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Public Works Government Service Canada (PWGSC)
Talbot River Dam and Portage Lock 39
Dam Safety Review Report

FINAL

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EXECUTIVE SUMMARY

KGS Group was requested by Public Works and Government Services Canada (PWGSC) to undertake a Dam Safety Review (DSR) of the Talbot River Dam and Lock 39 which are owned and operated by PCA. The DSR was completed in accordance with the Directive for Dam Safety Program of Parks Canada Dams and Water Retaining Structures (2009).

The Talbot River Dam and Portage Lock 39 are located on the Talbot River near the Town of Bolsover, ON and are part of the Trent-Severn Waterway. The Talbot River Dam is a concrete gravity structure that includes a sluiceway with two manually operated stoplog-controlled sluices. The sluiceway is flanked on either side by concrete bulkheads that incorporate overflow spillway sections. Portage Lock 39 is a navigational lock with two closure gates. The lock is located approximately 1.4 km downstream from the Talbot River Dam.

The dam and lock are owned and operated by the Trent-Severn Waterway (TSW) of Parks Canada Agency (PCA). The dam is a regulating dam, which is operated as required to maintain the navigation water levels on the waterway and provide water control of the watershed.

The DSR included a systematic review and evaluation of all aspects of the design, construction, operation, maintenance, processes and other systems affecting the dam's safety. The DSR also included a hydrological analysis, dam break analysis to determine hazard potential classification of the dam and the Inflow Design Flood (IDF). This DSR is the first one that has been completed for the Talbot River Dam and Lock 39. This report includes a review of operator and public safety at these dams.

For both the Sunny Day and Flood conditions, the Hazard Classifications in accordance with the PCA Directive was determined to be HIGH A for the Talbot River Dam and Lock 39. Based on this classification, the recommended IDF for the dam is the flood 37% of the way between the 1000-year flood and the PMF. The required discharge capacity is 219 m^3/s. The discharge capacity of the dam is adequate to pass the IDF with the minimum freeboard required in accordance with the CDA Guidelines for the concrete sluiceway and Lock 39 but the concrete core wall of the Right and Left earthfill embankments will be overtopped by approximately180 mm.

The dam satisfies the requirement for minimum freeboard, indicated in the CDA Guidelines, during non-flood conditions. The Design Basis Earthquake (DBE) at the Talbot River Dam is the 2.500 year earthquake.

Currently, the Talbot River Dam and Lock 39 site do not meet the regulatory requirements for the following reasons:

- The concrete dam and retaining walls of the lock were found to not meet the CDA stability requirements under certain loading conditions. The spillways and Lock 39 retaining walls did not meet the stability requirements for the usual summer and winter, and earthquake loading conditions. The sluiceway did not meet the stability requirements for the usual winter loading condition nor did the gravity bulkheads for the usual summer and winter loading conditions.
- The downstream slopes of the dam's north (right) earthfill embankment and the canal's southeast (right) earthfill embankment do not meet the stability requirements for the "normal" loading condition but the factor of safeties are greater than 1.0.
- The performance of the Talbot Dam earthfill embankments are not satisfactory under the IDF condition for two reasons:
    1. The IDF water level is higher than the top of the concrete core wall and therefore during IDF conditions significant damage/wash out of the embankment fill may occur.
    2. The estimated slope stability factor of safeties do not meet the CDA requirements and are less than unity.
- Flow control equipment is not being adequately maintained.
- The flow control equipment was not confirmed to be adequate for removing all stoplogs from the sluices under high flow conditions.
- There are no Operation, Maintenance and Surveillance (OMS) manual, Emergency Preparedness Plan (EPP) and Emergency Response Plan (ERP).
- Improvements are required to provide a safer working environment.
- Public safety is at risk due to insufficient guardrails at the lock, lack of public safety warning booms, and deficient signage.

The concrete structures and flow control equipment were found to be generally in good and fair condition, respectively. There are aspects that warrant attention prior to the next dam safety review.

In general, the geotechnical performance of the Talbot River Dam and Lock 39 structures has been good, and the performance of the Talbot Canal embankments has been fair.

Recommendations for repairs have been made and prioritized. Cost estimates have been established to an approximate accuracy of +/- 40%. Costs include engineering and construction costs but do not include PCA internal costs (i.e., inspections, maintenance activities, engineering review, project management, etc.). Recommendations to improve public and operator safety have also been provided. Detailed information is available in chapter 9. In summary, the recommendations are as follows:

- In the short term (i.e., within 2 years), it is recommended to:
    - Improve foot access to the dam by installing stairs to eliminate slip and fall hazards.
    - Implement ice monitoring and inspections programs.
    - Clear the vegetation on the embankments and abutments of the dam and on the canal embankments.
    - Implement a maintenance program for the dam's flow control equipment and the lock's mechanical operating equipment.
    - Carry out a full flow test, or if not practical, demonstrate/confirm (i.e., at an alternate dam) that all logs can be safely and reliable removed under full flow conditions with the existing flow control equipment.
    - Install guarding over the exposed actuator gears and pinions of the lock's fill and drain valves.
    - Prepare an OMS Manual, and EPP and ERP plans.
    - Close the gap between the guardrails on the walkways over the lock gates.
- In the medium term (i.e., within 5 years), it is recommended to:

- Repair localized areas of concrete deterioration of the dam, lock, and retaining walls in canal.
- Install post-tensioned anchors in the dam.
- Raise the height of the concrete core walls of dam's embankments to prevent overtopping and provide sufficient freeboard during an IDF event.
- Assess the needs to or risks not to increase the stabilities of the dam's and canal's embankments and/or modify the slopes of the embankments to increase their stability.
- Install a fall restraint system on the deck for the stoplog operations.
- Install safety booms upstream of the dam, fencing, and additional signage as per the 2011 CDA Public Safety Guidelines.
- Replace existing signage within the next 5 years with signage that is in accordance with the 2011 CDA Public Safety Guidelines.
- Replace the existing guardrails along the walkways over the lock and install new guardrails along the walls of the lock chamber in accordance with the National Building Code of Canada (NBCC).

- In the long term (i.e., 6 years or more) it is recommended to:

  - Repair localized areas of concrete deterioration of the lock, and the culvert canal.
  - Install tie-back anchors in the retaining walls of the lock chamber.
  - Upgrade the guardrails on the dam in accordance with the NBCC or restrict public access to the dam.

