNm−3

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.

Stilling Basin Geometry

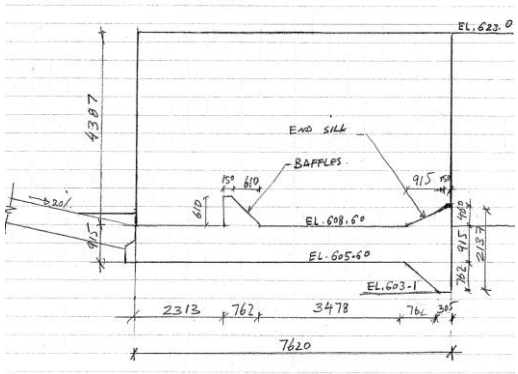
Upstream slab	:rise	1.00		0.2
Upstream slab	:run	5.00		0.2
		11.31		0.92342
Downstream: Incline angle	α =	11.31	Degree	
Earthfill slope at top of the wall	β=	42.72	Degree	
Baffle height at basin	J1	610.00	mm	= sin(a) r
End Sill height at Basin-D/S	J2	460.00		
Stilling Basin Inside Length	W_0	15240.00	mm	
Stilling Basin Total Length	W	16540.00	mm	= j / tan (a)
Wall Thickess (Top)	s1	230.00	mm	= measured
Wall thickness(Bot)	s2	650.00		
Baffle thickness at top	r1	150.00		= measured
Baffle sloped length at the bottom	r2	610.00		= measured
End Sill sloped length at the bottom	r3	915.00		
End Sill thickness	r4	150.00		
Wall height- U/S	h1	4387.00	mm	= measured
Slab thickness	h2	912.00		= measured
Shear key height	h3	762.00		= measured
Slab thickness	h4	912.00		= measured
End sill Height	h5	460.00		= measured
Wall height- D/S	h6	3927.00		= measured
Distance from wall Joint to Baffles	L1	2313.00	mm	= measured
Length of baffles	L2	762.00		= measured
Distance from Baffles to End Sills	L3	3478.00		= measured
Distance from End Sills to bottom of toewall	L4	762.00		
Thickness of Toewall/shearkey	L5	305.00		
	L	7620.00	mm	

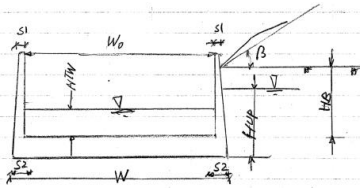




Elevations and Depths

Top of stem - Elevation	E1	623.00	ft	
Bottom of Sherar Key - Elevation	E2	603.00	ft	= E1 - j
Top of Sill elevation	E3	608.60	ft	
Bot of Sill elevation		605.60	ft	
Top of stem to top of water -	U_W	3352.80	mm	
Top of water elevation	E4	612.00	ft	= E1 - Uw
Tail Water height above top of the sil	HTW1	1036.32	mm	= j + h - Uw
Top of stem to top of water Table -	U_{WT}	200.00	mm	
Top of water Table elevation	E5	612.00	ft	= E1 - Uw
Tail Water height above top of the sil	HUP1	1036.32	mm	= j + h - Uw
Top of stem to soil - Upstream	U_S	3352.80	mm	
Top of soil elevation - Upstream	E6	620.00	ft	= E1 - Us
Eartfill Height above top of sill	HB	2000.00	mm	= j + h - Us
Top of stem to top of water - Downstream	D_W	3352.80	mm	
Top of water elevation - Downstream	E7	612.00	ft	= E1 - Dw
Water height - Downstream	HTW2	1036.32	mm	





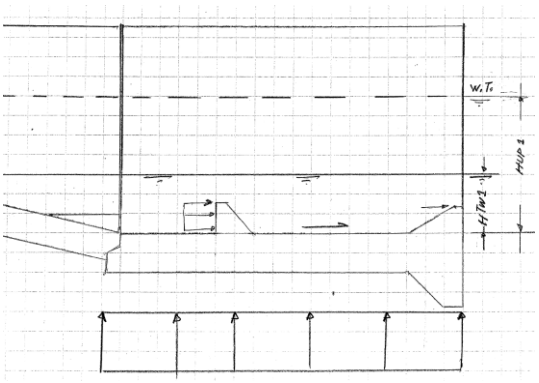
Typical Stilling Basin and Wall Section - Loading Condition: Extreme Seismic

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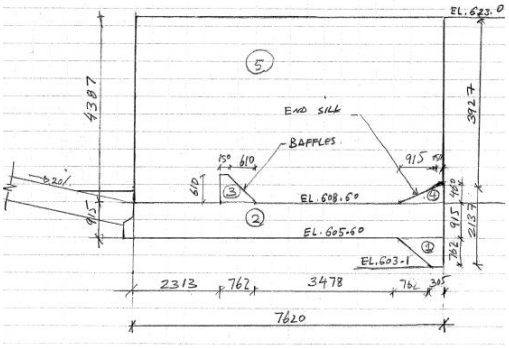
Material Properties

Concrete unit weight	γ_c	23.5	kN/m3	
Water unit weight	γ_w	9.81	kN/m3	
Saturated unit weight of soil	γ_{sat}	20	kN/m3	
Submerged unit weight of soil	γ_{sub}	10.19	kN/m3	= gsat - gw
Angle of internal friction of soil backfill	φs	30	°	fs = 30.0 o
Active earth pressure coefficient - upstream	K_a	0.33		= (1 - sin Fs)/ (1+ sin Fs) = (Tan(45 - Fs/2))^2
Passive earth pressure coefficient - downstream	K_p	3		= (1 + sin Fs)/ (1- sin Fs) = (Tan(45 + Fs/2))^2 = 1/Ka
Bedrock friction angle	φ_f	35	°	
Friction coefficient	μ	0.70		= tan ff

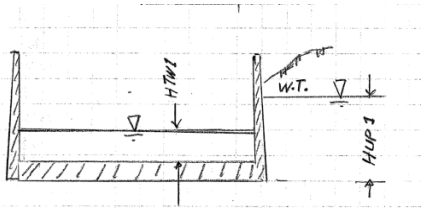
Pressures and Forces



width of stilling basin section	W´	16.54	m	
width of stilling basin section	W_0	15.24		
Water pressure on the wall	SP	10.166	kPa	= gsub Hs Ka
Water Table Force on the wallper unit width	SF'	5.268	kN/m	= 1/2 gsub Hs^2 Ka
Water Table force on the Wall	SF	40.140	kN	= SF' L'
Water Table Force on the Wall arm from toe	SF_a	3.810	m	= Hs/3
Silt pressure on the Wall	TP	13.333	kPa	= gw H'
Silt Force on the Wall per unit width	TF´	13.333	kN/m	= 1/2 gw H'2
Silt force on the wall	TF	220.533	kN	= TF' W'
Silt force on the wall arm from toe	TF_a	0.095	m	= H'/3
Upstream uplift pressure	U1	19.113	kPa	= HP
Upstream uplift pressure per unit width	U1	316.129	kN/m	= Uu W'
Downstream uplift pressure	U3	0.000	kPa	= TP
Downstream uplift pressure per unit width	U3	0.000	kN/m	= Du W'
Uplift force	UF	2408.906	kN	= ((Uu' + Du')/2) * B
Uplift force arm from toe	UF_a	3.810	m	= Centroid of trapezoid



Shear key rectangle section per unit width	C1´	12.284	kN/m	= gc (j b)
Shear key rectangle section force	C1	187.211	kN	= C2' W'
Shear key rectangle section arm from toe	$C1_a$	0.559	m	= B - b/2
Footing force per unit width	C2´	163.312	kN/m	= gc (j v) /2
Footing force	C2´	2701.178	kN	= C3' W'
Footing force arm from toe	$C2_a$	3.810	m	= (2/3) v
Baffles force per unit width	C3´	6.522	kN/m	= gc (b h)
Width of baffles	wb	460.000	mm	
Baffles force	C3	3.000	kN	= C1' W'
Number of Baales	n	17	m	= B - b/2
Baffles force total	C3t	51	kN	
Baffles force arm from toe	$C3_a$	4.952	kN	
End Sill force per unit width	C4	6.57	kN/m	= gsat (b h)
End Sill force	C4´	100.08	kN	= D' W'
End sill force arm from toe	$C4_a$	0.46	m	= B2/2 +v+B1



Wall Section Area	C5A	1.93	m^2	= gsat (b h)
Wall force per unit width	C5´	45.36	kN/m	= D' W'
Wall force	C5	691.31	kN	
Wall force arm from toe	C5a	3.81	m	= D' W' B2
Water Pressure per unit width	C6A	10.17	kPa	= gsat (b h)
Weight of Water force per unit width	C6´	168.15	kN/m	= D' W'
Weight of Water force	C6	1281.31	kN	= D' W' B2
Weight of Water force arm from toe	C6a	3.81	m	
LRIA (Note Ice loading on concrete per unit width	IF´	0.000		= Acting on the face of the dam
Ice force	IF	0.000		
LRIA (Note Ice force arm from toe	Ifa	0.919		= acting 0.3m below the normal winter water level

Typical Stilling Basin and Wall Section - Loading Condition: Extreme Seismic

Project No: 5684
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Seismic Parameters

The DE is the 2500-year earthquake.(NBC 2020)

Probability of Exceedance (%) in 50 years	0	1%	2%	5%	10%	40%
1/Probability (Years)	10000	4975	2475	975	475	98
Probability of exceedance per annum	0.0001	0.0002	0.0004	0.001	0.0021	0.0102
Peak Ground Acceleration [PGA (g), 9.81 m/s2]			0.177	0.107	0.0662	0.0156

https://www.seismescanada.rncan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php?code=nbc2020&latitude=42.703&longitude=-80.563&siteDesignation=XS&siteDesignationXS=E

Seismic Parameters

Concrete unit weight	γ_c	2400	kg/m3	= 2400 kg/m3 = assumed for reinforced concrete
Concrete unit weight	γ_c	150	lb/ft3	= 149.8 lb/ft3 = assumed for reinforced concrete

Concrete Horizontal Force	Cross Section Area	Volume (m3)	Weight	Weight (kip	Seismic coefficient	Horizontal Earthquake Force (kip)	Earthquake Force (kN)	Arm from Toe
	(m2)		(kg)	W	a	W.a		m
C1x _ Wall			442200.00	974.87	0.12	115.04	511.72	1.09
C2x -Ogee and Foundation				0.00	0.12			
C3x				0.00	0.12			
			442200.00	**974.87**	**2/3 of PGA**	**115.04**	**511.72**	

					(kN)	(m)	(kN-m)
Horizontal Forces							
	Hydrostatic load - Water Pressure				0.00	3.81	0.00
	Hydrostatic Pressure - Silt Load				0.00	0.10	0.00
	ice load				0.00	0.92	0.00
	Shear key			8.88	146.79		0.00
	EQ				-511.72	1.00	-511.72
			SHf=		146.79	Resisting forces	
			SHg=		-511.72	Sliding forces	
Vertical Forces							
	Uplift				-2408.91	3.81	-9177.93
C1	Shear key rectangle section force				187.21	0.56	104.65
C2	Footing force				2701.18	3.81	10291.49
C3	Baffle Weight				51.01	4.95	252.56
C4	End Sill force				100.08	0.46	45.54
C5	Wall force - 2 walls				1382.62	3.81	5267.79
C6	Weight of Water force				1281.31	3.81	4881.78
			S Vi =		5703.41	Downward forces	
			S Vh =		-2408.91	Uplift forces	

ΣV	3294.50	kN	S Vi + S Vh
ΣH	364.93	kN	SHf + SHg

Overturning Check

Restoring moments	$\Sigma M_{toe}R$	20843.80	kNm	
Overturning moments	$\Sigma M_{toe}O$	-9689.65	kNm	
Overturning Factor of Safety	O_{FOS}	2.15		= QSMtoe / P SMtoe = = restoring moment / overturning moment
Allowable overturning Factor of Safety	O_{allow}	1.50		
Efficiency	Efficiency	143%		

Sliding Check

Sliding Factor of Safety	SFOS =	6.32	=SV * m / SH
Allowable Sliding Factor of Safety	Sallow =	1.5	

Typical Stilling Basin and Wall Section - Loading Condition: Extreme Seismic

Project No: 5684
Project Name: LPRCA Deer Creek Dam
Designed/Checked By: M D Gilani
Date: 03/05/25

Efficiency	EFF	421%		

Foundation Pressure Check

Total vertical loads	ΣV	3294.50	kN	S V = S Vi + S Vh
Restoring Moment	M_R	20843.80	kNm	
Overturning Moment	M_O	9689.65	kNm	
Distance of resultant of all forces from toe	X´	3.39	m	=(MR - Mo) / S V
Total gravity wall base width	B	7.62	m	
	B/2	3.81	m	X' < B/2 resultant force is near the Toe
	B/6	1.27	m	
Eccentricity , Absolute (positive) value	e	0.42	m	l (B/2 - X') l (absolute value) and e must be less than B/6
Therefore : e > B/6, (not adequate)				
				e < B/6 ? Yes
	Therefore	e<B/6 is adequate		

Standard Bearing Pressure

Note: When the ground reaction falls with in the middle third of the base (e < B / 6), the pressure distribution will be trapezoidal, if (e = B/6) then qa = 0

Maximum pressure at Toe	Q_t	17.41	kPa	(S V / B L) x (1 - 6e / B)
Maximum pressure at Heel	Q_h	34.87	kPa	(S V / B L) x (1 + 6e / B),

Appendix E

Geotechnical Review

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Geotechnical Review - LPRCA Deer Creek Dam, Norfolk County, Ontario

April 17, 2025

Prepared for:
DM Wills Associates Limited

Cambium Reference: 21313-001

CAMBIUM INC.

866.217.7900

cambium-inc.com

Peterborough |Barrie | Oshawa | Kingston

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Figure 1 Site Location Plan
Figure 2 Slope Stability Sections

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Appendix A Borehole Logs (Previous Investigations by Others)
Appendix B Slope Stability Analysis – Results
Appendix C Site Photographs

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1.0 Introduction

Cambium Inc. (Cambium) was retained by DM Wills Associates Limited (Client) to complete a visual slope stability assessment and high-level slope stability analysis for the earthfill embankment slopes at the Deer Creek Dam in Norfolk County, Ontario (Site).

This report presents and summarizes the results of the visual slope assessment, as well as the background review and the methodology, assumptions and results of the slope stability analyses completed by Cambium.

It is noted, as referenced in subsequent sections of this report, that the slope stability analysis completed by Cambium is to be interpreted as a high-level/desktop analysis, based solely on the available background information and results of geotechnical investigations completed at the Site by others as referenced in this report. Cambium has not independently verified the results of the geotechnical investigations completed by others. If Cambium is retained to complete a confirmatory geotechnical investigation at the Site in the future, the results may be added to this report and the slope stability analyses adjusted, if necessary, to reflect current subsurface conditions in the earthfill embankment.

1.1 Reviewed Documents

The following project documents were received and reviewed during the drafting of this report:

[1] Woodstock Engineering Consultants Limited – Woodstock, Ontario
Big Creek Region Conservation Authority – Deer Creek Dam – Deer Creek – Langton, Ontario; Dam – General Layout, Dam – Embankment Typical Section, Boring Logs; Drawing No. 11-R-2, 12-R-3, 43; April 1968; Revision: As Constructed – July 1969.

[2] D.M. Wills Associates Limited – Peterborough, Ontario
5684 – Deer Creek DSR; Hydraulic analysis results, water level elevations for Sunny Day (Normal Operating Conditions) and 100-Year (Inflow Design Flood) conditions, hazard potential classification (HPC) and topographic survey data; received February 7 and 14, 2025.

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[3] William Trow Associates (Hamilton) Limited – Hamilton, Ontario
Deer Creek Dam – Additional Site Investigation and Re-Appraisal of Proposed Design; Project: H296; March 15, 1968.

[4] Vance, Needles, Bergendoff & Smith, Ltd. Consulting Engineers – Woodstock, Ontario
Preliminary Engineering Study and Report; Deer Creek Dam – Deer Creek – Big Creek Region Conservation Authority; Preliminary Engineering Study and Report; August 3, 1967.

[5] Riggs Engineering – London, Ontario
Dam Inspection Report – Norwich, Teeterville, Lehman, Deer Creek, Backus Mill, and Hay Creek Dams; December 2, 2014.

1.2 Standards and Guidelines

Applicable standards, guidelines and other normative documents utilized in preparing geotechnical engineering recommendations for this report are provided below.

[6] Geotechnical Design and Factors of Safety – Technical Bulletin – Ontario Ministry of Natural Resources; August 2011.

[7] Seismic Hazard Criteria, Assessment and Considerations – Technical Bulletin – Ontario Ministry of Natural Resources; August 2011.

[8] Classification and Inflow Design Flood Criteria – Technical Bulletin – Ontario Ministry of Natural Resources; August 2011.

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2.0 Site Description and Existing Conditions

The Deer Creek Dam is located approximately 21 km upstream from Lake Erie on Deer Creek, a tributary of Big Creek, and was originally constructed in 1969 by the Big Creek Region Conservation Authority. The dam came under the ownership of the Long Point Region Conservation Authority (LPRCA) following the merger of the Big Creek and Big Otter Region Conservation Authorities. A Site Location Plan showing the dam location is provided as Figure 1 of this report.

The dam is an earth embankment dam with a total length of ± 160 m, with a concrete spillway/chute structure set into the west abutment of the embankment dam. The concrete spillway/chute, with a width of ± 15 m, is outfitted with two bays of stop logs controlling head pond water levels. Norfolk County Road 45 extends atop the entire length of the dam, with a bridge facilitating passage of the roadway over the spillway section of the dam. A low-level intake is located to the northeast of the spillway, that facilitates the lowering of water levels and provides low flow augmentation to Deer Creek.

The crest of the dam was constructed at an elevation of ± 204.2 m above sea level (masl) [1]. According to as-built drawings of the dam components [1], the downstream slope was constructed at an inclination of ± 3.5H:1V, extending downward from the slope crest to an elevation of ± 191.7 masl, prior to transitioning to significantly flatter inclinations within a toe berm which is understood to have been constructed to provide additional stability to the downstream slope. The toe berm extends from the eastern wall of the spillway to ± 60 m east of the spillway. Near the east abutment of the earth embankment, the downstream slope extends further downward to the top of a concrete outlet structure and stilling basin, which directs water released from the reservoir into a bypass channel that merges into Deer Creek ± 80 m south of the outlet location. The release of water from the reservoir occurs through a concrete conduit/bypass pipe extending beneath the embankment dam. The height of the downstream embankment slope is ± 12.5 m where the toe berm is in place and increases to ± 16.2 m at the outlet structure/stilling basin location. The bed of the downstream bypass channel extending southward to Deer Creek is understood to be at an elevation of ± 187 masl.

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The upstream slope was constructed at an inclination of ± 3H:1V from the slope crest to an elevation of 195.1 masl, and at an inclination of ± 4H:1V from an elevation of 195.1 masl to the toe of the upstream slope, with the two slope sections separated by a bench with a width of ± 3 m [1]. From the bench elevation of 195.1 masl to an elevation of 202.8 masl, the upstream slope was protected with ± 0.6 m of rip rap material underlain by ± 0.3 m of granular filter material. Based on current topographic/bathymetric survey information provided to Cambium by the Client [2], it is understood that the toe of the upstream slope is at an elevation of ± 189 masl.

An inspection of the Deer Creek Dam was completed in 2014 by Riggs Engineering Limited [5], which identified the completion of a dam safety review (DSR) along with comprehensive dam inspections as a 5 to 10-year priority item at the time of the inspection. Cambium was retained to complete the geotechnical review presented in this report as part of the comprehensive dam safety review being undertaken by the Client. Based on Client liaison, it is understood that a full DSR has not been completed to date for the Deer Creek Dam.

2.1 Scope of Work

The scope of work completed by Cambium as part of the current geotechnical review is detailed in Cambium proposal 21313-P, and consists of the following components:

- Visual slope stability assessment to visually evaluate the stability and condition of the earth embankment slopes, including photo documentation.
- Background review of publicly available regional geologic mapping, existing geotechnical reports, as well as Client-provided drawings and survey information for the existing earth embankment dam.
- High-level slope stability analysis for one representative section of the earth embankment using industry standard two-dimensional modelling software using soil parameters from existing data (not verified by Cambium), with factors of safety calculated for relevant design scenarios according to Lakes and Rivers Improvement Act (LRIA) criteria.

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The Hazard Potential Classification (HPC) for the Lanark Dam is understood based on Client-provided information [2] to be High for failure under sunny day/normal operating conditions, with a potential loss of life of 1 to 9 persons. The HPC is also understood to be High for failure under inflow design flood (IDF) conditions, with a potential loss of life of 3 persons.