Table E-1 summarizes the estimated costs for the recommendations.

TABLE E-1

SUMMARY OF RECOMMENDATIONS ESTIMATED COSTS

SECTION	PRIORITY			TOTAL
	Short	Medium	Long	
Main Dam (Concrete)	$150k	$720k	-	$870k
Embankments, Canal Retaining Walls, and Culvert	$120k	$950k	$120k	$1,190k
Lock (Concrete)	$20k	$350k	$300k	$670
TOTAL	**$290**	**$2,020**	**$420k**	**$2,730**

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Figure 6.3.1-2: NBCC 2010 PGA Values for Talbot River Dam and Lock 39 Site from Geological Survey Canada

Figure 6.4.3-1: 3D Model of the Critical Timbers of the Lock Gate

Figure 6.5.2-1: Simplified Stratigraphic Model Used for Right Wing Earthfill Embankment Slope Stability Evaluation

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LIST OF APPENDICES

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Appendix B - Drawings

Appendix C - Structural & Geotechnical Inspection Sheets

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Appendix J - Structural Stability Analyses

Appendix K - Structural Assessment - Stoplogs & Lock Gates

Appendix L - Geotechnical Embankment Stability Analyses

1.0 INTRODUCTION

1.1 PURPOSE AND OBJECTIVES

KGS Group was retained by Public Works and Government Services Canada (PWGSC) to undertake a Dam Safety Review (DSR) of the Talbot River Dam and Portage Lock 39 in accordance with the Directive for Dam Safety Program of Parks Canada Dams and Water-Retaining Structures, and the Dam Safety Guidelines (2007) of the Canadian Dam Association.

The purpose of the DSR is to perform a systematic review and evaluation of all aspects of the design, construction, operation, maintenance, processes and other systems affecting the dam's and Lock's safety, including the dam management system. The DSR is one component of a dam safety management system which is developed in order to ensure safe management of the dam and lock throughout its life cycle. This DSR is the first one that has been completed for this site. As per the Directive and CDA Guidelines, subsequent independent DSRs of this site should be carried out at regular intervals.

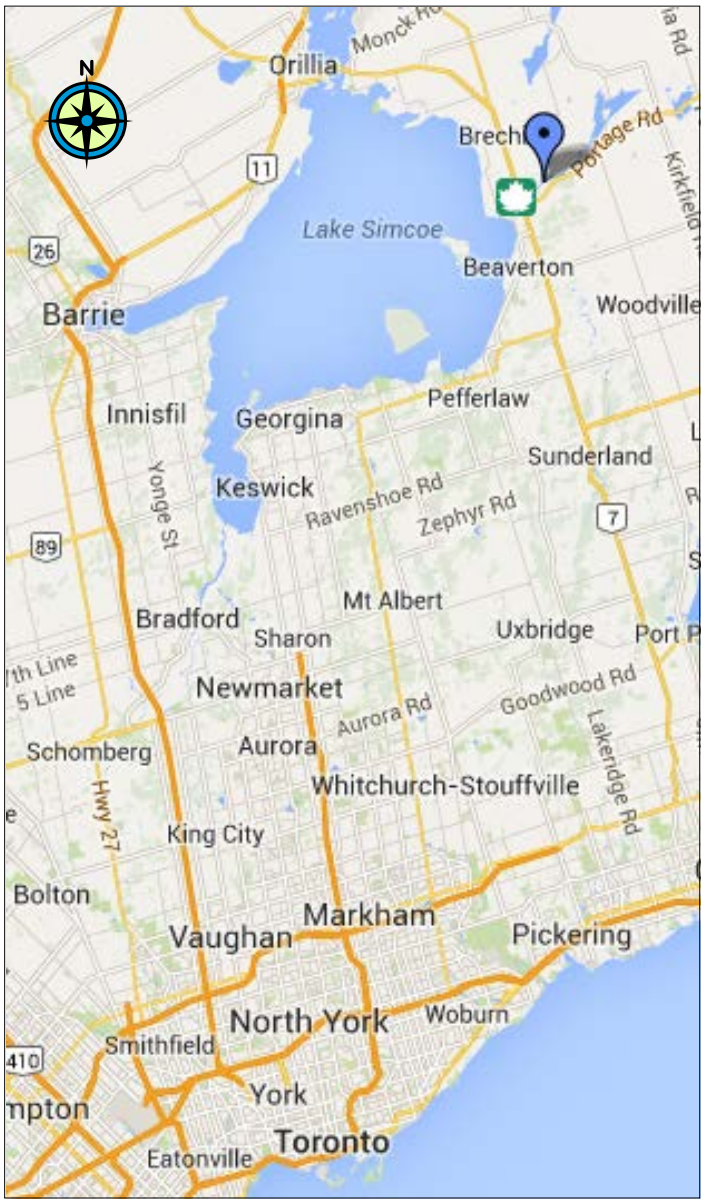
As part of the DSR, KGS Group engineers carried out a visual inspection of the structures, features of geological significance, flow control equipment, and the hydrology of the site and surrounding area.

This report presents the findings and results of the DSR and provides recommendations. In this report, "left" and "right" is referenced looking downstream.

1.2 SITE DESCRIPTION / LOCATION

The Talbot River Dam and Portage Lock 39 are located on the Talbot River are approximately 7 km West of the Town of Bolsover, ON and are part of the Trent-Severn Waterway (see Figure 1.2-1). The Talbot River watershed forms part of the headwaters of the Severn River basin and is located along the eastern edge of the basin.

FIGURE 1.2-1
LOCATION OF TALBOT RIVER DAM AND LOCK 39



The dam and lock are owned and operated by the Trent-Severn Waterway (TSW) of Parks Canada Agency (PCA). The Talbot River Dam is a concrete gravity dam and is operated to maintain the navigation water levels on the Trent-Severn Waterway and provide water control of the watershed. Portage Lock 39 is a navigational lock with two closure gates. The lock is located approximately 1.4 km downstream from the Talbot River Dam. Figure 1.1-2 shows the general layout of the dam, Lock 39 and the canal between the dam and Lock 39.

The Talbot River Dam was built in 1908 and has two sluices with stop logs and a set of two manually operated crab winches. This structure is flanked by north and south concrete gravity bulkhead structures that incorporate 6.1 m wide overflow spillways. It has a total length of approximately 48 m and the two sluices are each 7.6 m wide while the gravity section of the bulkheads are 6.7 m long. The dam has a total height of approximately 8.0 m and retains a reservoir depth of around 6.3 m.

Portage Lock 39 is a concrete gravity structure built in 1908 and has two stacked-timber mitre type gates. The lock is founded on timber piles. The lock is a single chamber with a depth of 6.55 m, length of 43.3 m and width of 10 m. Concrete walls line the canal immediately upstream and downstream of the lock.

The terminology used in this report for the site is shown on Figures 1.2-3 and 1.2-4.

FIGURE 1.2-2
SITE LAYOUT PLAN

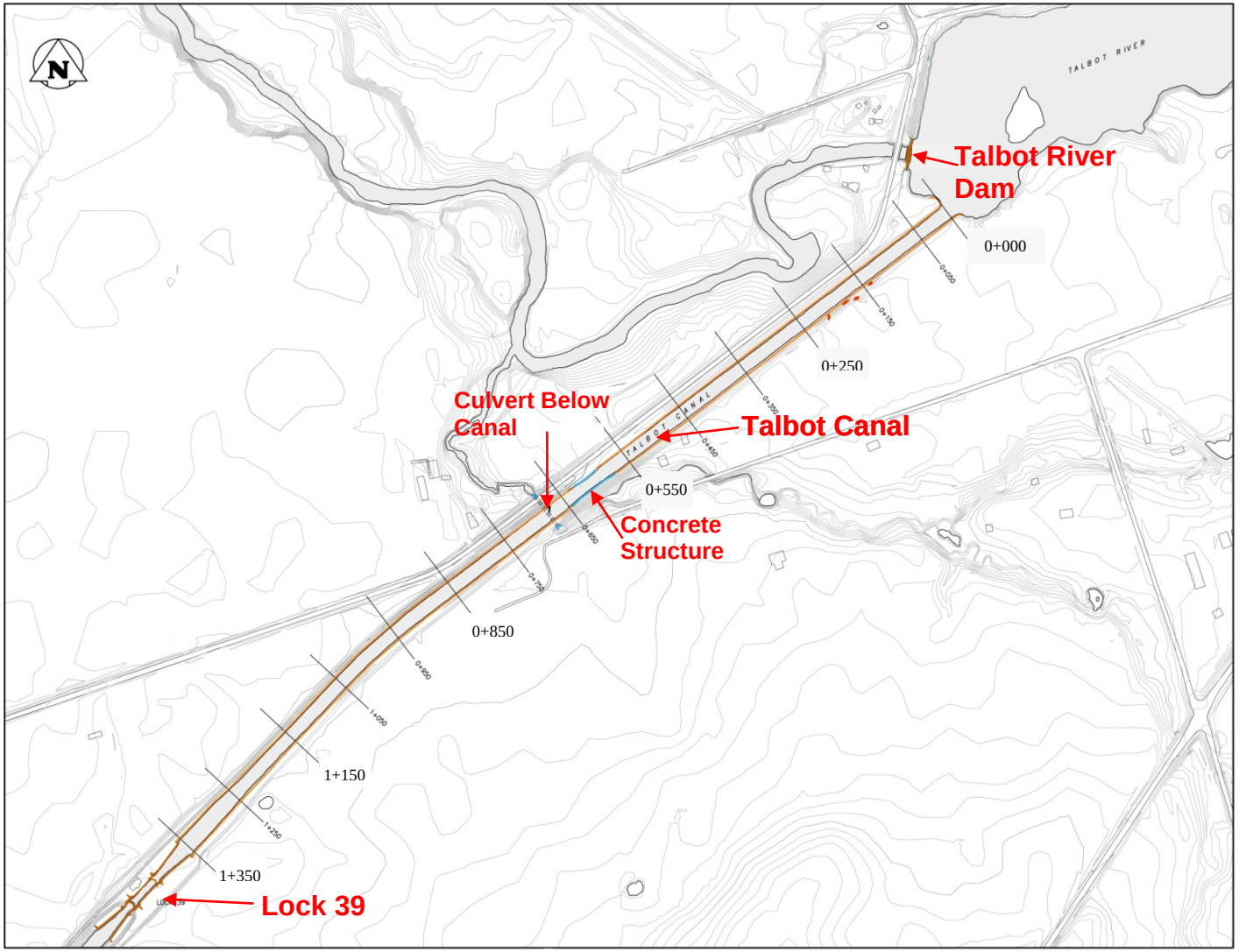
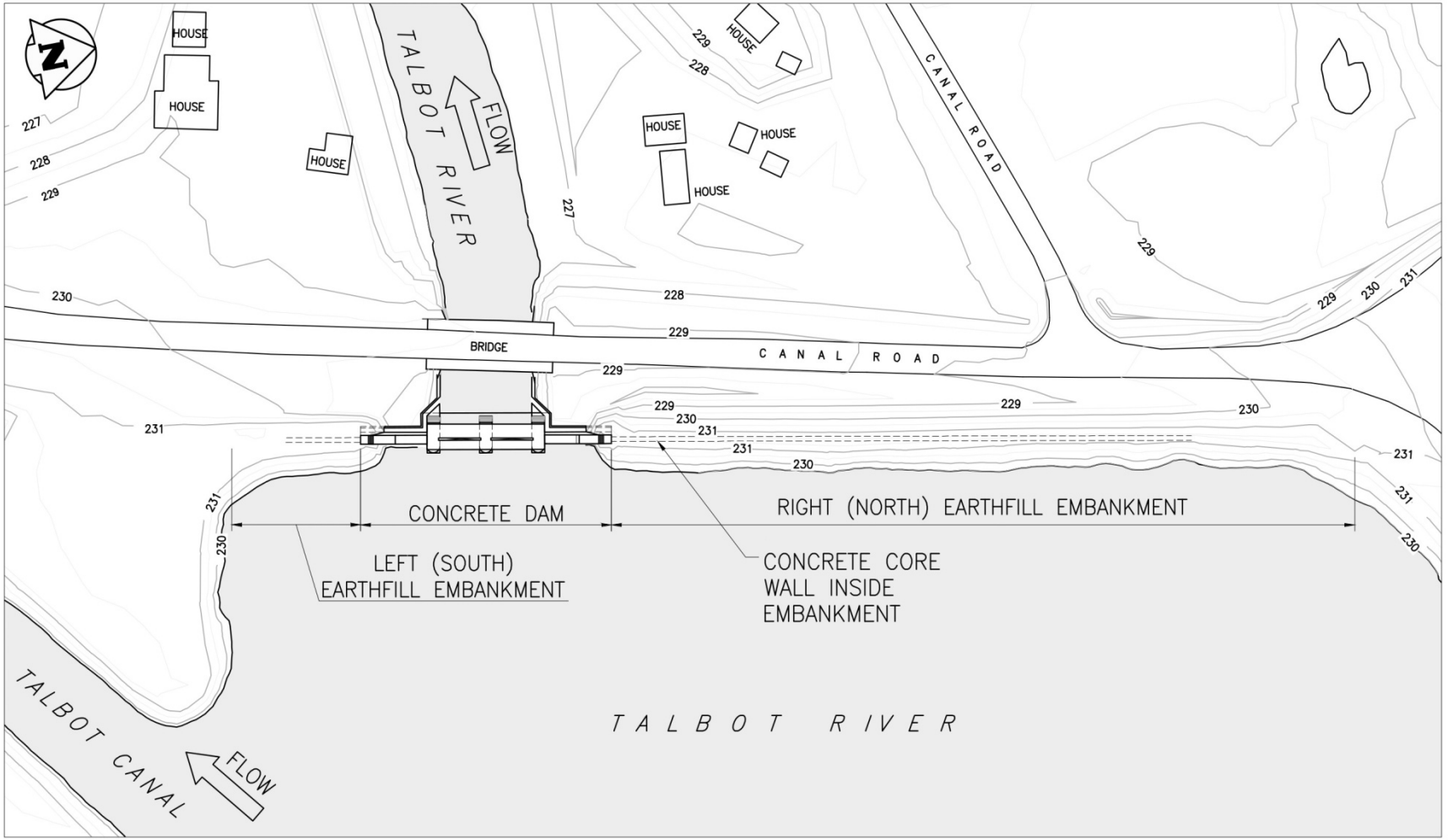
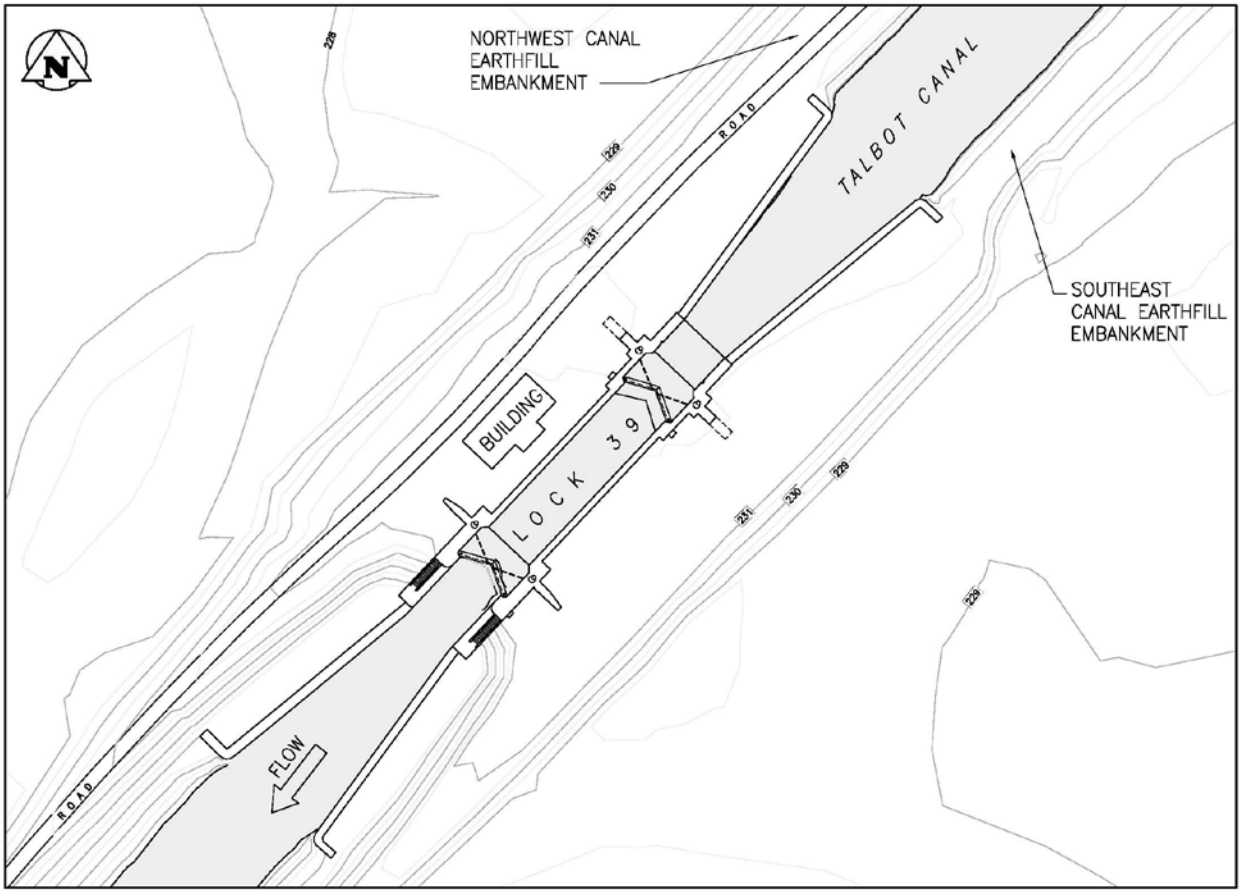