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3.0 Background Review

A background review of subsurface information obtained during borehole investigations conducted by others at the Site [3], [4] was completed to assist in modelling subsurface conditions for high-level slope stability analysis.

3.1 Regional Geology

Quaternary geologic mapping (Ontario Geological Survey, or OGS, 2000) maps the Deer Creek Dam within a formation of glaciomarine deposits consisting of sand, gravelly sand and gravel nearshore and beach deposits. A formation of glaciomarine deposits consisting of silt and clay, minor sand basin, and quiet water deposits is projected approximately 1 to 2 km east of the dam location. Physiography of Southern Ontario mapping (Chapman, L.J. and Putnam, D.F., 2007) indicates the Site within a formation of sand plains. OGS drift thickness mapping indicates overburden thicknesses (i.e., depth to bedrock) in the order of 70 to 80 m in the general vicinity of Deer Creek Dam.

3.2 Previous Geotechnical Investigations

Cambium conducted a factual review of the results of two geotechnical investigations completed in 1967 and 1968 by others prior to construction of the embankment dam, as presented in the investigation reports ([3], [4]). It is noted that no post-construction geotechnical data was available for Cambium to review.

During the aforementioned geotechnical investigations, six boreholes were advanced at or near the then-proposed dam location, as summarized in Table 1. The locations of the boreholes advanced at or near the dam location by others are shown on figures presented in the respective investigation reports ([3], [4]) and are also indicated on the as-built drawing showing a plan view of the embankment and spillway area (Drawing No. 5-R-5) [1].

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Table 1 Borehole Summary (Previous Investigations)

BH ID	Location	Ground Elevation (masl)	Termination Depth (m)	Termination Elevation (masl)
Borehole Investigation – 1967 (Vance, Needles, Bergendoff and Smith Ltd.)				
BH1	Valley bottom, upstream of proposed dam	188.2	15.4	172.8
BH2	West abutment area, on proposed roadway alignment	205.5	24.5	181.0
BH3	East abutment area, on proposed roadway alignment	211.5	27.9	183.6
Borehole Investigation – 1968 (William Trow Associates)				
BH4	Valley bottom, downstream of proposed dam	189.0	9.1	179.9
BH5	West abutment area, existing slope west of proposed spillway/chute	206.0	21.0	185.0
BH6	Within proposed reservoir area upstream of proposed dam	210.3	11.9	198.4

The borehole logs from the previous investigations detailed above are provided in Appendix A of this report for reference.

The subsurface conditions encountered in the boreholes advanced at the Site in 1967 and 1968 are summarized in the subsequent Sections of this report to the degree necessary to support modelling for slope stability analysis. It is noted that the following Sections of this report do not intend to replace prior geotechnical reporting, as such, for a detailed presentation of the encountered subsurface conditions, the investigation reports ([3], [4]) should be consulted.

3.2.1 Embankment Fill

Given that the borehole investigations conducted in 1967 and 1968 took place prior to the construction of the existing dam, no known boreholes have been advanced into or through the existing earth embankment to confirm the constructed soil conditions. Furthermore, Cambium has not been made aware of any significant maintenance or construction works that have occurred within the embankment since the original geotechnical investigations and subsequent

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construction of the dam. As such, the composition of the earth embankment dam is inferred based on the as-built drawings of the earth embankment dam [1] made available to Cambium.

According to the provided as-built drawings, the earth embankment dam was constructed as a zoned embankment dam with an impervious core, surrounded by a pervious shell on both the upstream and downstream sides, and including a granular filter with toe drains on the downstream side. The upstream slope surface, as mentioned above, was surfaced with ± 0.6 m of rip rap material between elevations of ± 195.1 and 202.8 masl, underlain by ± 0.3 m of granular filter blanket material. The granular filter blanket was extended downward along the upstream slope surface to an elevation of ± 193.5 masl. It is understood based on the provided as-built drawings that the downstream slope surface was finished with approximately 50 mm of topsoil and sodding.

Based on the provided as-built drawings [1], a sheet pile cutoff wall was constructed as an alternative to extending the impervious core downward to key into native soils of low permeability. The sheet pile elements are understood to have been advanced through a surficial deposit of more permeable native soils and embedded into native soils of lower permeability. The native soil formations encountered at the Site are described in further detail below.

3.2.2 Foundation Soils

According to the provided as-built drawings [1] and accompanying reports ([3], [4]) it is understood that the embankment dam was founded directly on native soils. Based on the results of the borehole investigations completed prior to construction of the existing dam, it is understood that the native foundation soils below the dam initially consist of a layer of non-cohesive alluvium (sand and silt, etc.) with a thickness of up to ± 3 m relative to original grades prior to construction of the dam. The alluvium soils were found to be of loose relative density during the borehole investigations and are understood to have been compacted in place prior to placing the first layers of embankment fill.

The alluvium soils are underlain by a clayey silt to silty clay, which was determined during the investigation works to be of stiff to hard consistency and generally extended to the termination

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depth of boreholes BH1 and BH4, which were advanced within the valley bottom on the upstream and downstream sides of the proposed dam, respectively. The aforementioned sheet pile cutoff is understood to have been installed through the surficial alluvium deposit, which was inferred to be of higher permeability, and embedded into the underlying clayey silt to silty clay.

3.2.3 Bedrock

Bedrock was not encountered in any of the boreholes advanced during the investigations completed prior to construction of the existing dam. The boreholes were terminated at elevations between ± 173 and 198 masl, corresponding to depths between ± 9 and 28 m relative to original grade at the borehole locations.

3.2.4 Groundwater

Current groundwater conditions at/near the dam location are likely to vary seasonally and correlate to water levels on the upstream and downstream sides of the dam. In general, the water level within the earth embankment dam is anticipated to decline throughout the embankment and adjacent natural slopes from the retained water level in the pond upstream of the dam to the downstream tailwater elevation. The water level elevations throughout the earth embankment will generally depend on the earth embankment composition and permeability of the materials used to construct different portions of the embankment.

Given that no monitoring wells are known to have been installed through the crest of the earth embankment, no data is known to be available regarding water level elevations within the earth embankment.

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4.0 Slope Stability Assessment

The slope stability assessment conducted by Cambium consisted of a visual slope stability assessment, conducted at the Site by a qualified geotechnical engineer, and subsequent high-level/desktop slope stability analyses based on the information obtained during the background review detailed in the preceding Sections of this report.

4.1 Visual Slope Stability Assessment

A visual assessment of the earth embankment and abutment areas was conducted by Cambium on October 29, 2024. Photographs taken during the assessment are provided in Appendix C and are referenced throughout this section.

The scope of the visual assessment included the upstream and downstream slopes of the earth embankment dam to the east of the concrete spillway/chute structure, the remaining embankment areas to the west of the spillway/chute structure and a general assessment of the bank slopes of the downstream bypass channel. The visual assessment was limited to areas easily accessible by foot and did not include areas submersed by water.

County Road 45 extends along the crest of the embankment dam, as shown in Photograph 1. General views of the downstream embankment slope to the east of the spillway structure are shown in Photographs 2 to 4, with the stabilizing toe berm visible in Photographs 3 and 4. At the time of the inspection, vegetation on the downstream slope was observed to consist primarily of grass, which appeared to have been maintained and cut prior to the inspection. No seepage was observed in any area of the downstream slope at the time of the inspection, and no signs of instability or distress such as sloughing or tension cracking were observed. Based on the results of the current topographic survey [2] completed by others, the inclination of the downstream slope is approximately 3.5H:1V, which corresponds to the slope inclination according to the as-built drawings [1].

The concrete spillway structure, which was constructed into the west abutment of the embankment dam, is shown in Photographs 5 and 6, with an additional general view of the downstream slope provided in Photograph 7. An assessment of the structural condition of the

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concrete spillway structure was beyond the scope of the visual assessment completed by Cambium, which was limited to the condition of the earth embankment slopes. It is noted that the walls of the spillway/chute structure function as retaining walls for the earthfill material used to construct the earth embankment slopes.

The height of the downstream slope increases by approximately 3 to 4 m at the location of the concrete outlet structure on the east side of the embankment dam. The outlet structure directs water released from the reservoir through a concrete conduit extending beneath the dam into the bypass channel, which extends southward to Deer Creek. Conditions at the concrete outlet structure location are shown in Photograph 8. The transition from the stabilizing toe berm, which is present at the toe of the downstream slope for the majority of the dam alignment, to the top of the outlet structure, which effects an increase of the total embankment height at the outlet structure location, is shown in Photograph 9.

Photographs 10 to 12 illustrate conditions within the bypass channel and demonstrate the extent of toe erosion along both sides of the bypass channel. Toe erosion and related sloughing (Photograph 12) were noted at various locations along the bypass channel between the concrete outlet structure and termination of the bypass channel at Deer Creek.

Conditions on the downstream side of the east abutment are shown in Photograph 13. Abutment conditions in this area appeared to be normal, with the embankment dam transitioning to a natural slope area that existed prior to the dam’s construction, which was densely vegetated with mature trees and brush.

On the west side of the spillway/chute structure (west abutment area), a steeper roadway embankment and ditch area exist, with the ditch directing collected drainage water into the spillway structure through an outlet in the spillway structure wall (Photographs 14 and 15). Based on the current topographic survey information [2], the height of the roadway embankment slope is in the order of 1 to 2 m on the downstream side, with a slope inclination of ± 2H:1V on average. The downstream roadway embankment transitions to (initially) flatter inclinations within the natural slope area to the south of the dam/roadway alignment, and it is inferred that dam construction works in this area were limited to minor grading efforts required

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to construct the roadway embankment and to match existing grades within the adjacent natural slope area to the south (Photograph 16). No seepage was observed in this area from the roadway embankment slope surface, and no signs of instability or distress such as sloughing or tension cracking were noted at the time of the inspection.

Conditions on the upstream slope are shown in Photographs 17 to 28. Conditions at the east abutment on the upstream side are shown in Photograph 17 and appeared to be normal, without any signs of instability noted at the time of the inspection.

Typical views of conditions on the upstream slope surface between the east abutment and the concrete lift control chamber, which was constructed into the upstream slope, are shown in Photographs 18 to 21. In general, rip rap protection appeared to be intact in this area and the slope inclination was observed to be relatively constant, with no notable sloughing observed. It is apparent that some displacement of rip rap material has occurred over time, and the rip rap material was observed to be inconsistent in size at some locations, with particle sizes sometimes ranging from gravel material to larger boulders within the same general area of the slope (Photograph 19). Vegetation consisted of dense brush and high grass both above the rip rap material and along the water's edge, with vegetation also growing within the rip rap material at certain locations. Based on the current topographic survey [2], the slope inclinations on the upstream slope generally appear to correspond to as-built inclinations in this area [1].

The top of the lift control chamber is shown in Photograph 22, and conditions within the remainder of the upstream slope between the lift control chamber and the spillway structure are shown in Photographs 23 to 28. In this area, some inconsistent slope inclinations were observed, which could be indicative of some localized sloughing of the upstream slope (Photographs 23 and 24). It is noted that given the dense coverage of the slope surface with vegetation and rip rap of variable size, a thorough and conclusive evaluation of the extent and potential causes of any changes in slope inclinations due to sloughing was not possible during the inspection conducted by Cambium. Sloughing in this area and inconsistent slope inclinations were noted during the visual assessment conducted in 2014 [5] by others, and can also be noted on the current topographic survey completed by others [2], which appears to show steeper slope inclinations of ± 2H:1V between elevations of 201 and 203 masl, and again

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between elevations of 198 and 200 masl. The inconsistent slope inclinations within the upstream slope as described above appear to extend 60 to 70 m eastward from the east wall of the concrete spillway structure based on the current topographic survey information [2], and as such, these conditions persist along a significant portion of the upstream slope. Based on verbal information provided by the Client during the inspection, it is understood that conditions in the affected area of the upstream slope have been stable since the inconsistent slope inclinations were first noted, based on continual visual observations being made by the dam owner/operator. The frequency of visual observations of the upstream slope area currently being conducted by the dam owner is not known to Cambium, and it is not known whether observations are taking place on a regular schedule and whether formal direction has been given to regularly inspect the upstream slope surface for signs of sloughing and/or additional changes on the upstream slope surface. It is noted that based on the provided topographic survey information, the average slope inclination in this area, taken from 198 to 203 masl, is estimated to be in the order of 2.7H:1V to 2.9H:1V throughout the affected area.

Vegetation on the upstream slope was dense, primarily consisting of high grass and dense brush, sometimes ranging up to mature trees near the spillway structure (Photograph 26). Some large boulders, as shown in Photograph 27, were noted within the rip rap material at surface.

4.2 Desktop Slope Stability Analysis

4.2.1 General

The slope stability analysis was performed using Slope/W, an industry standard, two-dimensional analysis software. The employed analysis methodology is the Morgenstern-Price method of slices, where the factor of safety is determined for a number of geometrically feasible slip surfaces. The slip surface with the lowest factor of safety is determinative for the respective load case. Detailed analysis results, including the determinative slip surface and corresponding factor of safety for each section and loading case, are presented in Appendix B.

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4.2.2 Geometry

The geometry of the analyzed sections was generally developed according to the as-built drawings of the earth embankment made available to Cambium [1], which include one typical section and one additional section to support a sensitivity analysis at the conduit/outlet structure location, due to the localized increase in the total height of the embankment at that location.

The composition of the earth embankment, including the impervious core, pervious shells and granular filter, was adopted according to the as-built drawings [1], which additionally indicate approximate thicknesses of the native foundation materials within the typical section. The sections developed using the as-built drawings were corroborated and checked for plausibility using updated topographic/bathymetric survey data supplied by the Client [2]. It is noted that the inconsistent slope inclinations noted within a significant portion of the the upstream slope, as described in detail above, were not explicitly modelled in the current analyses, given that the current average slope inclination in the affected area remains comparable to the as-built slope inclination of 3H:1V.

In accordance with Cambium proposal 21313-P, one section of the embankment dam was modelled, designated as Section 1-1. Given that the height of the embankment dam increases at the conduit/outlet structure location, a sensitivity analysis was conducted for the downstream slope by extending the downstream slope toe further downward to the top elevation of the outlet structure. This section was designated as Section 1A-1A and assumes that the composition of the embankment dam is the same at the conduit/outlet structure location as in the typical section. The section locations are shown on the appended Figure 2.

The general plausibility of the section geometry was confirmed by a review of the updated topographic and bathymetric survey information provided to Cambium by the Client [2]. A summary of the section geometries is provided in Table 2 below.

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Table 2 Embankment Section Geometry for Analysis

Parameter	Section 1-1	Section 1A-1A
Corresponding As-Built Section [1]	Typical Section	Section – Concrete Outlet/Bypass Channel Location
Slope Crest Elevation	204.2	204.2
Slope Toe Elevation – Downstream	191.7 (top of stabilizing toe berm)	187.9 (top of outlet structure for concrete conduit)
Slope Toe Elevation - Upstream	189.0	189.0
Slope Height – Downstream	12.5	16.3
Slope Height – Upstream	15.2	15.2
Slope Inclination – Downstream	3.5H:1V	3.5H:1V
Slope Inclination – Upstream (As-Built)	3H:1V (above 195.1 masl) 4H:1V (below 195.1 mas)	3H:1V (above 195.1 masl) 4H:1V (below 195.1 masl)

4.2.3 Subsurface Stratigraphy

The composition of the earth embankment, as well as subsurface conditions below the embankment dam, were adopted as indicated on the as-built section drawings [1]. It is noted that the foundation soil conditions noted on the as-built section drawings qualitatively reflect the results of the 1967 and 1968 geotechnical investigations (i.e., loose alluvium soils overlying less permeable, clayey silt to silty clay soils). It is reiterated that neither the composition of the earth embankment nor the subsurface conditions below the dam have been confirmed by Cambium based on a current subsurface investigation, and therefore the current modelling assumptions represent high-level assumptions only, which rely on the accuracy of the as-built drawings. The recommendations provided in subsequent Sections of this report pertaining to subsurface investigations recommended to confirm the assumptions underpinning the currently presented modelling and analysis should be consulted by the Client and dam owner/operator.

The preliminary soil mechanical parameters used for the purposes of slope stability analysis are presented in Table 3. The parameters are estimated based on Cambium's interpretation of the available geotechnical investigation results and on experience working with similar materials with respect to the embankment dam components (impervious core, pervious shell and granular filter materials).

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Table 3 Preliminary Soil Mechanical Parameters for Analysis

Stratum	Soil Mechanical Parameters		
	Friction Angle φ [°]	Cohesion c [kN/m^2]	Unit Weight γ [kN/m^3]
Impervious Core	25	2	19.0
Pervious Shell	30	0	19.5
Granular Filter	32	0	20.5
Sand and Silt (Alluvium); loose	28	0	19.0
Clayey Silt to Silty Clay; stiff to hard	25	5	19.0

It is noted that the preliminary soil strength parameters provided for the embankment dam components were not provided in the previous geotechnical reports, nor have they been substantiated or confirmed based on a borehole investigation including boreholes advanced through the crest of the earth embankments, as such, the parameters provided in Table 3 represent high-level estimates only and it is recommended, as detailed in the below sections of this report, that these parameters be confirmed in the field by advancing boreholes through the crest of both earth embankments.

4.2.4 Loading Cases

The modelled loading cases/design scenarios reflect the requirements detailed in the LRIA Technical Bulletin on Geotechnical Design and Factors of Safety [6], and are summarized in Table 4 below.

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Table 4 Loading Cases for Analysis

Loading Case ID	Description	Design Minimum Factor of Safety per LRIA Technical Bulletin [6]
LC1	Normal Operating Conditions	1.5
LC2	Pseudostatic Analysis	Greater than 1.0
LC3	Inflow Design Flood (IDF)	1.3
LC4	Rapid Drawdown (upstream only)	1.2 to 1.3 [1]

[1] *The higher factor of safety may be required if drawdown occurs frequently during dam operations.*

The water surface elevations required for analysis, as summarized in Table 5, were provided to Cambium by the Client [2] and are understood to be based on current hydraulic analyses for scenarios affecting the embankment dam. It is noted that the water surface within the earth embankment, between the provided upstream and downstream water surface elevations, was inferred based on engineering judgment, given that no monitoring wells have been installed in the embankments and therefore no data regarding the water surface elevation within the earth embankments is known to be available. Further commentary regarding the installation of monitoring wells in the earth embankments to confirm the assumptions made in the analyses presented in this report is provided in subsequent sections of this report.

Table 5 Water Levels for Analysis

Scenario	Headwater/Upstream Water Surface Elevation (masl)	Tailwater/Downstream Water Surface Elevation (masl)	Applies for Load Cases
Normal (Sunny Day)	200.27	185.85	LC1, LC2, LC4[1]
Inflow Design Flood (IDF)[2]	201.21	187.57	LC3

[1] *LC4 models rapid drawdown from the Normal (Sunny Day) water level in the reservoir.*

[2] *The Client-provided IDF water level elevations are in accordance with Table 2 in [8] for High HPC dams and represent the water surface 1/3 of the distance between the 1000-year flood and PMF (probable maximum flood).*

For the pseudostatic analysis, a Hazard Potential Classification (HPC) of High was assumed based on the information provided to Cambium by the Client [2], and the Normal (Sunny Day) water levels were applied. According to the LRIA Technical Bulletin governing seismic analyses for dams [7], a 1:2,500-year earthquake is the required design scenario for a High HPC dam with a potential loss of life of 10 or fewer persons. The 1:2,500-year peak ground

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acceleration (PGA) value was determined to be **0.177g** based on the 2020 National Building Code Seismic Hazard Calculation (Natural Resources Canada).

The aforementioned PGA value is based on a seismic site classification of **Site Class E – Soft Soil** for the purposes of providing a conservative analysis, given that some loose soils were encountered below the proposed dam footprint at the time of the investigation, and that the relative density/consistency of the soils within the embankment dam itself has not been confirmed by means of a borehole investigation or other field methods.

4.2.5 Analysis Results

The results of the slope stability analyses are summarized in Table 6 for the typical dam section designated as Section 1-1, along with the design minimum factors of safety according to LRIA criteria. The results of the sensitivity analysis conducted for the highest section of the dam at the location of the conduit/outlet structure, designated as Section 1A-1A, are provided in

Table 7. It is noted that the sensitivity analysis was conducted for the downstream slope only, given that the geometry of the upstream slope is assumed to be consistent throughout the embankment dam alignment.