FIGURE 1.2-3
GENERAL ARRANGEMENT OF TALBOT RIVER DAM



TALBOT RIVER DAM GENERAL ARRANGEMENT PLAN

FIGURE 1.2-4
GENERAL ARRANGEMENT OF LOCK 39



LOCK 39 GENERAL ARRANGEMENT PLAN

2.0 BACKGROUND INFORMATION

2.1 GENERAL

The Central Ontario Field Unit (COFU) of Parks Canada Agency owns and operates the TSW, which is a 386 km waterway that extends from the Bay of Quinte, on Lake Ontario at the City of Quinte West (Trenton), in the south to the Georgian Bay (Lake Huron) at Port Severn in the north. The waterway traverses two major watersheds: the Trent River Watershed and the Severn River Watershed.

The Trent River basin, which drains more than 12,000 km², encompasses some 218 lakes in the Haliburton Highlands region, 47 of which are directly controlled by the TSW. Water from these lakes flows south along the Gull River, Burnt River, Nogies Creek, Mississauga River, Eels Creek and Jack Creek systems into the Kawartha Lakes (see Figure 2.1-1). The Kawarthas drain down the Otonabee River into Rice Lake and on to Trenton on Lake Ontario via the Trent River. The Crowe River drains the eastern-most lakes in the Trent Watershed. Lake Scugog drains northward from the Oak Ridges Moraine to Sturgeon Lake. The neighbouring Severn River basin drains an area just over 6,000 km². Included in this watershed are the Canal Lake - Talbot River system, the Holland River, the Lake Simcoe - Couchiching basin, the Black River and the channels of the Severn River below the hamlet of Washago. The water levels and flows on the waterway are managed by 143 dams. These dams are also owned and operated by the COFU of Parks Canada Agency (PCA). Of the 143 dams, 100 are located on the Trent River watershed, including 47 dams and water retaining structures in the Haliburton sub-watershed to control various reservoir lakes and rivers feeding the waterway. The remaining 43 dams and water retaining structures associated to the waterway are located in the Severn River Watershed.

Lakes in these watersheds are reservoir lakes for the waterway and are located within the Canadian Shield. The topography of this area mainly consists of rocky outcrops and shallow overburden giving fast response to rainfall and high run-off.

The Talbot River Dam and Lock 39 are operated by the Northern Sector of the TSW. The sector is responsible for the operation and maintenance of 28 dams and 14 locks.

Based on the Directive, PCA has assigned the Talbot River Dam with a preliminary Dam Classification of High A (see Section 5).

FIGURE 2.1-1
LOCATION OF TALBOT RIVER DAM AND LOCK 39 IN RELATION TO THE WATERSHEDS OF THE TRENT-SEVERN WATERWAY
(REF: NATIONAL HISTORIC SITE OF CANADA WATER, PARKS CANADA)

2.2 REVIEW OF EXISTING DOCUMENTATION

As part of this DSR process, KGS Group has reviewed existing drawings and studies that were provided by PWGSC. The list of referenced documents is provided at the end of this report (see Section 13).

2.3 CONSTRUCTION / REPAIR HISTORY

Talbot River Dam and Lock 39 were both built in 1908. There were no drawings available that indicated any repair work has been done to the dam and lock. The shoreline of the canal from the dam to the lock is lined with gabion baskets. The PCA terms of reference for this DSR noted that subsequent to installing the gabion baskets, sheet piling was installed along the crest of the embankments because many of the gabion baskets have rotated or their wire baskets have broken. However, the presence of the sheet piling could not be confirmed and is discussed further in Section 4.4.1.

2.4 GEOLOGY

2.4.1 Regional Geology

The topography in the Talbot River Dam and Lock 39 area is fairly rugged with numerous ridges and valleys. The relief features in the area are mostly formed by rock outcrops. Some of the valleys are occupied by lakes and ponds while other lower areas that are above water level are cultivated. Site-specific design and construction-stage geological information for the dam or lock were not available within the background data. Information on the surficial overburden and the bedrock geology has been obtained from Ontario Geological Survey (OGS) maps, assessment documents related to the local area and the site reconnaissance.

The surface geology map from Ontario Geological Survey (http://www.mndm.gov.on.ca) showed that the area is mostly overlain by coarse- textured lacustrine deposits of sand, gravel, minor silt and clay. Sandy silt to silty sand textured till and modern aluvial deposits such as clay, silt, sand and gravel are also found in the vicinity of the dam site (see Figure 2.4.1-1).

The bedrock geological map of the site, obtained from Ontario Geological Survey (http://www.mndm.gov.on.ca), identified that the site is underlain by Paleozoic limestone belonging to the middle Ordovician Simcoe Group, (Figure 2.4.1-2). This bedrock consists of mostly limestone, dolostone, shale, arkose and sandstone.

FIGURE 2.4.1-1
SURFACE GEOLOGY FROM ONTARIO GEOLOGICAL SURVEY (OGS) - TALBOT RIVER DAM, LOCK 39 AND SURROUNDING AREA

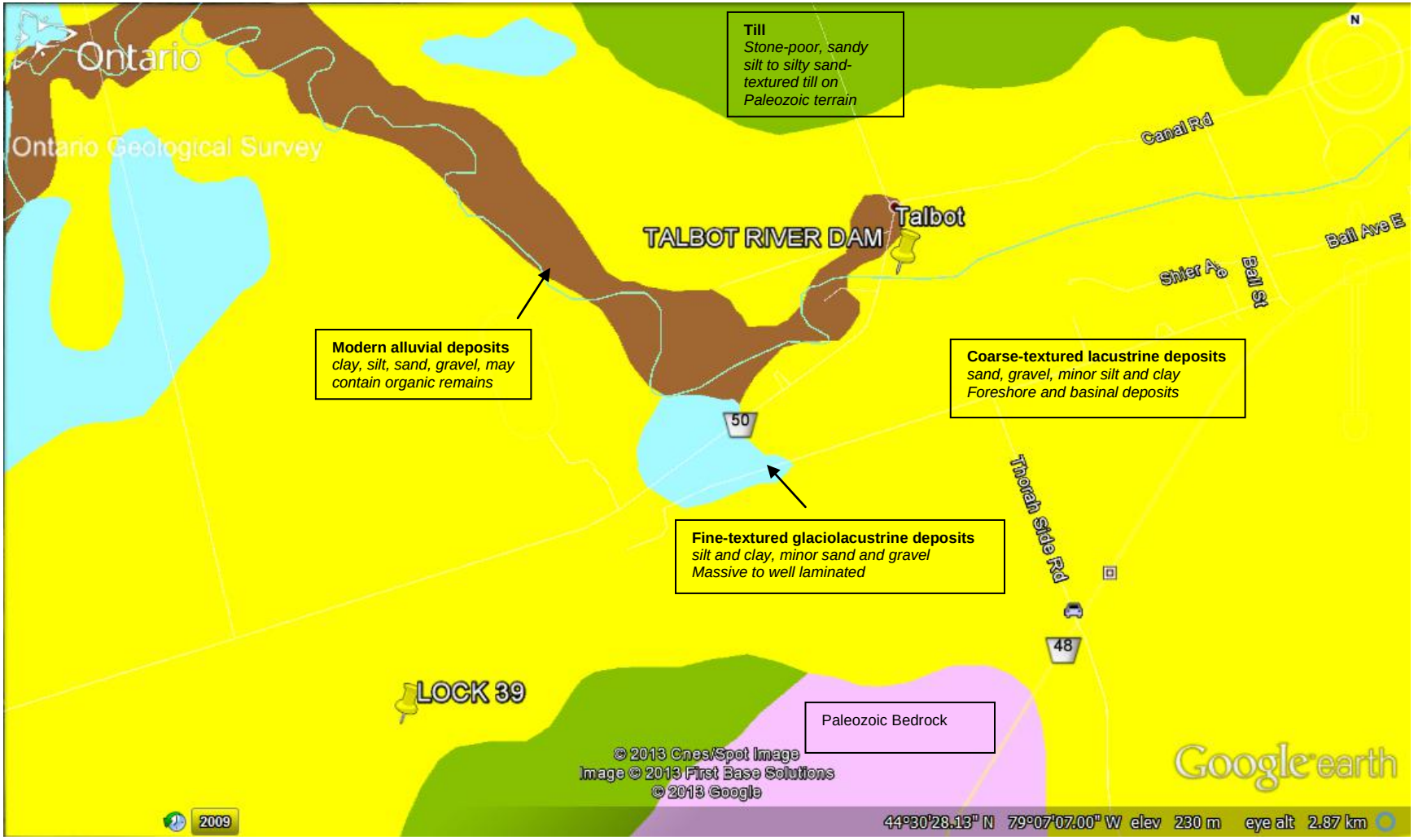


FIGURE 2.4.1-2
BEDROCK GEOLOGY FROM ONTARIO GEOLOGICAL SURVEY (OGS) - TALBOT RIVER DAM, LOCK 39 AND SURROUNDING AREA

2.4.2 Site Geology

The background information (including available drawings for the structures) indicated that the Talbot River Dam was founded on competent bedrock although the bedrock/ concrete contact interface was not visible at Talbot River Dam site. In general, this type of bedrock is competent and provides good strength and foundation support for the structures.

The background information also revealed that the Lock 39 structure was founded on end-bearing timber piles. A background search did not reveal whether the end bearing piles are founded on the underlying bedrock or competent / dense till soil.

The 2013 KGS Group site investigation program identified that the native soils encountered mainly at the site are of glacial origin and consisted of sandy silt till overlying bedrock. Native clayey silt to silty clay soil overlying sandy silt till was also encountered at Talbot Canal areas. The field investigation focused on the Talbot River Dam earthfill embankments, Talbot Canal earthfill embankments and embankment located at north side of the Lock 39 to identify overburden material (both native and fill) for use in the DSR analyses of stability and structure loading. A total of fifteen (15) test holes were drilled to the native soils/ bedrock. Coring of the bedrock was performed only at one test hole at Talbot River Dam approximately 250 mm in length. Coring identified that the bedrock encountered at the site is grey limestone and was found below the glacial till at an elevation of 225 m. A summary of the investigation program and results is provided in the geotechnical site investigation report. The report is annexed in Appendix A.

3.0 SITE DRAWINGS

Survey measurements were taken at the site as part of this DSR during the week of November 26, 2012. The survey program was carried out by T.A. Bunker Surveying Ltd., of Gravenhurst, Ontario. Detailed measurements of the various elements of the structures were made in order to update the existing drawings for the site.

The hydrotechnical dam safety analysis tasks (i.e., dam break analysis) and determining the topography of geotechnical features for the purpose of embankment stability analyses is based on the survey points collected for the site and contours obtained from Provincial Ontario Water Resource Information Program (WRIP) Digital Elevation Model (DEM). Appendix B provides "as found" drawings for the site.

4.0 INSPECTION AND DEFICIENCIES

4.1 RECORD OF OBSERVATION

KGS Group engineers visited the site on November 28, 2012 and carried out a visual inspection of the structures. The weather varied from partly sunny conditions to light snow fall and the temperature ranged from approximately -5° to 0° C. At the time of the visit, the ground was partly snow-covered. Photographic records of all the structures were made. KGS Group geotechnical engineers also performed a detailed visual inspection of the area around the dam, along the canal and the river banks and features of geological significance.

An underwater camera inspection was carried out, on December 4, 2012, of the upstream side of the face of the dam (i.e., including the gravity bulkheads and the sluice piers). The condition of the concrete below the water level, in general, appeared to be consistent with the condition of the concrete visually observed above the water level.

The observations were recorded on Dam Safety General Inspection (DSGI) sheets prepared specifically for the Talbot River Dam and Lock 39 site and are provided in Appendix C. Detailed measurements of the various elements of the structures were made in order to update the existing drawings for the dam (see Appendix B). KGS Group lead hydrotechnical engineer walked the areas upstream and downstream of the dam and lock to familiarize himself with the watershed and identified appropriate features for the hydrotechnical DSR tasks.

KGS Group inspected the flow control equipment at Talbot River Dam and the mechanical equipment of Lock 39. As part of the inspection the operations for Lock 39 was witnessed. A functional flow control equipment test was carried out by the PCA Operators on September 27, 2013 and was observed by KGS Group. KGS Group prepared mechanical inspection and functional flow test “check sheets” and they are provided in Appendices D and E, respectively. In addition, KGS Group carried out interviews with PCA Operators and went through a questionnaire with the Northern Sector Manager for the purpose of assessing the Operation Procedures and Public Safety. The completed questionnaire is provided in Appendix F.

Section 4.2 presents the inspection condition assessments for the concrete structures and Section 4.3 present the condition assessment and functional test results for the flow control equipment. Section 4.4 discusses and defines the geotechnical aspects and characteristics based on the inspection findings and review of background information.

Review of the operations procedures and operator and public safety aspects are presented later in this report in Sections 7 and 8.

4.2 DAM AND LOCK STRUCTURES CONDITION ASSESSMENT

4.2.1 General

The Talbot River Dam and Lock 39 are made up of several sections as shown in Table 4.2-1 below. The table also lists key features of the structural elements.