Table 6 Stability Analysis Results – Section 1-1 – Typical Section

Loading Case ID	Design Minimum Factor of Safety per LRIA Technical Bulletin [6]	Calculated Factor of Safety	Figure (Appendix B)
LC1 – Normal Operating Conditions (Downstream Slope)	1.5	2.1	B.1
LC2 – Pseudostatic Analysis (Downstream Slope)	Greater than 1.0	1.3	B.2
LC3 – IDF Condition (Downstream Slope)	1.3	2.1	B.3
LC1 – Normal Operating Conditions (Upstream Slope)	1.5	1.6	B.4
LC2 – Pseudostatic Analysis	Greater than 1.0	1.0	B.5

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Loading Case ID	Design Minimum Factor of Safety per LRIA Technical Bulletin [6]	Calculated Factor of Safety	Figure (Appendix B)
(Upstream Slope)			
LC3 – IDF Condition (Upstream Slope)	1.3	1.6	B.6
LC4 – Rapid Drawdown (Upstream Slope)	1.2 to 1.3	**0.9**	B.7

Table 7 Stability Analysis Results – Section 1A-1A – Conduit/Outlet Structure

Loading Case ID	Design Minimum Factor of Safety per LRIA Technical Bulletin [6]	Calculated Factor of Safety	Figure (Appendix B)
LC1 – Normal Operating Conditions (Downstream Slope)	1.5	1.8	B.8
LC2 – Pseudostatic Analysis (Downstream Slope)	Greater than 1.0	1.2	B.9
LC3 – IDF Condition (Downstream Slope)	1.3	1.7	B.10

As such, the LRIA design minimum factors of safety are anticipated to be met by the upstream and downstream embankment slopes under normal operating conditions, pseudostatic conditions and under IDF conditions. The design minimum factors of safety for the aforementioned loading cases were also met by the locally higher section of the downstream slope at the outlet structure location, based on the sensitivity analysis conducted by Cambium.

The upstream embankment slope does not fulfil LRIA design minimum factor of safety criteria under the rapid drawdown condition, where a factor of safety of 0.9 was calculated based on a drawdown from the normal operating water level in the reservoir to the upstream slope toe elevation of 189 masl. The design minimum factor of safety for the rapid drawdown condition is 1.2 where rapid drawdown is evaluated to occur infrequently, and 1.3 where rapid drawdown is evaluated to occur frequently within regular dam operations. Given that the factor of safety is below 1.0, the analysis indicates likely failure of the upstream embankment slope following a

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rapid drawdown of the water level in the reservoir from the normal operating (sunny day) water level to the toe of the upstream slope.

4.2.6 Discussion and Recommendations

As previously noted, the current slope stability analysis constitutes a high-level desktop analysis based on subsurface information obtained during borehole investigations conducted by others before construction of the existing embankment dam [4], as-built drawings of the earth embankment [3] and preliminary soil mechanical parameters adopted on a high-level basis by Cambium. Furthermore, Cambium has not been made aware of any significant maintenance or construction works that have occurred within the dam embankment since the aforementioned borehole investigations.

The subsurface conditions within the earthfill embankment, between the embankment crest and underlying native foundation material, have a significant influence on the analysis results (i.e., factors of safety) presented in this report. These conditions can be confirmed by advancing boreholes through the crest of the embankment dam, which is recommended to confirm the results of the analysis completed by Cambium and derived recommendations, particularly given that the Deer Creek Dam has been evaluated to have a High HPC. Consideration should be given to installing monitoring wells in the earth fill embankment in boreholes advanced from the crest and from the top of the stabilizing toe berm on the downstream side, to allow for the collection of water level data from the embankment dam and adjacent areas which can be utilized to further refine slope stability models. It is noted that in the absence of borehole investigation results confirming the outstanding subsurface information for the earth embankments, the analysis presented in this report should not be construed as providing final factors of safety for the analyzed embankments and should solely be interpreted as providing high-level estimates of the factors of safety for the required design scenarios.

In general, the downstream embankment slope satisfied the design minimum factors of safety for all LRIA design scenarios and appears on a high-level basis to be in stable condition, with no remedial works required. It is generally recommended that vegetation/grass on the

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downstream slope continue to be maintained. If any works are proposed within or near the downstream slope toe that would require excavations into the slope (i.e., any replacement of the concrete outlet structure or related works, works proposed along the spillway structure walls, etc.), a geotechnical engineer should be consulted to provide guidance during the design and construction phases.

The upstream embankment slopes did not meet LRIA criteria under the rapid drawdown condition, with the calculated factor of safety indicating a potential failure scenario. Though the cause of the inconsistent slope inclinations and potential sloughing on the upstream slope noted during the inspection of the dam conducted by others in 2014, the topographic survey works conducted by others as a part of the current dam safety review, and during the inspection conducted by Cambium, cannot be conclusively determined based on information available to date, it is possible that surficial sloughing may have occurred in conjunction with previous drawdown operations. It is recommended that available data and records regarding drawdown operations in the dam, particularly related to drawdown frequency and the extent and duration of drawdown during each event, be consulted and evaluated further to determine whether particular drawdown events can be correlated to instability observed within the upstream slope.

It is recommended that a remediation of the upstream slope be undertaken to regrade areas in which instability/sloughing has occurred to a constant, stable slope inclination. The stable slope inclination should be determined based on slope stability analysis and will be shallower than the as-built inclination of 3H:1V in the upper portion of the upstream slope, which returned a factor of safety of less than 1.0. Slope stability analysis conducted in the future to support dam remediation works should be based on the results of a borehole investigation completed according to the recommendations provided above, which will also provide information required to develop construction recommendations for the proposed remediation works. In conjunction with regrading works, it is recommended that the existing rip rap protection be remediated, such that a constant stable slope inclination is provided in all areas of the upstream slope, and so that all areas of the upstream slope are provided a consistent level of protection throughout. The rip rap remediation will require removal and appropriate re-

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placement of any larger boulders that have migrated downslope towards the water's edge. It is recommended, regardless of the schedule determined for the remediation/regrading works described above, that larger vegetation (i.e., mature trees, dense shrubs, high grass, etc.) growing within, above and below the rip rap material on the upstream slope be removed in the short term and that such periodic vegetation removal be regularly undertaken by the dam owner/operator as a part of ongoing and regular maintenance protocols. A geotechnical engineer should be consulted during the planning phase for any significant works within the embankment slope areas, including tree/vegetation removal and related excavations, and should be retained to supervise the proposed works during construction and inspect the works upon completion, as deemed necessary by the geotechnical engineer during the planning phase.

In light of the factor of safety determined for the rapid drawdown condition, it is not recommended that a rapid drawdown be induced in the reservoir without providing appropriate measures to mitigate a potential slope failure. Mitigation measures can include developing a protocol to ensure that rapid drawdown occurs over a sufficiently long period of time to allow for pore water pressures within the embankment to dissipate in conjunction with the drawdown of the water level in the reservoir. Additional borehole investigation works to determine the nature and permeability of the embankment fill material are recommended to provide a basis for developing this type of mitigation strategy, and installation of monitoring wells in the embankment could provide a basis for defining an acceptable rate of drawdown (i.e., by stipulating that water level measurements in the wells be conducted in conjunction with any drawdown in the reservoir). If rapid drawdown is anticipated to occur in the reservoir with little or no control of the rate of drawdown being possible, it is recommended to initiate remediation measures in the near term, given that rapid drawdown may cause successive failure of the upstream slope, which even if surficial may progress sufficiently to affect the ability of the dam to perform the engineering function of retaining water in the reservoir. If required based on confirmatory analyses as described above, further mitigation measures could include flattening of the embankment slopes, or installation of stabilizing toe berms along the upstream embankment toe to reduce the overall slope height.

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It is noted that in accordance with the LRIA Technical Guide [6], more sophisticated methods of analysis to evaluate dam behaviour and factors of safety under pseudostatic conditions may be warranted for High or Very High HPC dams, which applies for the Deer Creek Dam. In particular, given the sand-dominant foundation soils that are likely to be saturated in most if not all areas of the dam alignment, it is recommended that the potential for liquefaction during an earthquake event be further evaluated. A modelling of liquefaction failure and determination of the factor of safety against liquefaction failure are beyond the scope of the current analysis, furthermore, it is recommended that foundation soil conditions below the dam be confirmed by a borehole investigation, with boreholes being advanced through the dam crest at a number of locations along the dam alignment. It is recommended that borehole investigations at the dam location be scoped to include continuous sampling to accurately capture the transition from the embankment fill material to the underlying native soils, and to allow for samples of the native soils to be submitted for particle size distribution testing so that the composition of the native sand-dominant soils can be assessed with respect to liquefaction risks.

It is recommended that a CCTV inspection of the concrete conduit/bypass pipe, which extends beneath the dam and allows the release of water from the reservoir into the bypass channel to Deer Creek, be conducted in the near term by others to confirm the condition of the bypass pipe, given that any leakages within the pipe can be a cause of successive internal erosion of the dam and if present, could represent a potential failure mechanism for the dam. If the CCTV inspection indicates any potential or confirmed damage or leakages along the bypass pipe alignment, Cambium should be informed and retained to evaluate the inspection results from a geotechnical perspective and to provide commentary and recommendations based on the observations made. It is understood that a CCTV inspection of the bypass pipe will be recommended as part of the DSR being completed by the Client.

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5.0 Report Limitations

5.1 Design Review and Inspections

Cambium should be contacted to review and approve design drawings, prior to tendering or commencing construction, to ensure that all pertinent geotechnical-related factors have been addressed. It is important that onsite geotechnical supervision be provided at this site for excavation and backfill procedures, deleterious soil removal, subgrade inspections and compaction testing.

5.2 Changes in Site and Project Scope

This geotechnical engineering report is intended for planning and design purposes only.

Subsurface conditions can be altered by the passage of sufficient time, natural occurrences, and human intervention. In particular, consideration should be given to contractual responsibilities as they relate to control of groundwater seepage, disturbance of soils, and frost protection.

The design parameters provided, and the engineering advice offered in this report are intended for use by the owner and its retained design consultants. If there are changes to the project scope and development features, these interpretations made of the subsurface information, for geotechnical design parameters, advice, and comments relating to constructability issues and quality control may not be complete for the project. Cambium should be retained to conduct further review to interpret the implications of such changes with respect to this report.

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6.0 Closing

We trust that the information contained in this report meets your current requirements. If you have questions or comments regarding this document, please do not hesitate to contact the undersigned at (705) 719-0700.

Respectfully submitted,

Cambium Inc.

DocuSigned by:
D29A94473BBC4E1...

Josef Schweighofer, M.Sc., P.Eng.
Project Manager - Geotechnical

Signed by:
E2EEE82F32D347D...

Rob Gethin, P.Eng.
Group Manager – Geotechnical

DS

LICENSED PROFESSIONAL ENGINEER
R. L. GETHIN
100196175
PROVINCE OF ONTARIO

RLG/js

2025-04-17

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7.0 Standard Limitations

Limited Warranty

In performing work on behalf of a client, Cambium relies on its client to provide instructions on the scope of its retainer, and, on that basis, Cambium determines the precise nature of the work to be performed. Cambium undertakes all work in accordance with applicable accepted industry practices and standards. Unless required under local laws, other than as expressly stated herein, no other warranties or conditions, either expressed or implied, are made regarding the services, work or reports provided.

Reliance on Materials and Information

The findings and results presented in reports prepared by Cambium are based on the materials and information provided by the client to Cambium and on the facts, conditions and circumstances encountered by Cambium during the performance of the work requested by the client. In formulating its findings and results into a report, Cambium assumes that the information and materials provided by the client or obtained by Cambium from the client or otherwise are factual, accurate and represent a true depiction of the circumstances that exist. Cambium relies on its client to inform Cambium if there are changes to any such information and materials. Cambium does not review, analyze, or attempt to verify the accuracy or completeness of the information or materials provided, or circumstances encountered, other than in accordance with applicable accepted industry practice. Cambium will not be responsible for matters arising from incomplete, incorrect, or misleading information or from facts or circumstances that are not fully disclosed to or that are concealed from Cambium during the provision of services, work or reports.

Facts, conditions, information, and circumstances may vary with time and locations and Cambium's work is based on a review of such matters as they existed at the particular time and location indicated in its reports. No assurance is made by Cambium that the facts, conditions, information, circumstances, or any underlying assumptions made by Cambium in connection with the work performed will not change after the work is completed and a report is submitted. If any such changes occur or additional information is obtained, Cambium should be advised and requested to consider if the changes or additional information affect its findings or results.

When preparing reports, Cambium considers applicable legislation, regulations, governmental guidelines, and policies to the extent they are within its knowledge, but Cambium is not qualified to advise with respect to legal matters. The presentation of information regarding applicable legislation, regulations, governmental guidelines, and policies is for information only and is not intended to and should not be interpreted as constituting a legal opinion concerning the work completed or conditions outlined in a report. All legal matters should be reviewed and considered by an appropriately qualified legal practitioner.

Site Assessments

A site assessment is created using data and information collected during the investigation of a site and based on conditions encountered at the time and particular locations at which fieldwork is conducted. The information, sample results and data collected represent the conditions only at the specific times at which and at those specific locations from which the information, samples and data were obtained and the information, sample results and data may vary at other locations and times. To the extent that Cambium's work or report considers any locations or times other than those from which information, sample results and data was specifically received, the work or report is based on a reasonable extrapolation from such information, sample results and data but the actual conditions encountered may vary from those extrapolations.

Only conditions at the site and locations chosen for study by the client are evaluated; no adjacent or other properties are evaluated unless specifically requested by the client. Any physical or other aspects of the site chosen for study by the client, or any other matter not specifically addressed in a report prepared by Cambium, are beyond the scope of the work performed by Cambium and such matters have not been investigated or addressed.

Reliance

Cambium's services, work and reports may be relied on by the client and its corporate directors and officers, employees, and professional advisors. Cambium is not responsible for the use of its work or reports by any other party, or for the reliance on, or for any decision which is made by any party using the services or work performed by or a report prepared by Cambium without Cambium's express written consent. Any party that relies on services or work performed by Cambium or a report prepared by Cambium without Cambium's express written consent, does so at its own risk. No report of Cambium may be disclosed or referred to in any public document without Cambium's express prior written consent. Cambium specifically disclaims any liability or responsibility to any such party for any loss, damage, expense, fine, penalty or other such thing which may arise or result from the use of any information, recommendation or other matter arising from the services, work or reports provided by Cambium.

Limitation of Liability

Potential liability to the client arising out of the report is limited to the amount of Cambium's professional liability insurance coverage. Cambium shall only be liable for direct damages to the extent caused by Cambium's negligence and/or breach of contract. Cambium shall not be liable for consequential damages.

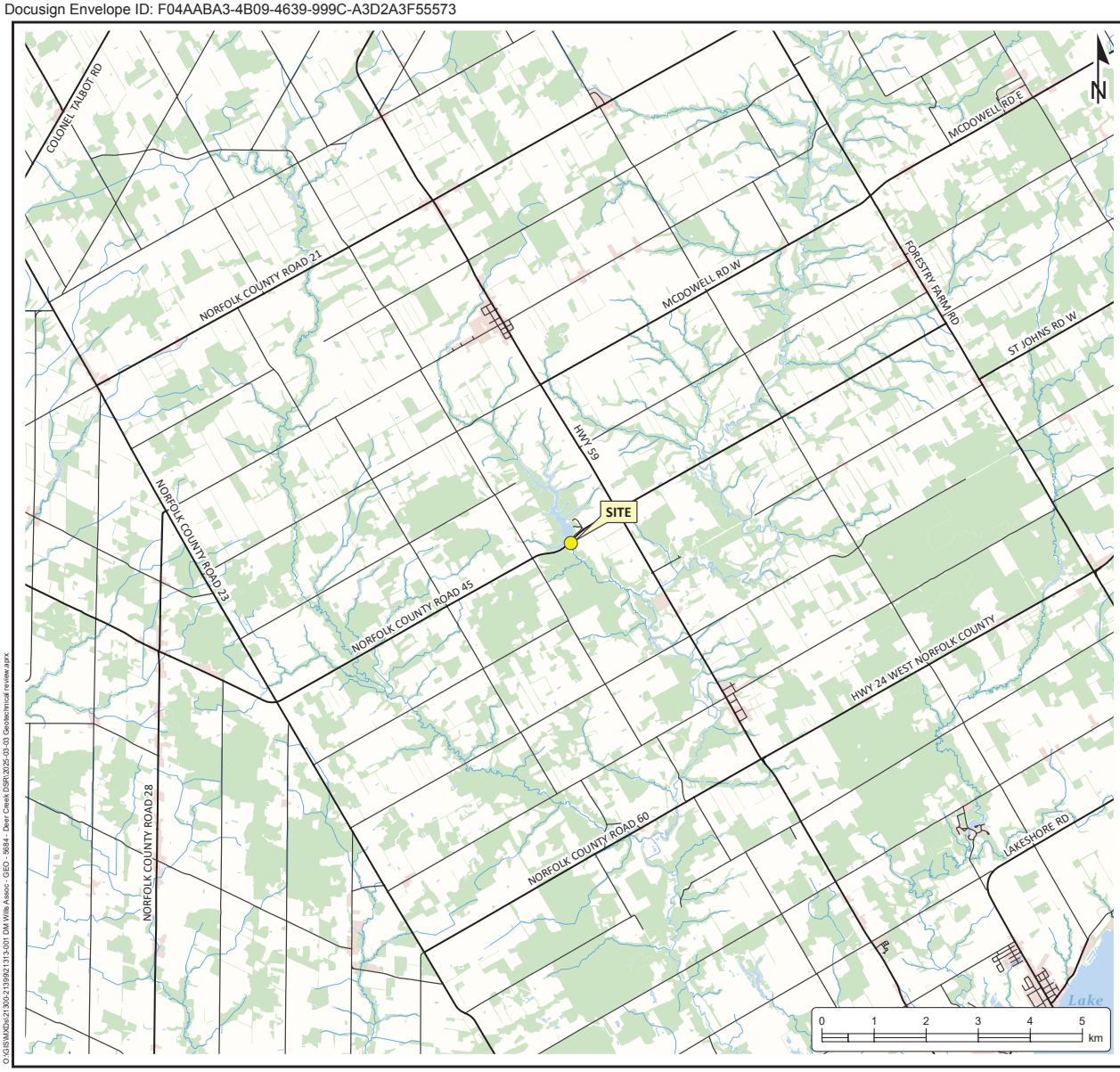
Personal Liability

The client expressly agrees that Cambium employees shall have no personal liability to the client with respect to a claim, whether in contract, tort and/or other cause of action in law. Furthermore, the client agrees that it will bring no proceedings nor take any action in any court of law against Cambium employees in their personal capacity.

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Appended Figures

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GEOTECHNICAL REVIEW
LPRCA DEER CREEK DAM

DM WILLS ASSOCIATES LIMITED
Norfolk County Road 45
Langton, Ontario

LEGEND

- Major Road
- Minor Road
- Watercourse
- Water Area
- Wooded Area
- Built Up Area

Notes:
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- Distances on this plan are in metres and can be converted to feet by dividing by 0.3048.
- Cambium Inc. makes every effort to ensure this map is free from errors but cannot be held responsible for any damages due to error or omissions. This map should not be used for navigation or legal purposes. It is intended for general reference use only.

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www.cambium-inc.com

SITE LOCATION PLAN

Project No.: 21313-001		Date: March 2025	Rev.:
Scale: 1:100,000		Projection: NAD 1983 UTM Zone 17N	
Created by: DBC	Checked by: JS	Figure: 1	

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GEOTECHNICAL REVIEW
LPRCA DEER CREEK DAM

DM WILLS ASSOCIATES LIMITED
Norfolk County Road 45
Langton, Ontario

LEGEND

Slope Stability Section

Notes:
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SLOPE STABILITY SECTIONS

Project No.: 21313-001		Date: March 2025	
		Rev.:	
Scale: 1:1,000		Projection: NAD 1983 UTM Zone 17N	
Created by: DBC	Checked by: JS	Figure:	2

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Appendix A

Borehole Logs (Previous Investigations by Others)

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WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 2

PROJECT No. H296

BOREHOLE No. 1 (V.N.B. & S.)
PROJECT Deer Creek Dam
LOCATION nr. Langton, Ontario
HOLE LOCATION see Dwg. 1
HOLE ELEVATION 617.6
DATUM Geodetic

LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
2" DIA. CONE
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
UNCONFINED COMPRESSION
VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT
SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET
		617.6	0
	1' Topsoil SAND & SILT (ALLUVIUM) - loose, brown, layered, frequently clayey, some wood fragments & organics	607.6	10
	SILT - v.stiff, wet, grey, clayey to ~18ft. with rare thin c. silt or f. sand seam non- or only slightly cohesive below 18ft., layered, some thin pure silt or v. fine sand partings. Frequent slump texture.	573.5	20 30
	SILTY CLAY- cohesive or clayey silt to silty clay, occasional non-cohesive silt layers or seams		40
	Terminated	567.1	50
			60 70 80 90 100 110

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT.
20 40 60 80
SHEAR STRENGTH P.S.F

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS
% DRY WEIGHT
10 20 30

SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
1	
2	
3	135.0
4	133.0
5	143.0
6	
7	
8	138.0
9	128.0
10	129.0
11	129.0
12	
13	

Notes:
1. Hole advanced cased by washboring techniques, for Vance Needles, Bergendoff & Smith.
2. W.L. at ground surface at time of field investigation (May 1967)
3. Piezometer installed at 50ft.

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WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No 3

PROJECT No H296

BOREHOLE No 2 (V.N.B. & S.)
PROJECT Deer Creek Dam
LOCATION nr. Langton, Ontario
HOLE LOCATION see Dwg. 1
HOLE ELEVATION 674.3
DATUM Geodetic

LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
2" DIA. CONE
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
UNCONFINED COMPRESSION
VANE TEST AND SENSITIVITY (S_t)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT
SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET
		674.3	0
	2' Topsoil SAND - layered, fine sand, some coarse silt	668.3	
	CLAYEY SILT (TILL) - brownish to grey layered clayey silt to silty clay. Rare v. fine sand seam, some areas with till-like texture. Layered areas frequently slump-textured.	651.5	10 20
	SILT - v. dense, grey, predominantly non-cohesive but with occasional clayey seam. Pure silt seams frequently dry.	639.5	30
	CLAYEY SILT - layered silt to silty clay. Some pure silt partings, with trace v. fine sand	618.0	40 50
	SILT - predominantly non-cohesive, some seams and partings with v.fine sand, rare cohesive zone exhibits fine layering (varving)		60 70
	Terminated	593.8	80

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT
20 40 60 80
SHEAR STRENGTH P.S.F

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT
10 20 30

SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
1	
2	
3	132.0
4	135.0
5	134.0
6	
7	
8	
9	
10	
11	
12	
13	
14	
15	
16	
17	

Notes:
1. Hole advanced cased by washboring for Vance Needles Bergendoff & Smith
2. Samples re-identified after 7 months by Wm. Trow Associates (Hamilton) Ltd.
3. W.L. recorded.

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WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 4

PROJECT No. H296

BOREHOLE No. 3 (V.N.B & S)
PROJECT Deer Creek Dam
LOCATION nr. Langton, Ontario
HOLE LOCATION see Dwg. 1
HOLE ELEVATION 693.9
DATUM Geodetic

LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
2" DIA. CONE
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
UNCONFINED COMPRESSION
VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT
SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET
		693.9	0
	2' Topsoil	692.0	
	SAND - fine, light brown, v. dense		10
			20
		672.0	
	SILT - v. dense, grey, layered, cohesive in part with some silty clay layers		30
			40
		646.0	
	SILTY CLAY - grey, some silt layers or partings, v. stiff		50
		633.0	60
	SILT - non-cohesive, some very fine sand, v. dense, wet. Thin clay layers and slump texture at 70-78ft.		70
			80
			90
	Terminated	602.4	
			100
			110

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT. (20, 40, 60, 80)
SHEAR STRENGTH P.S.F

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT (10, 20, 30)

SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
1	
2	
3	
4	
5	
6	130
7	137
8	135
9	
10	
11	
12	
13	128
14	
15	
16	
17	
18	
19	

Notes:
1. Hole advanced cased by washboring techniques for Vance Needles, Bergendoff & Smith.
2. Final water level not recorded before hole grouted & capped.
3. Above soil details based on re-examination of samples by Wm. Trow Associates (Hamilton) Ltd.

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WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 5

PROJECT No. H296

BOREHOLE No. 4
PROJECT Deer Creek Dam
LOCATION Nr. Langton, Ontario
HOLE LOCATION see Dwg. 1
HOLE ELEVATION 620.0 approx.
DATUM Geodetic (interpolated from contour plan)

LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
2" DIA. CONE
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
UNCONFINED COMPRESSION
VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^LI
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT
SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET
		620.0	0
	ALLUVIUM - greyish brown to grey layered fine sand, cohesive sand, fine sandy silt and trace silty clay ▽	615.0	
		613.5	
	SILT - grey, dense or very stiff, layered silt, clayey silt, sandy silt or more rarely fine very silty sand. Wet cohesive sand layers with pea gravel at 14ft.3ins, 15ft.10ins., 17ft.3ins. and 18ft.10ins.		10
			20
	Terminated	590.0	30
			40

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT. (20, 40, 60, 80)
SHEAR STRENGTH P.S.F

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

SAMPLE TYPE AND No.

NATURAL UNIT WEIGHT P.C.F.

Notes:

1. Hole augered and uncased. Sampled continuously from 3.1/2 to 30ft.
2. W.L. at 5ft. shortly after completion No subsequent change

Borehole 4

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WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 6
PROJECT No. H296

BOREHOLE No. 5
PROJECT Deer Creek Dam
LOCATION nr. Langton, Ontario
HOLE LOCATION West Abutment/Spillway
HOLE ELEVATION 676.0
DATUM Geodetic (interpolated from contour plan)

LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
2" DIA. CONE
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
UNCONFINED COMPRESSION
VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX (LI)
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT
SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET
		676.0	0
	SAND - compact, brown, fine	667.5	Est.
	CLAYEY SILT grey, irregularly layered and seamed. Predominantly cohesive silt to very silty clay. Some non-cohesive areas. Trace fine gravel sizes	667.0	10
			20
		651.	
	SILT - v. dense, grey, non-cohesive, some areas with trace to some very fine sand. Wet and slight caving at approx. 35ft.		30
		640.0	
	SILT - finely layered, grey cohesive silt to very fine sandy silt. Some very plastic thin clay seams.		40
		631.0	
	SILT & V. FINE SAND - very dense, grey non-cohesive silt to v. silty v. fine sand.	624.0	50
	COHESIVE SILT - finely layered, grey cohesive to clayey, trace v. fine sand		60
		613.0	
	SILTY-SAND - v.fine, v.silty sand to v. sandy silt. Moist, v. dense, rare clay parting brownish Terminated	607.0	70
			80
			90
			100
			110

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT. (20, 40, 60, 80)
SHEAR STRENGTH P.S.F.

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT (10, 20, 30)

SAMPLE TYPE AND No.: 1, 2, 2A, 3, 4, 5, 5A, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21

NATURAL UNIT WEIGHT P.C.F.

Notes:
1. Hole advanced by continuous sampling using cable-tool drill drig. Some drillwater added. Cased to 16ft.8ins.
2. W.L. at 8.1/2ft. with hole open to 10ft.
With hole open to 34ft. and cased to 16ft.8ins., W.L. & wet slurry at 27ft.
Hole advanced to termination at 69ft, open and dry. W.L. rose to 35ft. overnight. Hole then grouted

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WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 7

PROJECT No. H296

BOREHOLE No. 6

PROJECT Deer Creek Dam

LOCATION nr. Langton, Ontario

HOLE LOCATION Proposed Main Borrow Area

HOLE ELEVATION 690.0 feet approx.

DATUM Geodetic (interpolated from contour plan)

LEGEND

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 2" DIA. CONE

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
- UNCONFINED COMPRESSION
- VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^LI

ATTERBERG LIMITS
- LIQUID LIMIT
- PLASTIC LIMIT

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
		690.0	0		
	SAND - compact to dense, light brown, moist, fine			Bulk Sample 1	
			10	Bulk Sample 2	
		677.0			
	SILT - layered clayey and sandy silt. Brown, some wet sand seams. Very wet and sandy from 19ft. to 21ft. ~672			Bulk Sample 3	
		669.0	20		
	SILTY CLAY - grey, moist very silty clay. V.stiff, some lamination with pure silt partings. Less clayey and frequently non-cohesive towards bottom of borehole.		30	Bulk Sample 4	
	Terminated	651.0	40		

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT. (20, 40, 60, 80)

SHEAR STRENGTH P.S.F

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT (10, 20, 30)

Notes:
1. Hole augered and uncased.
2. Open to 39.1/2ft. on completion. Caved & wet at ~18ft. overnight.

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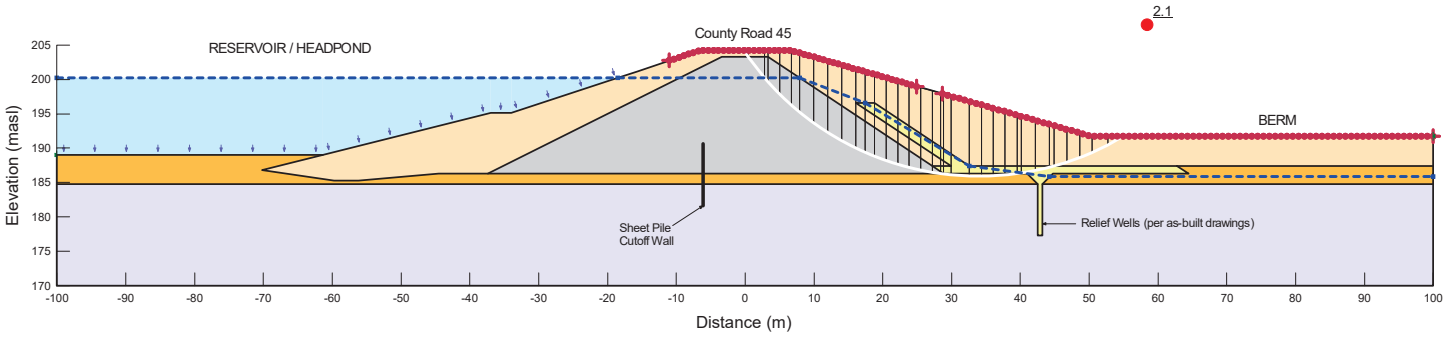
Appendix B

Slope Stability Analysis – Results

Docusign Envelope ID: F04AABA3-4B09-4639-999C-A3D2A3F55573

Figure B.1
Section 1-1
Loading Case LC1 - Normal Operating Conditions
Downstream

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	A: Impervious Core	Mohr-Coulomb	19	2	25
	B: Pervious Shell	Mohr-Coulomb	19.5	0	30
	C: Granular Filter	Mohr-Coulomb	20.5	0	32
	D: Sand and Silt (Alluvium); loose	Mohr-Coulomb	19	0	28
	E: Clayey Silt to Silty Clay; stiff to hard	Mohr-Coulomb	19	5	25

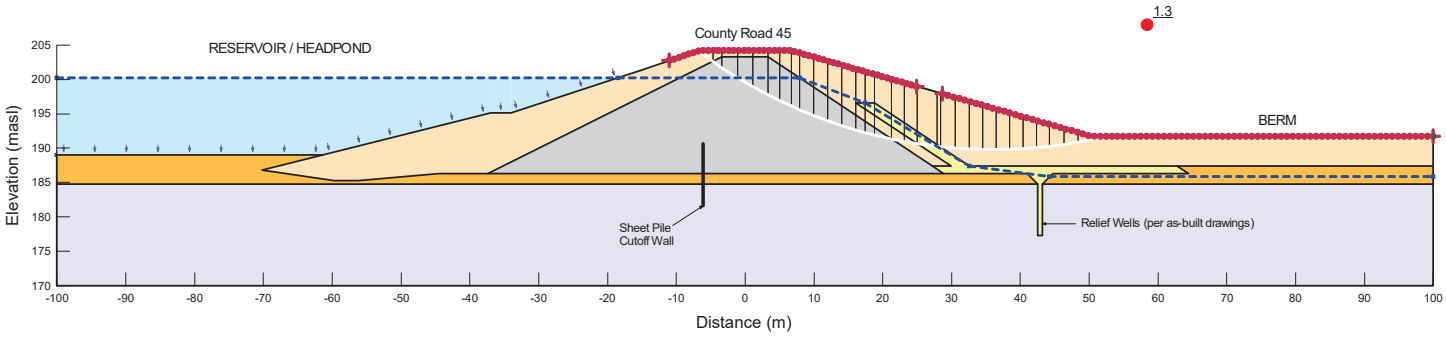


SLOPE/W Analysis	
B.1_S1_LC1_DS.gsz	
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Docusign Envelope ID: F04AABA3-4B09-4639-999C-A3D2A3F55573

Figure B.2
Section 1-1
Loading Case LC2 - Pseudostatic Analysis
Downstream

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	A: Impervious Core	Mohr-Coulomb	19	2	25
	B: Pervious Shell	Mohr-Coulomb	19.5	0	30
	C: Granular Filter	Mohr-Coulomb	20.5	0	32
	D: Sand and Silt (Alluvium); loose	Mohr-Coulomb	19	0	28
	E: Clayey Silt to Silty Clay; stiff to hard	Mohr-Coulomb	19	5	25

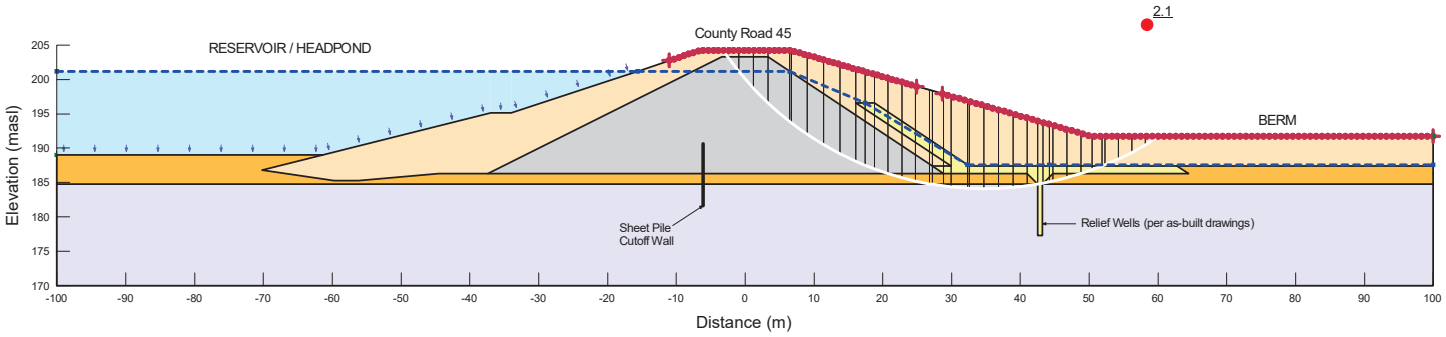


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Docusign Envelope ID: F04AABA3-4B09-4639-999C-A3D2A3F55573

Figure B.3
Section 1-1
Loading Case LC3 - Inflow Design Flood
Downstream

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	A: Impervious Core	Mohr-Coulomb	19	2	25
	B: Pervious Shell	Mohr-Coulomb	19.5	0	30
	C: Granular Filter	Mohr-Coulomb	20.5	0	32
	D: Sand and Silt (Alluvium); loose	Mohr-Coulomb	19	0	28
	E: Clayey Silt to Silty Clay; stiff to hard	Mohr-Coulomb	19	5	25

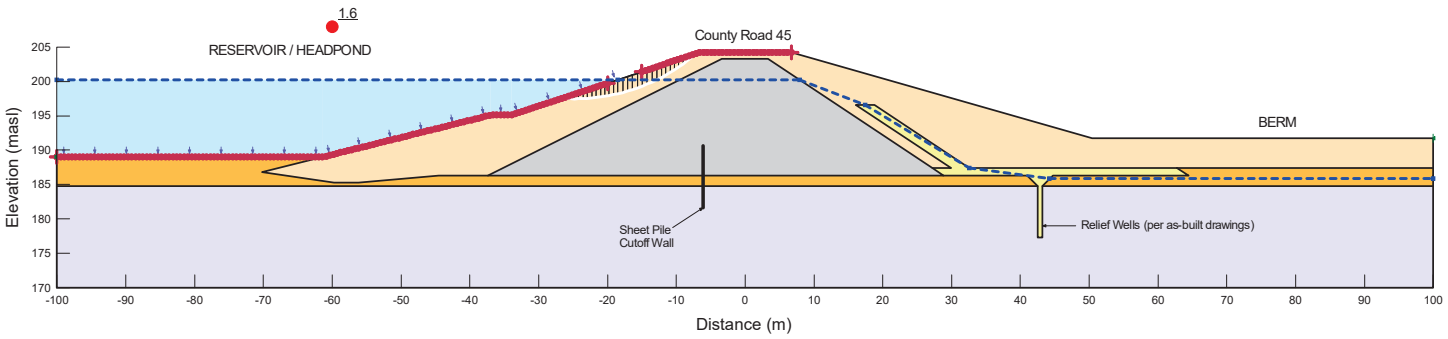


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Docusign Envelope ID: F04AABA3-4B09-4639-999C-A3D2A3F55573

Figure B.4
Section 1-1
Loading Case LC1 - Normal Operating Conditions
Upstream

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	A: Impervious Core	Mohr-Coulomb	19	2	25
	B: Pervious Shell	Mohr-Coulomb	19.5	0	30
	C: Granular Filter	Mohr-Coulomb	20.5	0	32
	D: Sand and Silt (Alluvium); loose	Mohr-Coulomb	19	0	28
	E: Clayey Silt to Silty Clay; stiff to hard	Mohr-Coulomb	19	5	25

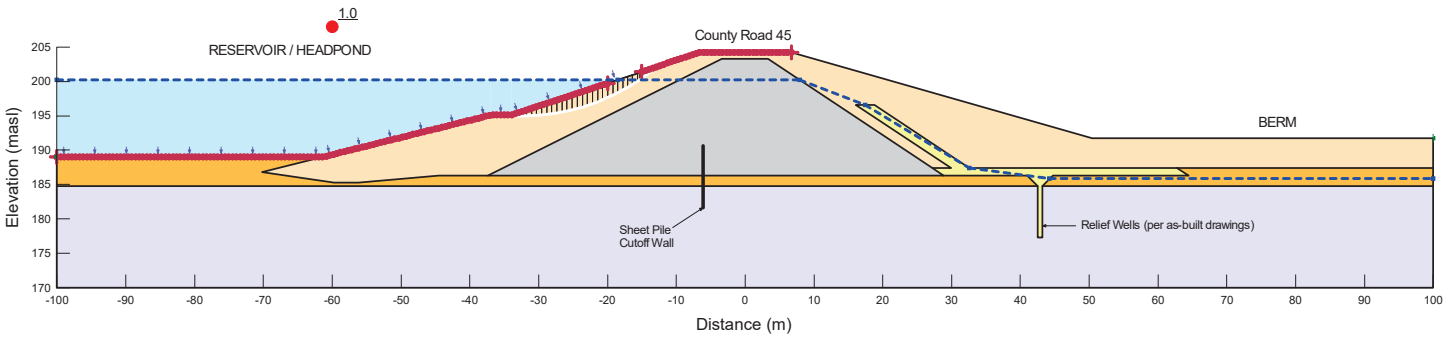


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Docusign Envelope ID: F04AABA3-4B09-4639-999C-A3D2A3F55573

Figure B.5
Section 1-1
Loading Case LC2 - Pseudostatic Analysis
Upstream

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	A: Impervious Core	Mohr-Coulomb	19	2	25
	B: Pervious Shell	Mohr-Coulomb	19.5	0	30
	C: Granular Filter	Mohr-Coulomb	20.5	0	32
	D: Sand and Silt (Alluvium); loose	Mohr-Coulomb	19	0	28
	E: Clayey Silt to Silty Clay; stiff to hard	Mohr-Coulomb	19	5	25

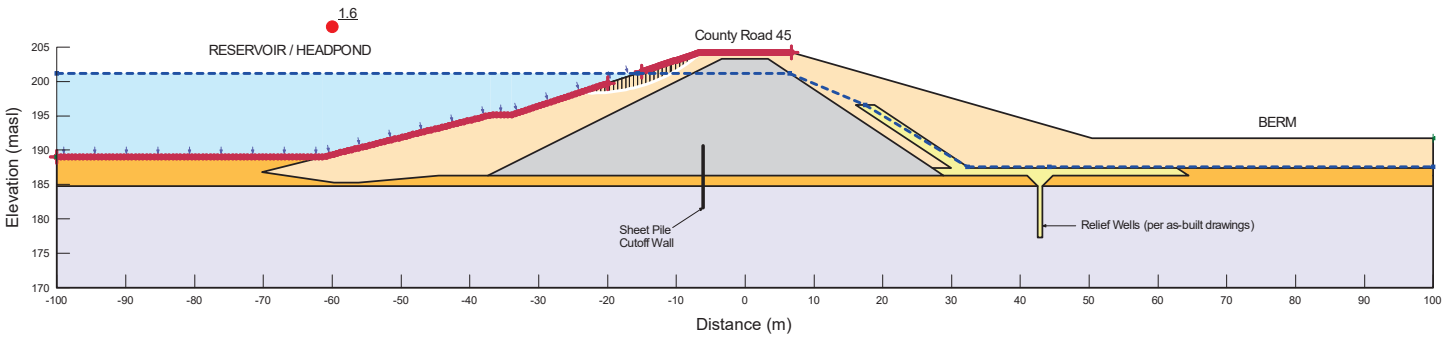


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B.5_S1_LC2_US.gsz	
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Docusign Envelope ID: F04AABA3-4B09-4639-999C-A3D2A3F55573