TABLE 4.2-1
FEATURES OF TALBOT RIVER DAM AND LOCK 39

DAM/STRUCTURE NAME	TYPE	FUNCTION	HEIGHT (m)	LENGTH (m)	CONSTRUCTION YEAR
Talbot River Dam					
Log Sluices	Concrete	Flow Control	8.0 m	22.9m	1908
South (Left) Bulkhead - Gravity Section	Concrete	Control		6.7 m	1908
South (Left) Bulkhead - Spillway	Concrete	Control		6.1 m	1908
North (Right) Bulkhead – Gravity Section	Concrete	Control		6.7 m	1908
North (Right) Bulkhead - Spillway	Concrete	Control		6.1 m	1908
Lock 39					
Chamber	Concrete	Flow Control	6.55 m	43.3 m	1908
Upstream Walls	Concrete	Shore Protection			1908
Downstream Walls	Concrete	Shore Protection			1908

The various elements of the inspected structures are discussed below. Detailed inspection observations and a detailed photographic record are provided in the DSGI sheets (see Appendix C).

4.2.2 Talbot River Dam

PHOTO 4.2.2-1
TALBOT RIVER DAM, DOWNSTREAM VIEW

PHOTO 4.2.2-2
TALBOT RIVER DAM, UPSTREAM VIEW

South Gravity Bulkhead

The South Gravity Bulkhead is comprised of two parts; the overflow section and the gravity section.

The deck surface of the gravity section is concrete and the suspended deck over the overflow section is made up of wooden boards sitting on steel I-beams.

A few areas of rotten boards were observed during the inspection. It was noted that PCA planned to replace the wooden boards next spring.

Log Sluice

The log sluice deck (see Photo 4.2.2-3) comprises wooden plank decking supported on steel I beams. Some deteriorated planks were observed (see Photo 4.2.2-4). KGS Group was advised by PCA that the deteriorated planks have subsequently been replaced.

PHOTO 4.2.2-3
SLUICE DECK

PHOTO 4.2.2-4
DAMAGED PLANK ON SLUICE DECK

A number of possible tripping hazards were identified such as stoplog supports, rail stop ends etc. These have been painted yellow to make them more visible but a possible hazard is present due to these elements.

The upstream guardrail is galvanized tubular steel with chain link infill and these are in good condition. The downstream guardrail has supporting posts made up from steel angle-iron. The horizontal elements are 3 strands of wire cable. It was noted that a number of the posts had baseplates with four holes for bolts. However, not all holes have been utilized and the stability of the guardrail may be compromised and should be checked.

PHOTO 4.2.2-5
MISSING OR OMITTED BOLTS ON GUARDRAIL BASEPLATES

The sluiceway piers are generally in good condition although some areas of erosion were observed downstream of the stoplogs, immediately above the rollway (see Photos 4.2.2-6 and 4.2.2-7). The upper section of the piers appears to be new concrete which has either been added or which is a repair of the top of the concrete. There are zebra mussels encrusting the pier surface upstream of the stoplogs.

PHOTO 4.2.2-6
UPSTREAM PIERS

There is map cracking of the concrete piers downstream of the stoplogs (see Photo 4.2.2-8). While this is possibly indicative of alkali-aggregate reaction (AAR) in the concrete there is no major stress evident although the cracks at the downstream end of the piers are open and are up to 5 mm wide. White deposits along some cracks and horizontal joints also suggest the possibility of AAR in the concrete. Note that the presence of AAR does not necessarily mean that there is a problem here. It could be a very slow reaction or the reaction could already have ceased. While the piers should be monitored, no action is required at this time.

Based on a visual inspection from the deck and from the downstream side of the dam, the gains appeared to be in good condition except in one location. The south gain of the north sluice is missing some concrete adjacent to the downstream steel liner (see Photo 4.2.2-9).

PHOTO 4.2.2-7
SLUICEWAY

PHOTO 4.2.2-8
SOUTH PIER, CRACKS

PHOTO 4.2.2-9
DAMAGED CONCRETE ON NORTH SLUICE, SOUTH GAIN

North Gravity Bulkhead

The bulkheads are each comprised of two parts. There is a gravity portion and an overflow portion. The overflow portion is adjacent to the sluiceway and has a wooden deck supported on

steel I beams. The top of the bulkhead deck was not visible due to snow conditions. The steel beams supporting the overflow section deck appear to be in good condition with good seatings.

PHOTO 4.2.2-10
NORTH BULKHEAD DECK

PHOTO 4.2.2-11
UNDERSIDE OF NORTH BULKHEAD DECK

The upstream face of the bulkhead is in good condition but there are areas where the concrete has spalled or is about to spall. These areas are located along horizontal construction joints. There is map cracking evident in the concrete face. The cracks appear to be relatively tight in the lower areas but the cracks towards the top of the structure are open. There are white deposits along the cracks which can be expected to be carbonated silica gel or calcite. Horizontal joints just below rollway level and approximately 0.6 m below deck level are open and there is spalling of the concrete along these joints. The upper 0.3 m of the mass section appears to be a new concrete cap.

PHOTO 4.2.2-12
UPSTREAM VIEW OF BULKHEAD

The downstream face of the bulkhead is in fair condition. There is one area of spalled concrete and some minor vegetation growth. There is evidence of seepage through the bulkhead as the lower face was wet with algae growth present and there was some water in the adjacent drainage channel. The drainage channel immediately downstream of the bulkhead was in generally good condition but, towards the outlet end at the tailrace the channel is undermined and in poor condition. The condition of the outlet end of the drainage channel should be monitored and should be repaired within the next 10 years.

PHOTO 4.2.2-13
NORTH BULKHEAD, DOWNSTREAM VIEW

PHOTO 4.2.2-14
DRAINAGE CHANNEL NORTH SIDE

4.2.3 Lock 39

Lock 39 is a concrete gravity structure founded on timber piles. The actual chamber is 43.3m long by 10 m wide but the concrete walls continue upstream and downstream of the lock proper and at the downstream end, adjacent to the gates there is a concrete bulkhead structure with stairs on each bank of the canal. The wooden lock gates were found to be in good condition with no obvious deterioration observed.

PHOTO 4.2.3-1
LOCK 39, LOOKING DOWNSTREAM (WEST)

4.2.3.1 South Wall

Within Lock

The face of the south wall is generally in good condition. At the downstream end, the wall has had some concrete repairs in the area close to the gate operating mechanism and these repairs are failing with areas of concrete spalling. There are also areas of spalled concrete along the foot of the lock wall, particularly towards the downstream end of the lock. There is a large spalled area approximately midway along the lock. These deteriorated areas should be repaired within the next 2 to 4 years. There is some erosion near the high water line near the lock gates that is likely due to boats abrading against the lock wall.

The vertical joints within the lock remain in good condition.