Figure B.6
Section 1-1
Loading Case LC3 - Inflow Design Flood
Upstream

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	A: Impervious Core	Mohr-Coulomb	19	2	25
	B: Pervious Shell	Mohr-Coulomb	19.5	0	30
	C: Granular Filter	Mohr-Coulomb	20.5	0	32
	D: Sand and Silt (Alluvium); loose	Mohr-Coulomb	19	0	28
	E: Clayey Silt to Silty Clay; stiff to hard	Mohr-Coulomb	19	5	25

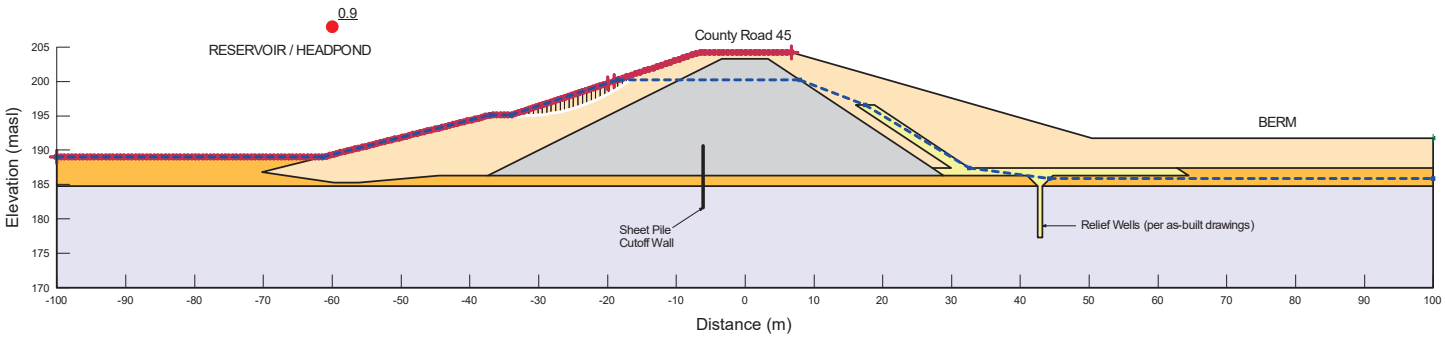


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B.6_S1_LC3_US.gsz	
2025-02-28	1:850

Docusign Envelope ID: F04AABA3-4B09-4639-999C-A3D2A3F55573

Figure B.7
Section 1-1
Loading Case LC4 - Rapid Drawdown
Upstream

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	A: Impervious Core	Mohr-Coulomb	19	2	25
	B: Pervious Shell	Mohr-Coulomb	19.5	0	30
	C: Granular Filter	Mohr-Coulomb	20.5	0	32
	D: Sand and Silt (Alluvium); loose	Mohr-Coulomb	19	0	28
	E: Clayey Silt to Silty Clay; stiff to hard	Mohr-Coulomb	19	5	25

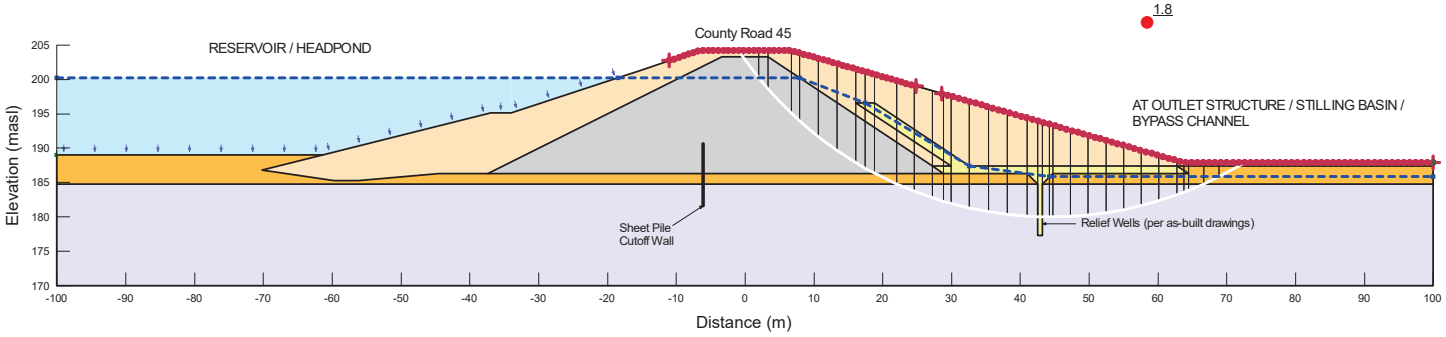


SLOPE/W Analysis	
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Figure B.8
Section 1A-1A
Loading Case LC1 - Normal Operating Conditions
Downstream

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	A: Impervious Core	Mohr-Coulomb	19	2	25
	B: Pervious Shell	Mohr-Coulomb	19.5	0	30
	C: Granular Filter	Mohr-Coulomb	20.5	0	32
	D: Sand and Silt (Alluvium); loose	Mohr-Coulomb	19	0	28
	E: Clayey Silt to Silty Clay; stiff to hard	Mohr-Coulomb	19	5	25

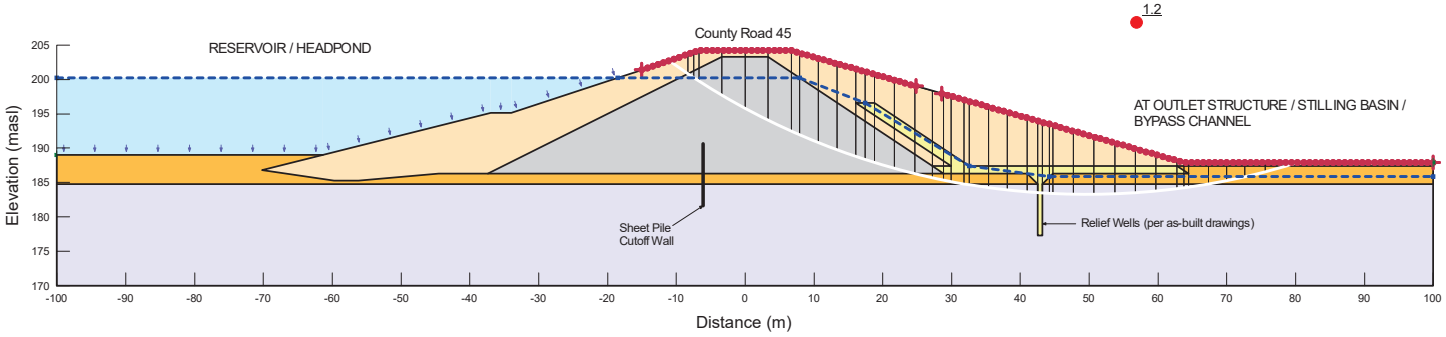


SLOPE/W Analysis	
B.8_S1A_LC1_DS.gsz	
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Docusign Envelope ID: F04AABA3-4B09-4639-999C-A3D2A3F55573

Figure B.9
Section 1A-1A
Loading Case LC2 - Pseudostatic Analysis
Downstream

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	A: Impervious Core	Mohr-Coulomb	19	2	25
	B: Pervious Shell	Mohr-Coulomb	19.5	0	30
	C: Granular Filter	Mohr-Coulomb	20.5	0	32
	D: Sand and Silt (Alluvium); loose	Mohr-Coulomb	19	0	28
	E: Clayey Silt to Silty Clay; stiff to hard	Mohr-Coulomb	19	5	25

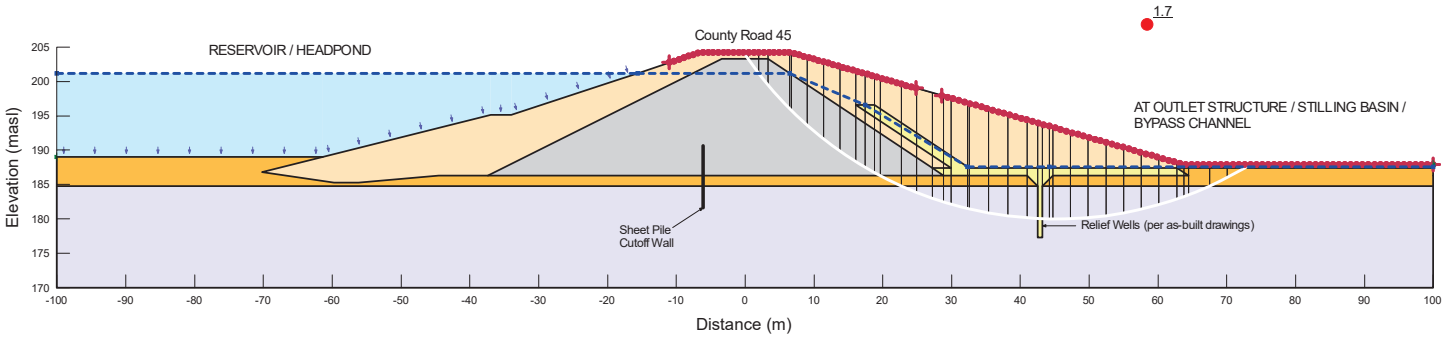


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B.9_S1A_LC2_DS.gsz	
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Figure B.10
Section 1A-1A
Loading Case LC3 - Inflow Design Flood
Downstream

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	A: Impervious Core	Mohr-Coulomb	19	2	25
	B: Pervious Shell	Mohr-Coulomb	19.5	0	30
	C: Granular Filter	Mohr-Coulomb	20.5	0	32
	D: Sand and Silt (Alluvium); loose	Mohr-Coulomb	19	0	28
	E: Clayey Silt to Silty Clay; stiff to hard	Mohr-Coulomb	19	5	25



SLOPE/W Analysis	
B.10_S1A_LC3_DS.gsz	
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Appendix C

Site Photographs

Docusign Envelope ID: F04AABA3-4B09-4639-999C-A3D2A3F55573



Photograph 1 View of County Road 45 extending along crest of embankment dam, looking east



Photograph 2 View of County Road 45 and top of downstream slope, looking east

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Photograph 3 View of downstream embankment slope, looking south towards stabilizing berm at slope toe, with spillway/chute structure visible on right



Photograph 4 Additional view of downstream slope with berm, and higher slope section at outlet structure location visible

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Photograph 5 View of concrete spillway/chute built into west abutment area of downstream slope, looking south towards continuation of Deer Creek



Photograph 6 Additional view of concrete spillway and downstream embankment slope, looking north

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Photograph 7 Additional view of downstream embankment slope, looking north



Photograph 8 Views of higher section of downstream slope at concrete outlet structure location

Docusign Envelope ID: F04AABA3-4B09-4639-999C-A3D2A3F55573



Photograph 9 View of height difference between top of stabilizing berm at toe of downstream slope and concrete outlet structure, with bypass channel to Deer Creek visible in background



Photograph 10 View of bypass channel with erosion of bank slopes visible

Docusign Envelope ID: F04AABA3-4B09-4639-999C-A3D2A3F55573



Photograph 11 Additional views of toe erosion along bank slopes of bypass channel



Photograph 12 Additional view of toe erosion along bank slopes of bypass channel, at a meander apex of the channel

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Photograph 13 View of east abutment area, transition to natural slope on downstream side



Photograph 14 View of west abutment area on downstream side, with road embankment slope and ditch area visible, and spillway structure visible in background

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Photograph 15 Additional view of west abutment area on downstream side, with ditch outlet into spillway visible, and transition to natural slope area on right



Photograph 16 Views of downstream slope on west side of spillway/chute structure, and adjacent natural slope area looking south (L) and looking north (R)

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Photograph 17 View of east abutment area on upstream side



Photograph 18 View of upstream slope and reservoir/headpond, with rip rap protection visible

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Photograph 19 View of upstream slope with rip rap protection and vegetation visible



Photograph 20 Additional view of upstream slope surface with condition of rip rap protection, and vegetation visible

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Photograph 21 View of upstream slope with vegetation visible



Photograph 22 View of lift control chamber in upstream slope

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Photograph 23 Additional view of rip rap conditions on upstream slope surface



Photograph 24 View of upstream slope between lift control chamber and bridge over spillway, looking west with rip rap visible (L), looking east with vegetation visible (R)

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Photograph 25 View of upstream slope surface, looking east



Photograph 26 View of upstream slope with some mature vegetation (small trees, larger brush) growing within rip rap protection

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Photograph 27 View of larger boulder within rip rap protection



Photograph 28 Additional views of rip rap protection on upstream slope surface

Appendix F

Public Safety Assessment

Public Safety Risk Assessment Input: Deer Creek Dam

Project No: 5684
Project Name: Dam Safety Review Report
Designed/Checked By: SO / DG
Date: January 0, 1900

Risk Tolerance

Risk Level	Lower	Upper
Low	1	4
Medium	5	9
High	10	25

Risk Rating and Risk Level			Incident Consequences				
			Insignificant	Minor	Moderate	Major	Catastrophic
			1	2	3	4	5
Incident Likelihood	Almost Certain	5	Medium (5)	High (10)	High (15)	High (20)	High (25)
	Likely	4	Low (4)	Medium (8)	High (12)	High (16)	High (20)
	Possible	3	Low (3)	Medium (6)	Medium (9)	High (12)	High (15)
	Unlikely	2	Low (2)	Low (4)	Medium (6)	Medium (8)	High (10)
	Rare	1	Low (1)	Low (2)	Low (3)	Low (4)	Medium (5)

Component Areas

Component Area Name	Component Area Description
Headpond	The Headpond component area boundary extends from the upstream side of the dam to a distance of approximately 25 m upstream of the dam and encompasses the shoreline to a distance of 2 m back from the normal (summer) water's edge, and an area approximately 10m in diameter around the intake of the low flow valve.
Structure	The Structure component area boundary includes the concrete control structure, the two earth embankment sections, the concrete spillway and the low flow spillway.
Tailrace	The Downstream Area component area extends from the downstream side of the dam to a distance of approximately 70 m downstream.

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Date	Name	Signature

Headpond

Dam Name	Deer Creek Dam Long Point Region Conservation Authority

Describe the boundary of the Component:

The Headpond component area boundary extends from the upstream side of the dam to a distance of approximately 25 m upstream of the dam and encompasses the shoreline to a distance of 2 m back from the normal (summer) water's edge, and an area approximately 10m in diameter around the intake of the low flow valve.

Activity Location within the Hazardous Area	Activity Description	Potential Hazard																									Risk Reduction Measures Present at the Time of Assessment														Risk Level at the Time of Assessment: Risk Assessment				Comments
		Rapidly increasing in water levels	Rapidly increasing water flows	Strong currents or undertows	Frequently dry riverbed	Presence of spillway with sluicegate	Automatic Emergency Operation of Spillway Ga	Presence of spillway with stop-logs	Presence of overflow spillway or dam	Presence of discharge valve/pipe	Submerged hydraulic jump	Submerged underwater structures	Remote control flow equipment	Automatic control flow equipment	Steep or slippery banks	Falling from height >3 metres	Pinching or crushing	Thin ice	Changing flow/depth may result in Stranding	Floating debris	Flow or level changes as a result of maintenanc	Unsecured mechanical/electrical equipment	Unsecured or exposed live electrical conductor	Inadequate guardrails/handrails for public	Open holes or tripping	Wind / Rough Water	Other (define)	Signage	Public Education (Local Initiatives)	Safety Buoys	Safety Booms	Audible Danger Signalling Devices	Visual Danger Signalling Devices	Fencing/Railings	Barricades (Vehicle or People)	Security Patrols	24/7 Video Surveillance	Operational Controls (Procedures)	Write a letter to the Adjacent Property Owner	Other (define)	Other (define)	Incident Likelihood Rating (ILR)	Incident Consequences Rating (ICR)	Risk Rating	Risk Level	
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	1	2	3	4	5	6	7	8	9	10	11	12	13	14	ILR	ICR	RR	RL	
From Water/Ice	Fishing from Boat			X				X		X										X								X			X							X				3	2	6	Medium	
	Boating (under power)			X				X		X										X								X			X							X				3	2	6	Medium	
	Sailing																																													
	Windsurfing																																													
	Canoeing/Kayaking/Rowing			X				X		X										X								X			X							X				3	3	9	Medium	
	Waterskiing																																													
	Swimming			X				X		X										X								X			X			X				X				3	3	9	Medium	
	Jet Ski																																													
	Scuba Diving																																													
	Swimming/Diving																																													
	Skating																																													
	Ice fishing																																													
	Snowmobiling																																													
	Paddle Boarding																																													
From Shore	Fishing from Shore													X											X			X						X								1	2	2	Low	
	Walking													X											X			X						X								1	2	2	Low	
	Climbing																																													
	Camping																																													
	Picnicking																																													
	ATV / Dirt Biking																																													
	Hiking													X											X			X						X								1	2	2	Low	
	Skiing																																													
	Snowshoeing																																													
	Driving																																													
	Biking																																													
	Scuba Diving																																													
	Swimming / Diving																																													
	Accessing electrical equipment																																													
	Accessing mechanical equipment																																													

CDA Public Safety Around Dams Risk Assessment Tool

Headpond

Anticipated (Current or Expected) Risk Level

As a result of recent risk reduction measures implemented or additional risk reduction measures to be implemented in the immediate future

Additional Risk Reduction
Measures have recently been implemented or will be implemented in the immediate future

Risk Characterization

Signage	Public Education (Local Initiatives)	Safety Buoys	Safety Booms	Audible Danger Signalling Devices	Visual Danger Signalling Devices	Fencing/Railings	Barricades (Vehicle or People)	Security Patrols	24/7 Video Surveillance	Operational Controls (Procedures)	Write a letter to the Adjacent Property Owner	Other (define)	Other (define)	Incident Likelihood Rating (ILR)	Incident Consequences Rating (ICR)	Risk Rating	Current or Expected Risk Level	Comments (Includes assumptions, conclusions or observations)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	ILR	ICR	RR	RL	
X			X											1	2	2	Low	Install new year-round public safety boom with yellow floats, a hazard buoy at the intake of the valve system, install large upstream facing public safety sign on the upstream side of the bridge and develop operational controls.
X			X											1	2	2	Low	Install new year-round public safety boom with yellow floats, a hazard buoy at the intake of the valve system, install large upstream facing public safety sign on the upstream side of the bridge and develop operational controls.
X			X											1	3	3	Low	Install new year-round public safety boom with yellow floats, a hazard buoy at the intake of the valve system, install large upstream facing public safety sign on the upstream side of the bridge and develop operational controls.
X			X											1	3	3	Low	Install new year-round public safety boom with yellow floats, a hazard buoy at the intake of the valve system, install large upstream facing public safety sign on the upstream side of the bridge and develop operational controls.
X			X											1	2	2	Low	Install new year-round public safety boom with yellow floats and install large upstream facing public safety sign on the upstream side of the bridge.
X			X											1	2	2	Low	Install new year-round public safety boom with yellow floats and install large upstream facing public safety sign on the upstream side of the bridge.
X			X											1	2	2	Low	Install new year-round public safety boom with yellow floats and install large upstream facing public safety sign on the upstream side of the bridge.

CDA Public Safety Around Dams Risk Assessment Tool

Date	Name	Signature

Structure

Describe the boundary of the Component:

The Structure component area boundary includes the concrete control structure, the two earth embankment sections, the concrete spillway and the low flow spillway.

Dam Name	Deer Creek Dam Long Point Region Conservation Authority

Activity Location within the Hazardous Area	Activity Description	Potential Hazard																								Risk Reduction Measures Present at the Time of Assessment														Risk Assessment				Comments
		Rapidly increasing in water levels	Rapidly increasing water flows	Strong currents or undertows	Frequently dry riverbed	Presence of spillway with sluicegate	Automatic Emergency Operation of Spillway Ga	Presence of spillway with stop-logs	Presence of overflow spillway or dam	Presence of discharge valve/pipe	Submerged hydraulic jump	Submerged underwater structures	Remote control flow equipment	Automatic control flow equipment	Steep or slippery banks	Falling from height >3 metres	Pinching or crushing	Thin ice	Changing flow/depth may result in Stranding	Floating debris	Flow or level changes as a result of maintenanc	Unsecured mechanical/electrical equipment	Unsecured or exposed live electrical conductor	Inadequate guardrails/handrails for public	Open holes or tripping	Slippery surfaces	Other (define)	Signage	Public Education (Local Initiatives)	Safety Buoys	Safety Booms	Audible Danger Signalling Devices	Visual Danger Signalling Devices	Fencing/Railings	Barricades (Vehicle or People)	Security Patrols	24/7 Video Surveillance	Operational Controls (Procedures)	Write a letter to the Adjacent Property Owner	Other (define)	Other (define)	Incident Likelihood Rating (ILR)	Incident Consequences Rating (ICR)	Risk Rating	Risk Level	
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	1	2	3	4	5	6	7	8	9	10	11	12	13	14	ILR	ICR	RR	RL	
From Water/Ice	Fishing from Boat																																											
	Boating (under power)																																												
	Sailing																																												
	Windsurfing																																												
	Canoeing/Kayaking/Rowing																																												
	Waterskiing																																												
	Swimming																																												
	Jet Ski																																												
	Scuba Diving																																												
	Swimming/Diving																																												
	Skating																																												
	Ice fishing																																												
	Snowmobiling																																												
	Paddle Boarding																																												
From Structure	Fishing from Shore/Structure															X										X		X						X							2	2	4	Low	
	Walking																									X		X						X	X						4	2	8	Medium	
	Climbing			X				X	X							X										X		X						X							3	4	12	High	
	Camping																																												
	Picnicking																																												
	ATV / Dirt Biking																																												
	Hiking																																												
	Skiing																																												
	Snowshoeing																																												
	Driving																									X								X	X						5	1	5	Medium	
	Biking																									X								X	X						4	1	4	Low	
	Scuba Diving																																												
	Swimming / Diving																																												
	Accessing electrical equipment																																												
	Accessing mechanical equipment																																												

Revised: October 15, 2011

CDA Public Safety Around Dams Risk Assessment Tool

Structure

Anticipated (Current or Expected) Risk Level

As a result of recent risk reduction measures implemented or additional risk reduction measures to be implemented in the immediate future

Additional Risk Reduction
Measures have recently been implemented or will be implemented in the immediate future

Signage	Public Education (Local Initiatives)	Safety Buoys	Safety Booms	Audible Danger Signalling Devices	Visual Danger Signalling Devices	Fencing/Railings	Barricades (Vehicle or People)	Security Patrols	24/7 Video Surveillance	Operational Controls (Procedures)	Write a letter to the Adjacent Property Owner	Other (define)	Other (define)	Incident Likelihood Rating (ILR)	Incident Consequences Rating (ICR)	Risk Rating	Current or Expected Risk Level	Comments (Includes assumptions, conclusions or observations)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	ILR	ICR	RR	RL	
														2	2	4	Low	Install railings around the low flow outlet.
X														4	2	8	Medium	
X						X								1	4	4	Low	Install railings around the low flow outlet , install downstream facing public safety sign, install No Trespasing signs at the ends of the concrete spillway.
														5	1	5	Medium	
														4	1	4	Low	

CDA Public Safety Around Dams Risk Assessment Tool

Date	Name	Signature

Tailrace

Describe the boundary of the Component:

The Downstream Area component area extends from the downstream side of the dam to a distance of approximately 70 m downstream.

Dam Name	Deer Creek Dam Long Point Region Conservation Authority

Activity Location within the Hazardous Area	Activity Description	Potential Hazard																									Risk Reduction Measures Present at the Time of Assessment													Risk Assessment				Comments
		Rapidly increasing in water levels	Rapidly increasing water flows	Strong currents or undertows	Frequently dry riverbed	Presence of spillway with sluicegate	Automatic Emergency Operation of Spillway Ga	Presence of spillway with stop-logs	Presence of overflow spillway or dam	Presence of discharge valve/pipe	Submerged hydraulic jump	Submerged underwater structures	Remote control flow equipment	Automatic control flow equipment	Steep or slippery banks	Falling from height >3 metres	Pinching or crushing	Thin ice	Changing flow/depth may result in Stranding	Floating debris	Flow or level changes as a result of maintenanc	Unsecured mechanical/electrical equipment	Unsecured or exposed live electrical conductor	Inadequate guardrails/handrails for public	Open holes or tripping	Slippery Surfaces	Other (define)	Signage	Public Education (Local Initiatives)	Safety Buoys	Safety Booms	Audible Danger Signalling Devices	Visual Danger Signalling Devices	Fencing/Railings	Barricades (Vehicle or People)	Security Patrols	24/7 Video Surveillance	Operational Controls (Procedures)	Write a letter to the Adjacent Property Owner	Other (define)	Other (define)	Incident Likelihood Rating (ILR)	Incident Consequences Rating (ICR)	Risk Rating	Risk Level	
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	1	2	3	4	5	6	7	8	9	10	11	12	13	14	ILR	ICR	RR	RL	
From Water/Ice	Fishing from Boat																																											
	Boating (under power)																																											
	Sailing																																											
	Windsurfing																																											
	Canoeing/Kayaking/Rowing																																											
	Waterskiing																																											
	Swimming																																											
	Jet Ski																																											
	Scuba Diving																																											
	Swimming/Diving																																											
	Skating																																											
	Ice fishing																																											
	Snowmobiling																																											
	Paddle Boarding																																											
	Fly Fishing																																											
	Wading			X				X	X	X										X						X															3	2	6	Medium	
From Shore/Structure	Fishing from Shore													X											X			X													3	2	6	Medium	
	Walking													X											X			X													2	3	6	Medium	
	Climbing																																												
	Camping																																												
	Picnicking																																											
	ATV / Dirt Biking																																											
	Hiking													X											X			X													2	2	4	Low	
	Skiing																																												
	Snowshoeing																																											
	Driving																																											
	Biking																																											
	Scuba Diving																																											
	Swimming / Diving																																											
	Accessing electrical equipment																																											
	Accessing mechanical equipment																																											

CDA Public Safety Around Dams Risk Assessment Tool

Tailrace

Anticipated (Current or Expected) Risk Level

As a result of recent risk reduction measures implemented or additional risk reduction measures to be implemented in the immediate future

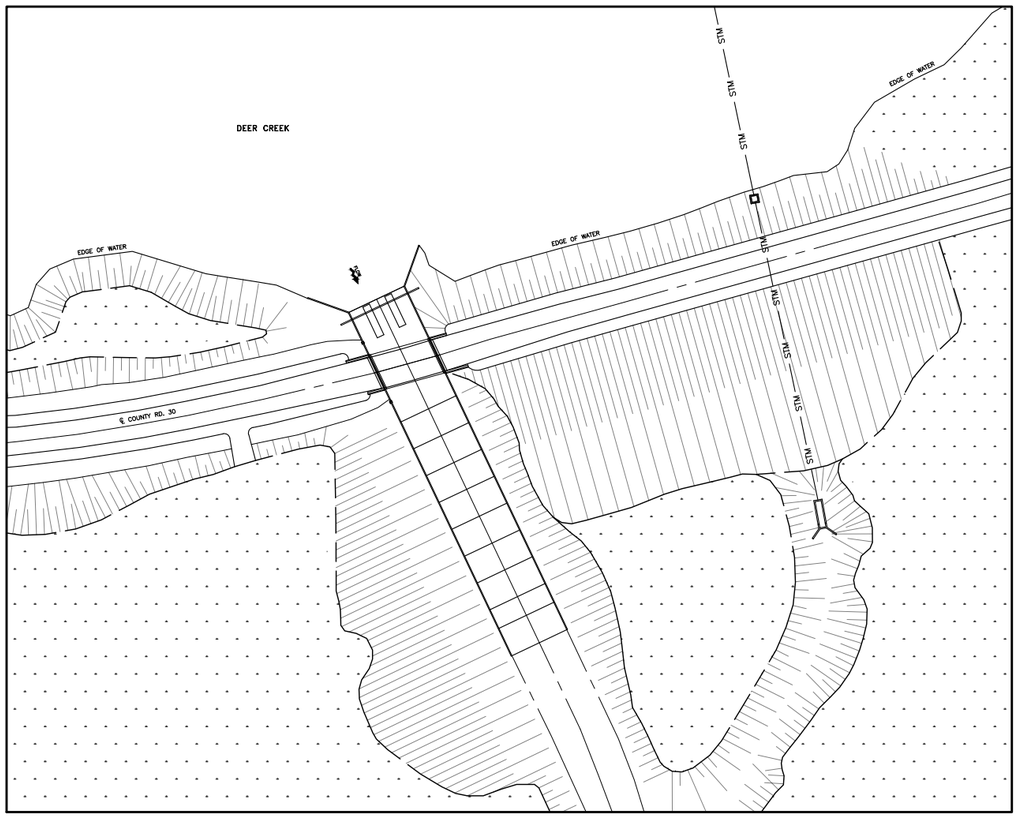
Additional Risk Reduction Measures have recently been implemented or will be implemented in the immediate future														Risk Characterization				Comments (Includes assumptions, conclusions or observations)
Signage	Public Education (Local Initiatives)	Safety Buoys	Safety Booms	Audible Danger Signalling Devices	Visual Danger Signalling Devices	Fencing/Railings	Barricades (Vehicle or People)	Security Patrols	24/7 Video Surveillance	Operational Controls (Procedures)	Write a letter to the Adjacent Property Owner	Remove Downstream ATV Trail	Other (define)	Incident Likelihood Rating (ILR)	Incident Consequences Rating (ICR)	Risk Rating	Current or Expected Risk Level	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	ILR	ICR	RR	RL	
X														1	2	2	Low	Install downstream facing public safety sign, install No Trespasing signs at the ends of the concrete spillway.
X														1	2	2	Low	Install downstream facing public safety sign, install No Trespasing signs at the ends of the concrete spillway.
X														1	3	3	Low	
																		Install downstream facing public safety sign, install No Trespasing signs at the ends of the concrete spillway.
X														1	2	2	Low	Install downstream facing public safety sign, install No Trespasing signs at the ends of the concrete spillway.

Appendix G

Deterioration and Rehabilitation Drawings

LPRCA DEER CREEK DAM

LONG POINT REGION CONSERVATION AUTHORITY
DETERIORATION SURVEY PACKAGE



SHEET LIST TABLE	
SHEET TITLE	SHEET NUMBER
SITE PLAN	01
PRIMARY STRUCTURE DETERIORATION	02
BYPASS OUTLET DETERIORATION	03

D.M. WILLS ASSOCIATES LIMITED ANSI-B 2019-11

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CREATED: 2025-02-27
MODIFIED: 2025-02-28 16:45



SITE PLAN
SCALE 1:300

LONG POINT REGION CONSERVATION AUTHORITY

DEER CREEK DAM

SITE PLAN

SHEET 01

D.M. Wills Associates Limited
150 Jameson Drive
Peterborough, Ontario
Canada, K9J 0B9

P. 705.742.2297
F. 705.748.9944
E. wills@dmwills.com

METRIC
MILLIMETRES OR METRES (UNLESS NOTED)

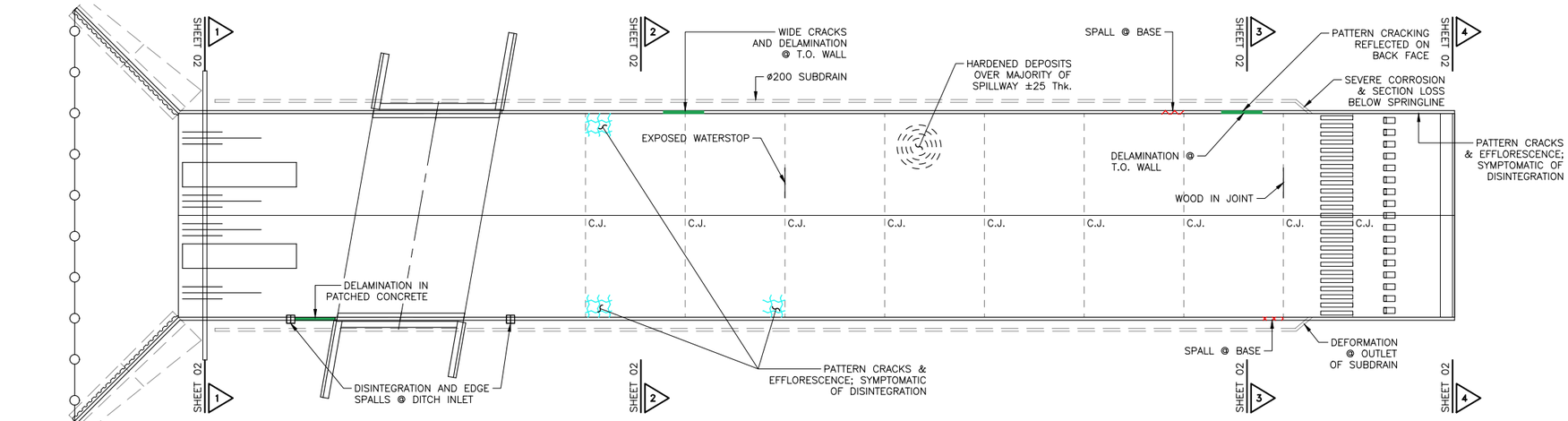
LEGEND:

- CB — CATCH BASIN
- MH — MANHOLE
- LS — LIGHT STANDARD
- O/H — OVERHEAD WIRE
- MW — MONITORING WELL
- GAS — GAS LINE
- STM — STORM SEWER
- FO — BURIED FIBRE OPTIC LINE
- FOUND MONUMENT

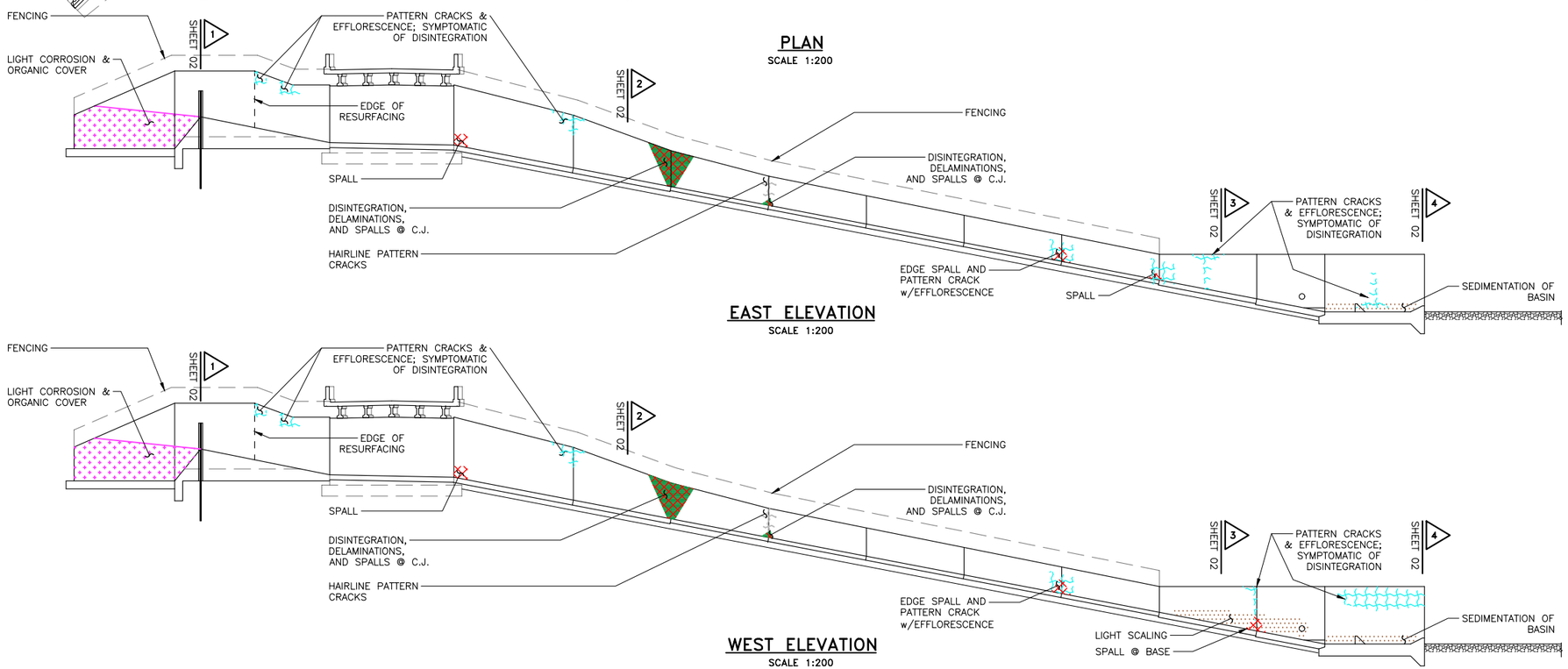
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D.M. WILLS ASSOCIATES LIMITED ANSI–B 2019–11

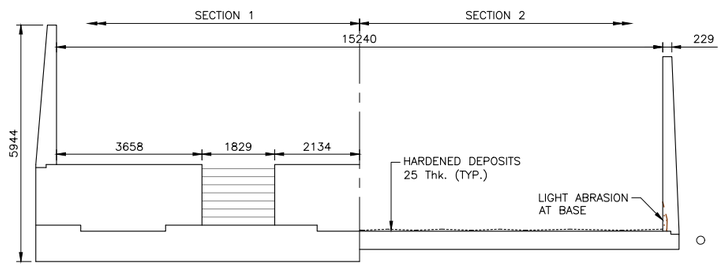


PLAN
SCALE 1:200

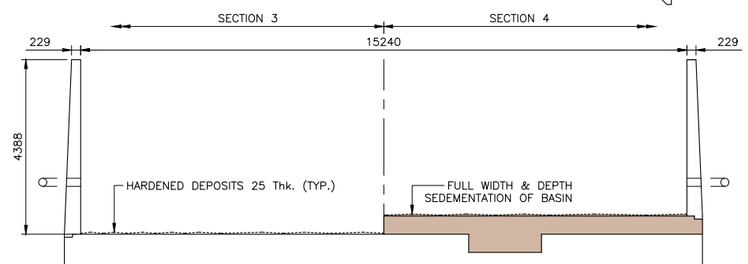


EAST ELEVATION
SCALE 1:200

WEST ELEVATION
SCALE 1:200



1 2 SECTION 1&2
SCALE 1:75



3 4 SECTION 3&4
SCALE 1:75

LONG POINT REGION CONSERVATION AUTHORITY

DEER CREEK DAM

PRIMARY STRUCTURE DETERIORATION

SHEET 02

D.M. Wills Associates Limited
150 Jameson Drive
Peterborough, Ontario
Canada, K9J 0B9
P. 705.742.2297
F. 705.748.9944
E. wills@dmwills.com

METRIC
MILLIMETRES OR METRES (UNLESS NOTED)

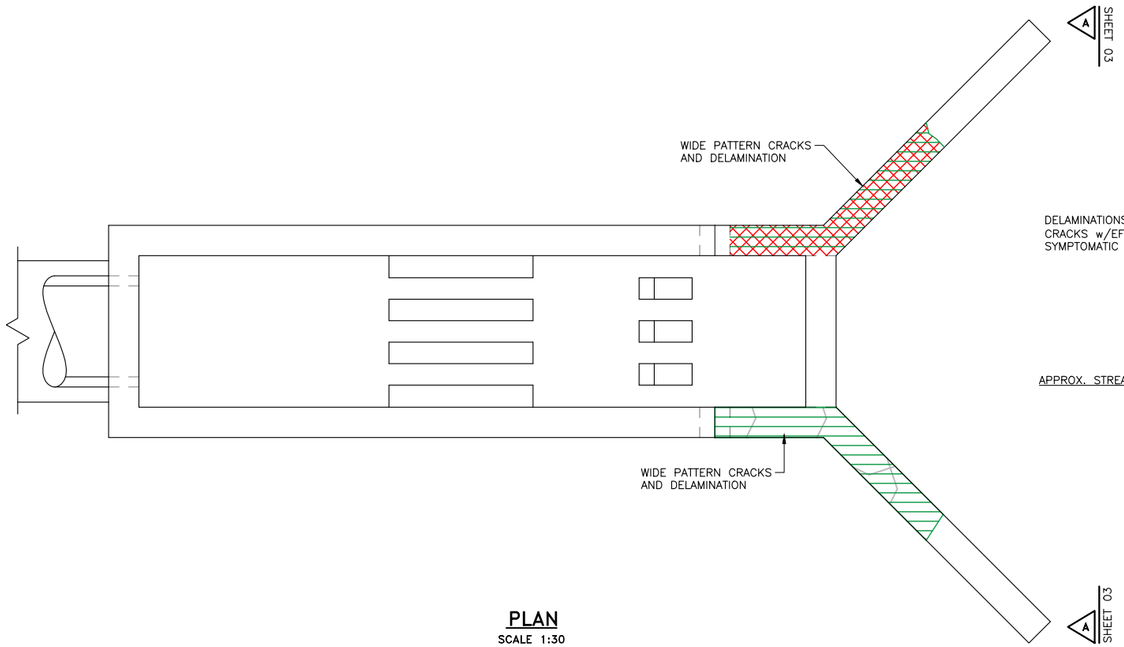
CONCRETE SURFACE DETERIORATION

- MEDIUM CONCRETE CRACKS
- WIDE CONCRETE CRACKS
- MEDIUM EFFLORESCENCE
- WIDE EFFLORESCENCE
- WET EFFLORESCENCE
- LIGHT SCALING
- MEDIUM SCALING
- SEVERE SCALING
- SURFACE RUST STAINS
- SURFACE RUST STAINS w/MINOR SCALING
- SPALLS
- DELAMINATIONS
- PATCHES
- MAP/PATTERN CRACKS
- BEARING IN FAIR CONDITION
- BEARING IN POOR CONDITION
- MEDIUM STAINED CONCRETE CRACKS
- WIDE STAINED CONCRETE CRACKS

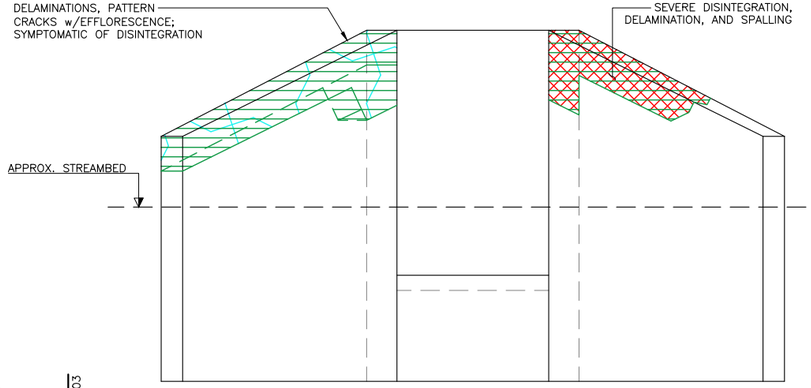
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DATE	BY	DESCRIPTION

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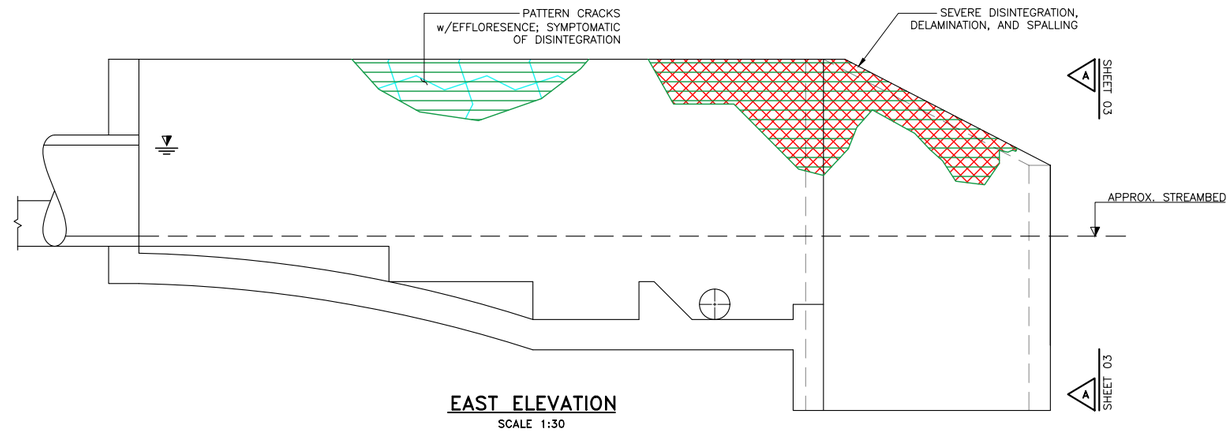
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D.M. WILLS ASSOCIATES LIMITED
ANSI–B
2019–11



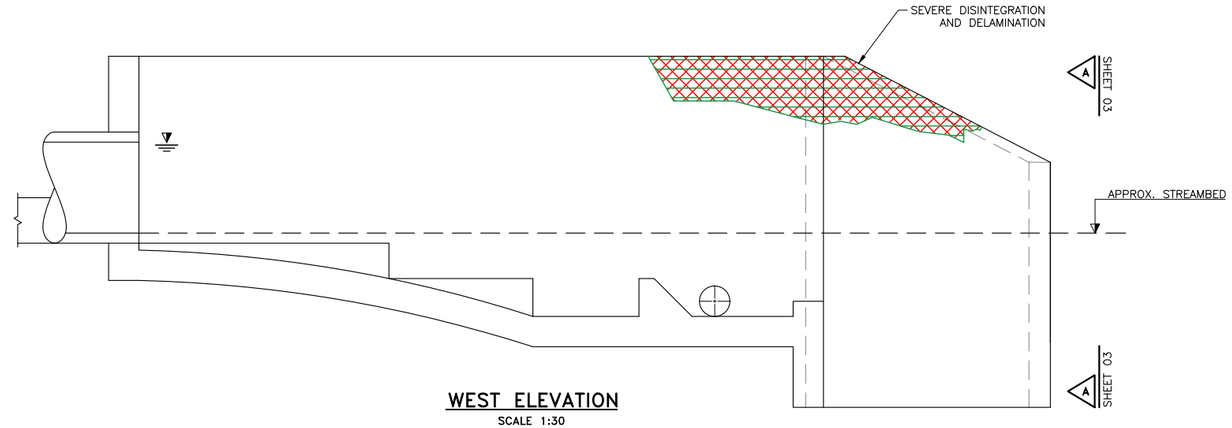
PLAN
SCALE 1:30



A SECTION A–A
SCALE 1:30



EAST ELEVATION
SCALE 1:30



WEST ELEVATION
SCALE 1:30

LONG POINT REGION CONSERVATION AUTHORITY

DEER CREEK DAM

BYPASS OUTLET DETERIORATION

SHEET 03

D.M. Wills Associates Limited
150 Jameson Drive
Peterborough, Ontario
Canada, K9J 0B9

P. 705.742.2297
F. 705.748.9944
E. wills@dmwills.com

METRIC
MILLIMETRES OR METRES (UNLESS NOTED)

REVISIONS		
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–	–	–
DATE	BY	DESCRIPTION

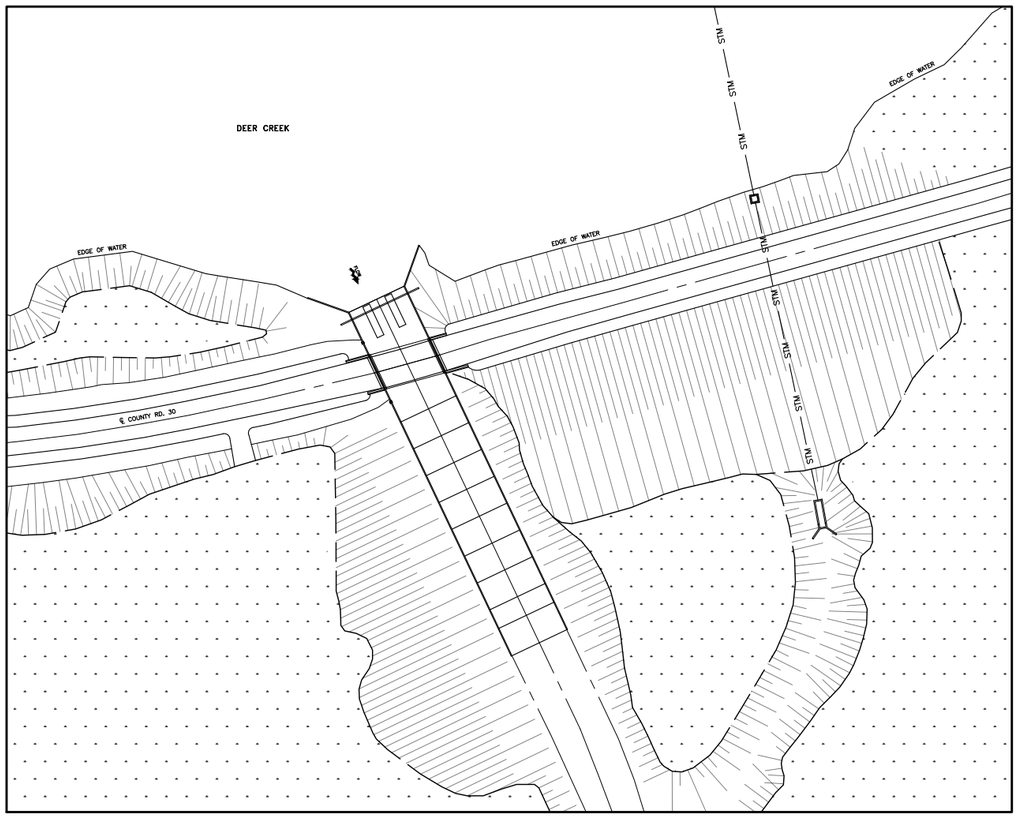
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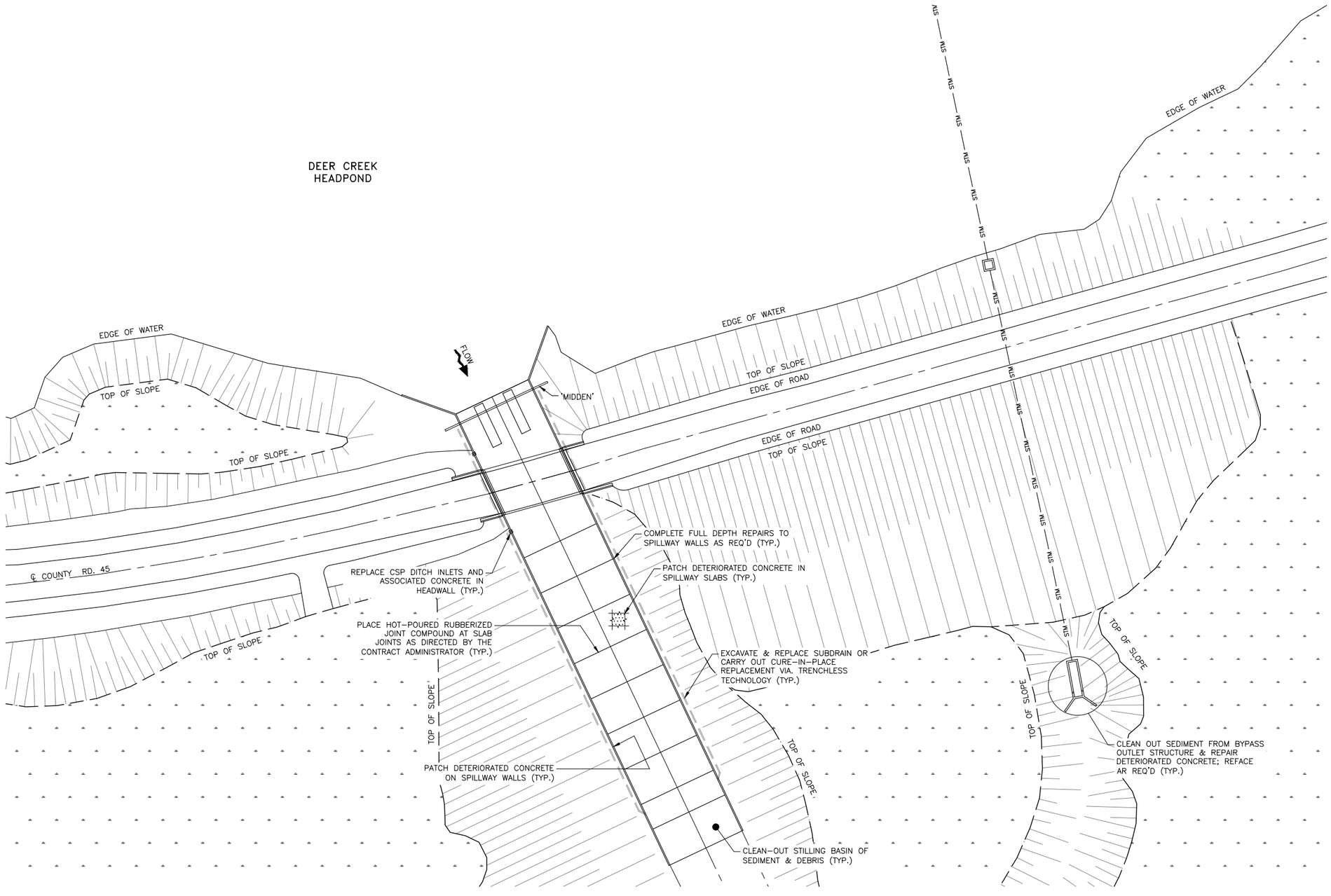
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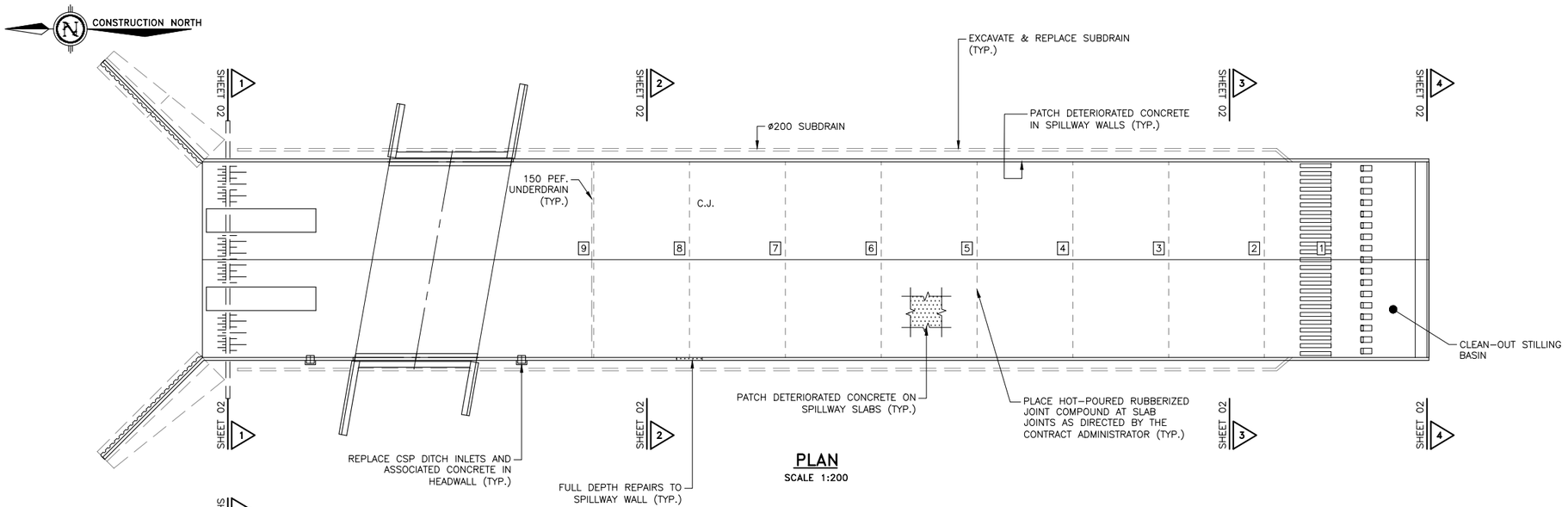
D.M. WILLS ASSOCIATES LIMITED ANSI–B 2019–11

LPRCA DEER CREEK DAM

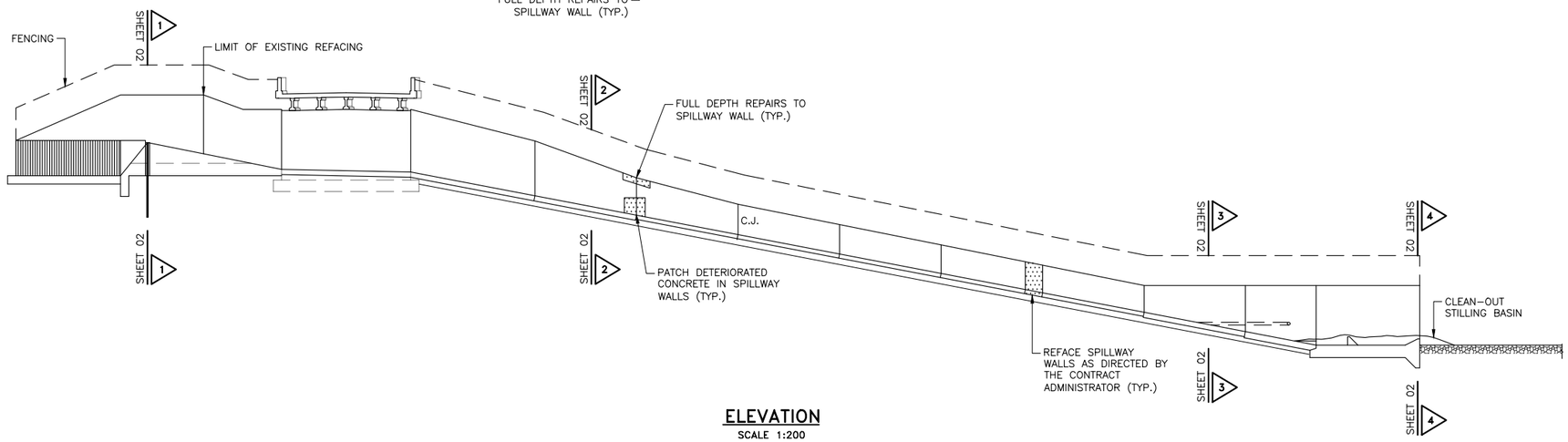
LONG POINT REGION CONSERVATION AUTHORITY
PRELIMINARY REHABILITATION PACKAGE


DEER CREEK
EDGE OF WATER
EDGE OF WATER
STN
STN
STN

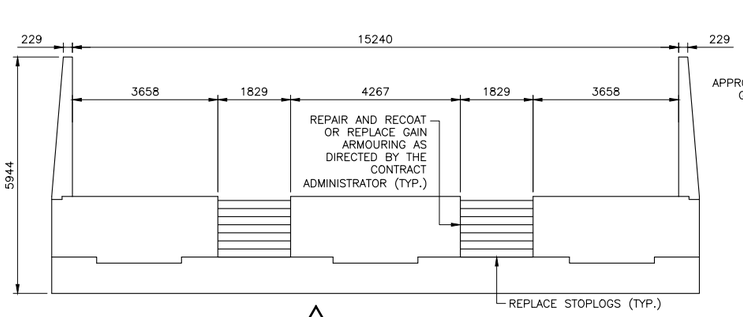

DEER CREEK
HEADPOND
EDGE OF WATER
TOP OF SLOPE
TOP OF SLOPE
FLOW
'MIDDEN'
EDGE OF WATER
EDGE OF ROAD
TOP OF SLOPE
EDGE OF ROAD
TOP OF SLOPE
STM
STM
STM



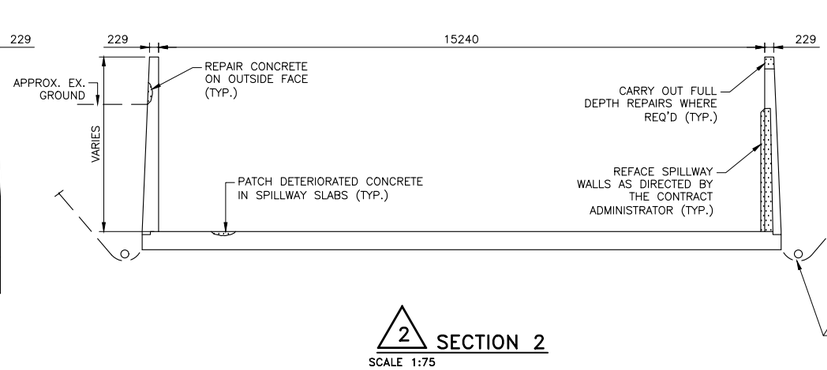
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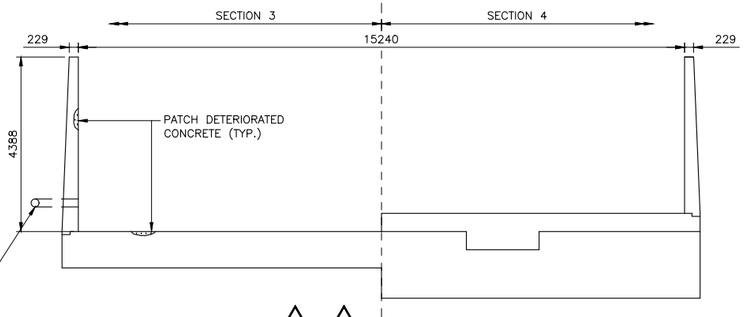
ELEVATION
SCALE 1:200



1 SECTION 1-1
SCALE 1:75



2 SECTION 2
SCALE 1:75



3 4 SECTION 3&4
SCALE 1:75

LONG POINT REGION CONSERVATION AUTHORITY

DEER CREEK DAM

PRIMARY STRUCTURE GENERAL ARRANGEMENT

SHEET 02

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150 Jameson Drive
Peterborough, Ontario
Canada, K9J 0B9

P. 705.742.2297
F. 705.748.9944
E. wills@dmwills.com

METRIC
MILLIMETRES OR METRES (UNLESS NOTED)

GENERAL NOTES:

1. CLASS OF CONCRETE

   CSA A23.1&A23.2 EXPOSURE CLASS C1 (5-8% AIR)
   SPECIFIED 28 DAY STRENGTH: 35 MPA

2. CLEAR COVER TO REINFORCING STEEL

   FOOTINGS 100±25
   REMAINDER 70±20 UNLESS OTHERWISE NOTED

3. REINFORCING STEEL

   REINFORCING STEEL SHALL BE GRADE 400W.

   UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL BARS SHALL BE CLASS B.

   BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12-1, UNLESS INDICATED OTHERWISE.

4. CONSTRUCTION NOTES

   THE CONTRACTOR SHALL DESCALE AND CLEAN THE SPILLWAY SURFACE TO EXPOSE ORIGINAL CONCRETE PRIOR TO CARRYING OUT DELINEATION OF CONCRETE REMOVALS.

   THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS, AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWING PRIOR TO COMMENCEMENT OF THE WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.

   ALL WORK IS TO BE PERFORMED IN ACCORDANCE WITH THE OCCUPATIONAL HEALTH AND SAFETY ACT AND REGULATIONS FOR CONSTRUCTION PROJECTS.

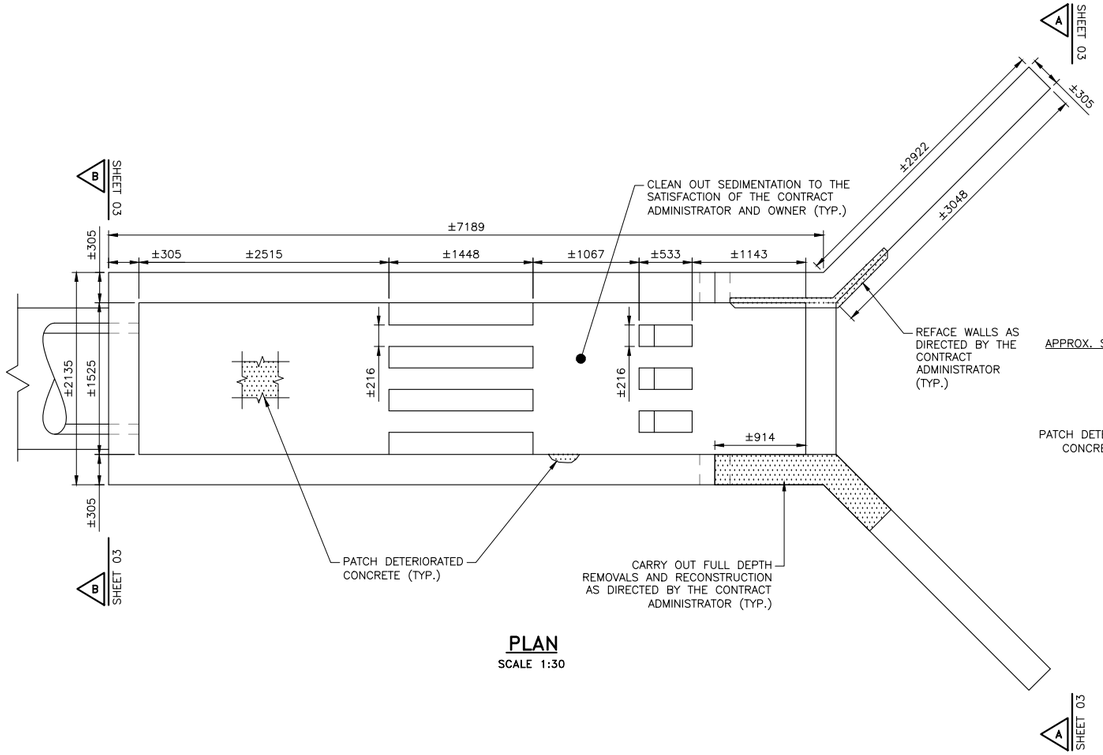
   ALL PROPRIETARY REPAIR MATERIALS SHALL BE SELECTED FROM THE DESIGNATED SOURCES OF MATERIALS LIST AND SHALL BE USED IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATIONS.

   ANY DAMAGE DONE TO THE EXISTING STRUCTURE DURING REMOVALS OR RECONSTRUCTION SHALL BE REPAIRED BY THE CONTRACTOR TO THE SATISFACTION OF THE CONTRACT ADMINISTRATOR AND AT NO COST TO THE OWNER.

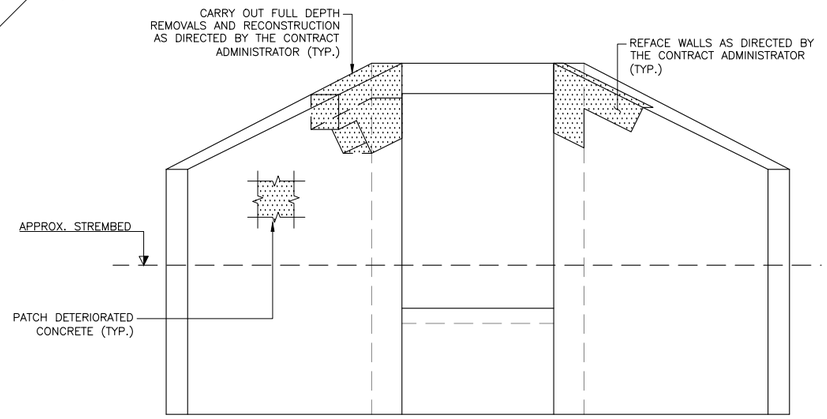
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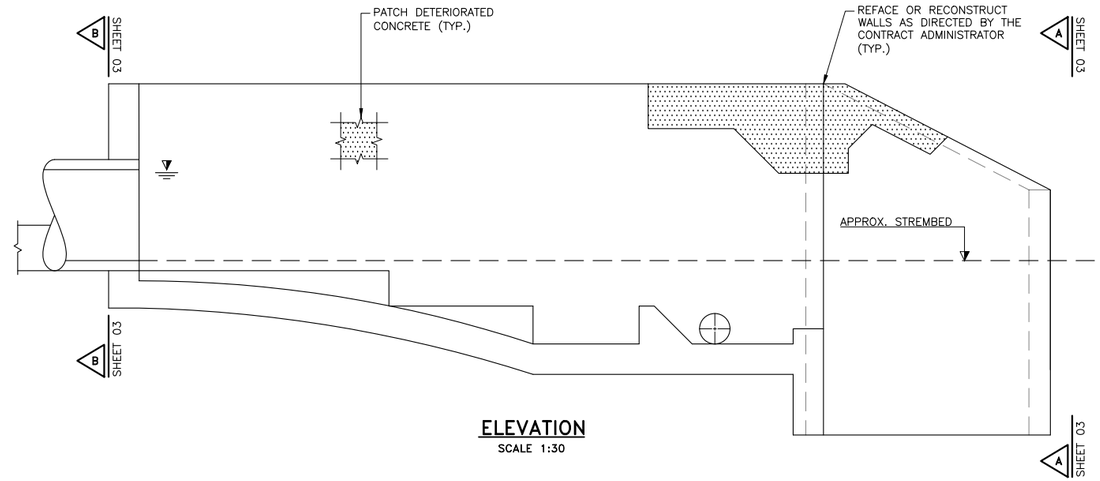
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MODIFIED: 2025-02-28 16:58
D.M. WILLS ASSOCIATES LIMITED ANSI-B 2019-11



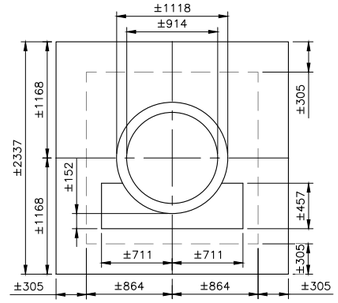
PLAN
SCALE 1:30



A SECTION A–A
SCALE 1:30



ELEVATION
SCALE 1:30



B SECTION B–B
SCALE 1:30

LONG POINT REGION CONSERVATION AUTHORITY

DEER CREEK DAM

BYPASS OUTLET STRUCTURE
GENERAL ARRANGEMENT

SHEET 03

D.M. Wills Associates Limited
150 Jameson Drive
Peterborough, Ontario
Canada, K9J 0B9

P. 705.742.2297
F. 705.748.9944
E. wills@dmwills.com

METRIC
MILLIMETRES OR METRES (UNLESS NOTED)

GENERAL NOTES:

1. CLASS OF CONCRETE

   CSA A23.1&A23.2 EXPOSURE CLASS C1 (5–8% AIR)
   SPECIFIED 28 DAY STRENGTH: 35 MPA

2. CLEAR COVER TO REINFORCING STEEL

   FOOTINGS 100±25
   REMAINDER 70±20 UNLESS OTHERWISE NOTED

3. REINFORCING STEEL

   REINFORCING STEEL SHALL BE GRADE 400W.

   UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL BARS SHALL BE CLASS B.

   BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12–1, UNLESS INDICATED OTHERWISE.

4. CONSTRUCTION NOTES

   THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS, AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWING PRIOR TO COMMENCEMENT OF THE WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.

   ALL WORK IS TO BE PERFORMED IN ACCORDANCE WITH THE OCCUPATIONAL HEALTH AND SAFETY ACT AND REGULATIONS FOR CONSTRUCTION PROJECTS.

   ALL PROPRIETARY REPAIR MATERIALS SHALL BE SELECTED FROM THE DESIGNATED SOURCES OF MATERIALS LIST AND SHALL BE USED IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATIONS.

   ANY DAMAGE DONE TO THE EXISTING STRUCTURE DURING REMOVALS OR RECONSTRUCTION SHALL BE REPAIRED BY THE CONTRACTOR TO THE SATISFACTION OF THE CONTRACT ADMINISTRATOR AND AT NO COST TO THE OWNER.

REVISIONS		
–	–	–
–	–	–
–	–	–
–	–	–
DATE	BY	DESCRIPTION

DESIGN	– –	CHK	– –	CODE	CHBDC–19	LOAD CL–625–ONT	DATE FEB 2025
DRAWN	– –	CHK	– –	SITE	– –		DWG S03

FILE NAME: U:\5600–5699\5684 – LPRCA Deer Creek Dam DSR\02 Drawings\Current Drawings\03 General Arrangement (Bypass Outlet Structure).dwg
CREATED: 2025–02–28
MODIFIED: 2025–02–28 16:57
D.M. WILLS ASSOCIATES LIMITED ANSI–B 2019–11

Deer Creek Dam - Full Rehabilitation

Preliminary Cost Estimate

Date: 16-Apr-25

Schedule A - Contract Items

Item #	Item Code	Item Description	Unit	Quantity	Estimators Unit Price	Extension
A1	SP	Mobilization and Demobilization	LS	1	$120,000.00	$120,000.00
A2	SP	Temporary Wildlife Fence	LS	1	$10,000.00	$10,000.00
A3	MUNI.182	Environmental / Watercourse Protection	LS	1	$15,000.00	$15,000.00
A4	MUNI.314	Granular A	T	1070	$100.00	$107,000.00
A5	MUNI.405	Pipe Subdrain	m	110	$200.00	$22,000.00
A6	MUNI.421	600mm Corrugated Galvanized Pipe Culverts & Grating	m	1.2	$1,000.00	$1,200.00
A7	MUNI.510	Removal of Pipe Subdrains	m	110	$50.00	$5,500.00
A8	MUNI.510	Removal of Pipes and Culverts	LS	1	$750.00	$750.00
A9	MUNI.706	Temporary Traffic Control Signs	LS	1	$10,000.00	$10,000.00
A10	MUNI.802	Topsoil, Imported	m^3	70	$100.00	$7,000.00
A11	MUNI.804	Seed & ESC Blankets	m^2	470	$15.00	$7,050.00
A12	MUNI.902	Earth Excavation for Structure	LS	1	$25,000.00	$25,000.00
A13	MUNI.902	Dewatering Structure Excavations	LS	1	$100,000.00	$100,000.00
A14	MUNI.904	Dowels into Concrete	EA	10	$85.00	$850.00
A15	MUNI.904	Concrete in Substructure	LS	1	$4,000.00	$4,000.00
A16	MUNI.905	Reinforcing Steel Bar	T	0.5	$10,000.00	$5,000.00
A17	MUNI.914	Form and Fill Grooves	m	153	$90.00	$13,770.00
A18	MUNI.928	Access to Work Area, Platforms, & Scaffolding	LS	1	$30,000.00	$30,000.00
A19	MUNI.928	Concrete Removal - Partial Depth Type A	m^3	6.7	$3,500.00	$23,450.00
A20	MUNI.928	Concrete Removal - Partial Depth Type C	m^3	1.2	$12,000.00	$14,400.00
A21	MUNI.928	Concrete Removal - Full Depth	m^3	1.7	$10,000.00	$17,000.00
A22	MUNI.930	Concrete Refacing, Form and Pump	m^3	1	$15,000.00	$15,000.00
A23	MUNI.930	Concrete Patches, Formed Surfaces	m^3	0.2	$20,000.00	$4,000.00
A24	MUNI.930	Concrete Patches, Unformed Surfaces	m^3	6.7	$2,000.00	$13,400.00

Subtotal: $571,370.00

50% Contingency: $285,700.00

Total Estimated Project Cost: $857,070.00

** Rehabilitation of County Road Bridge superstructure is not included in this estimate. Additional cost to rehabilitate the bridge is ±$1.25M w/o contingency.*

Deer Creek Dam - Spillway Rehabilitation

Preliminary Cost Estimate

Date: 16-Apr-25

Schedule A - Contract Items

Item #	Item Code	Item Description	Unit	Quantity	Estimators Unit Price	Extension
B1	SP	Mobilization and Demobilization	LS	1	$60,000.00	$60,000.00
B2	MUNI.182	Environmental / Watercourse Protection	LS	1	$15,000.00	$15,000.00
B3	MUNI.421	600mm Corrugated Galvanized Pipe Culverts & Grating	m	1.2	$1,000.00	$1,200.00
B4	MUNI.510	Removal of Pipes and Culverts	LS	1	$750.00	$750.00
B5	MUNI.517	Temporary Flow Bypass System	LS	1	$50,000.00	$50,000.00
B6	MUNI.706	Temporary Traffic Control Signs	LS	1	$10,000.00	$10,000.00
B7	MUNI.904	Dowels into Concrete	EA	4	$85.00	$340.00
B8	MUNI.904	Concrete in Substructure	LS	1	$3,000.00	$3,000.00
B9	MUNI.905	Reinforcing Steel Bar	T	0.4	$10,000.00	$4,000.00
B10	MUNI.914	Form and Fill Grooves	m	153	$90.00	$13,770.00
B11	MUNI.928	Access to Work Area, Platforms, & Scaffolding	LS	1	$30,000.00	$30,000.00
B12	MUNI.928	Concrete Removal - Partial Depth Type A	m^3	6.6	$3,500.00	$23,100.00
B13	MUNI.928	Concrete Removal - Partial Depth Type C	m^3	0.8	$12,000.00	$9,600.00
B14	MUNI.928	Concrete Removal - Full Depth	m^3	1.4	$10,000.00	$14,000.00
B15	MUNI.930	Concrete Refacing, Form and Pump	m^3	0.6	$15,000.00	$9,000.00
B16	MUNI.930	Concrete Patches, Formed Surfaces	m^3	0.2	$20,000.00	$4,000.00
B17	MUNI.930	Concrete Patches, Unformed Surfaces	m^3	6.6	$2,000.00	$13,200.00

Subtotal: $260,960.00

50% Contingency: $130,500.00

Total Estimated Project Cost: $391,460.00

Preliminary Scope

- Site access & isolation (access ramp/mud mat, d/s cofferdam, pumping)
- Environmental protection measures (silt & wildlife fencing, turbidity curtain, monitoring)
- Working platforms & ladders
- Remove and replace 600mm dia. ditch inlet pipes
- Removal of sedimentation and deteriorated concrete
- Placement of new concrete and any reinforcement / dowels
- Placement of joint filler material at transverse slab joints
- Site restoration, grading, topsoil and seed
** Rehabilitation of County Road Bridge superstructure is not included in this estimate. Additional cost to rehabilitate the bridge is ±$1.25M w/o contingency.*

Deer Creek Dam - Subdrain Replacement

Preliminary Cost Estimate

Date: **16-Apr-25**

Schedule A - Contract Items

Item #	Item Code	Item Description	Unit	Quantity	Estimators Unit Price	Extension
C1	SP	Mobilization and Demobilization	LS	1	$50,000.00	$50,000.00
C2	SP	Temporary Wildlife Fence	LS	1	$10,000.00	$10,000.00
C3	MUNI.182	Environmental / Watercourse Protection	LS	1	$15,000.00	$15,000.00
C4	MUNI.314	Granular A	T	1070	$100.00	$107,000.00
C5	MUNI.405	Pipe Subdrain	m	110	$200.00	$22,000.00
C6	MUNI.510	Removal of Pipe Subdrains	m	110	$50.00	$5,500.00
C7	MUNI.706	Temporary Traffic Control Signs	LS	1	$10,000.00	$10,000.00
C8	MUNI.802	Topsoil, Imported	m^3	70	$100.00	$7,000.00
C9	MUNI.804	Seed & ESC Blankets	m^2	470	$15.00	$7,050.00
C10	MUNI.902	Earth Excavation for Structure	LS	1	$25,000.00	$25,000.00
C11	MUNI.902	Dewatering Structure Excavations	LS	1	$25,000.00	$25,000.00
C12	MUNI.928	Access to Work Area, Platforms, & Scaffolding	LS	1	$30,000.00	$30,000.00

Subtotal: $313,550.00

50% Contingency: $156,800.00

Total Estimated Project Cost: $470,350.00

Preliminary Scope

- Site access & isolation (access ramp/mud mat, d/s cofferdam, pumping)
- Environmental protection measures (silt & wildlife fencing, turbidity curtain, monitoring)
- Working platforms & ladders
- Excavation along 55m of each side of spillway structure
- Removal and replacement of 200mm dia. subdrain
- Site restoration, backfill, grading, topsoil and seed

Deer Creek Dam - Outlet Structure Rehabilitation

Preliminary Cost Estimate

Date: **16-Apr-25**

Schedule A - Contract Items

Item #	Item Code	Item Description	Unit	Quantity	Estimators Unit Price	Extension
D1	SP	Mobilization and Demobilization	LS	1	$25,000.00	$25,000.00
D2	MUNI.182	Environmental / Watercourse Protection	LS	1	$5,000.00	$5,000.00
D3	MUNI.314	Granular A	T	50	$150.00	$7,500.00
D4	MUNI.706	Temporary Traffic Control Signs	LS	1	$5,000.00	$5,000.00
D5	MUNI.902	Earth Excavation for Structure	LS	1	$10,000.00	$10,000.00
D6	MUNI.902	Dewatering Structure Excavations	LS	1	$50,000.00	$50,000.00
D7	MUNI.904	Dowels into Concrete	EA	8	$85.00	$680.00
D8	MUNI.904	Concrete in Substructure	LS	1	$4,000.00	$4,000.00
D9	MUNI.905	Reinforcing Steel Bar	T	0.1	$10,000.00	$1,000.00
D10	MUNI.928	Access to Work Area, Platforms, & Scaffolding	LS	1	$7,500.00	$7,500.00
D11	MUNI.928	Concrete Removal - Partial Depth Type A	m^3	0.1	$15,000.00	$1,500.00
D12	MUNI.928	Concrete Removal - Partial Depth Type C	m^3	0.5	$12,000.00	$6,000.00
D13	MUNI.928	Concrete Removal - Full Depth	m^3	0.2	$12,000.00	$2,400.00
D14	MUNI.930	Concrete Refacing, Form and Pump	m^3	0.4	$15,000.00	$6,000.00
D15	MUNI.930	Concrete Patches, Formed Surfaces	m^3	0.1	$20,000.00	$2,000.00
D16	MUNI.930	Concrete Patches, Unformed Surfaces	m^3	0.1	$10,000.00	$1,000.00

Subtotal: **$134,580.00**

50% Contingency: $67,300.00

Total Estimated Project Cost: **$201,880.00**

Preliminary Scope

- *Site access & isolation (access ramp/mud mat, d/s cofferdam, pumping, blocking of inflow pipe)*
- *Environmental protection measures (silt fencing, turbidity curtain, monitoring)*
- *Working platforms & ladders*
- *Removal of sedimentation, deteriorated concrete, minor earth excavation*
- *Placement of new concrete and any reinforcement / dowels*
- *Site restoration, backfill, grading, topsoil and seed*