PHOTO 4.2.3-2
SOUTH WALL LOOKING UPSTREAM (EAST)

PHOTO 4.2.3-3
SOUTH WALL, DOWNSTREAM END

Upstream of Lock

Upstream of the lock, there is deep erosion at the high water line which is probably related to abrasion from boat operations in the canal. All along the wall there is spalling of concrete along a joint just above winter water level. At several locations along the wall, the deterioration expands into large, deep freeze/thaw affected spall areas. The concrete here is reduced to rubble to a significant depth. The observed deterioration should be addressed within the next 2 to 4 years.

PHOTO 4.2.3-4
APPROACH CHANNEL SOUTH WALL

PHOTO 4.2.3-5
UPSTREAM END, SOUTH APPROACH CHANNEL WALL

Downstream of Lock

The wall downstream of the lock is visually in good condition.

The bulkhead at the downstream gates is map cracked with some small areas of spalling just below the concrete cap. There is extensive build-up of white deposits on the face of the concrete along cracks and horizontal joints. The cracking and white deposits suggest that alkali-aggregate reaction (AAR) may be present in the concrete and that there is moisture movement through the concrete, although there is no clear evidence of any movement or consequent distress in the structure.

PHOTO 4.2.3-6
SOUTH DOWNSTREAM WALL

PHOTO 4.2.3-7
SOUTH DOWNSTREAM BULKHEAD

4.2.3.2 North Wall

Within Lock

The face of the north wall is generally in good condition. There are, however, a number of areas of deterioration. Most of the damage is at the base of the wall where there are areas of

freeze/thaw deterioration. This is likely caused by movement of moisture from the ground behind the wall through the concrete. This is saturating the concrete and, in the winter, the wall face is exposed to freezing and thawing cycles that are breaking down the concrete. This deterioration should be addressed within the next 2 to 4 years.

Near the gates, there is minor waterline erosion of the concrete which is probably due to abrasion from the boats operating in the canal in the summer.

At the downstream end of the wall has had some concrete repairs in the area close to the gate operating mechanism and these repairs are generally intact with areas of concrete spalling present. There are also areas of spalled concrete along the foot of the lock wall, particularly towards the downstream end of the lock. There is a large spalled area approximately midway along the lock. These deteriorated areas should be repaired within the next 2 to 4 years.

The vertical joints within the lock remain in good condition

PHOTO 4.2.3-8
NORTH WALL LOOKING DOWNSTREAM (WEST)

PHOTO 4.2.3-9
NORTH WALL, DOWNSTREAM END

Upstream of Lock

Upstream of the lock, there is shallow erosion at the high water line. Just below the winter water level there is an open horizontal joint with associated areas of freeze/thaw spalling. There are also a number of large areas of spalled concrete where the deterioration appears to extend deep into the concrete. These areas should be repaired in the next 2 to 4 years. Apart from these issues the wall is in good condition.

PHOTO 4.2.3-10
UPSTREAM NORTH WALL

Downstream of Lock

The wall downstream of the lock is generally in fair condition. At the upstream end, particularly close to the lock gates, the concrete at high water level is eroded, likely due to abrasion from boats during lock activities. There is one badly spalled and fractured area in the vertical face near a construction joint immediately downstream of the foot of the stairway.

PHOTO 4.2.3-11
NORTH DOWNSTREAM WALL

The bulkhead at the downstream gates is map cracked with minor areas of spalling concrete. There is some build-up of white deposits on the face of the concrete along cracks and horizontal joints. The cracking and white deposits suggest that alkali-aggregate reaction (AAR) may be present in the concrete and that there is moisture movement through the concrete, although there is no clear evidence of any movement or consequent distress in the structure.

PHOTO 4.2.3-12
NORTH DOWNSTREAM WALL BULKHEAD

4.2.4 Culvert

There is a large concrete culvert that runs under the canal near Lock 39 (between the lock and Talbot River Dam). As part of the project scope, KGS Group was requested to inspect this culvert. In addition to the actual culvert structure, the canal is lined with concrete retaining walls that run upstream and downstream of the culvert location.

4.2.4.1 Description

The north end of the culvert has vertical concrete sides and an arch ceiling. The concrete at this end is in very good condition. Some evidence of moisture movement and calcite deposits was observed. The floor of the culvert appears to be rock. There is a lot of debris, loose rock and sediment on the floor. A few metres into the structure and extending through to the south end, the structure changes to a rough concrete vertical wall (approximately 600 mm tall surmounted by a corrugated, galvanized steel arch.

Water runs northwards through the culvert and is typically 200 mm deep along the culvert.

PHOTO 4.2.4-1
CULVERT, NORTH END

PHOTO 4.2.4-2
CORRUGATED STEEL SECTION OF THE CULVERT

4.2.4.2 Condition

At the interface between the wing wall and the culvert and the entrance arch at the north end there is a large area of deteriorated concrete at and above the water level. This appears to mostly be due to freeze/thaw mechanisms.

PHOTO 4.2.4-3
SPALLED CONCRETE AT NORTH END OF CULVERT

The section with the concrete arch is generally in good condition with some minor cracking and calcite deposits.

The galvanized arch is generally in good condition; the concrete support wall is in fair to poor condition. There is extensive shallow spalling of the concrete and a number of areas where the concrete has eroded and spalled to significant depths, leaving large voids beneath the arch. The thickness of the concrete is unknown and should be confirmed by PCA. The importance of this deterioration is dependent on how thick the wall is, but as a minimum, it would be advisable to remove the spalled/deteriorated concrete and replace with new concrete and ensure the integrity of the supporting structure within the next 3 to 5 years.

PHOTO 4.2.4-4
ERODED/ SPALLED CONCRETE AT BASE OF CULVERT

At the south end of the culvert the concrete wing walls are in fair to good condition but there is extensive spalling of the concrete around the arch and up along the vertical joint above the arch. This deterioration would not be considered structurally significant at this stage but reinforcing steel is exposed and ongoing deterioration can be expected. The source and cause of the concrete deterioration should be assessed and appropriate repairs should be carried out at the inlet and outlet of the culvert within the next 3 to 5 years.

PHOTO 4.2.4-5
SOUTH END OF CULVERT

(Note concrete spalling around arch and along vertical joint.)

4.2.5 Concrete Retaining Walls – Canal Embankment

The embankment retaining walls on both sides of the canal in the vicinity of the culvert are in very poor condition. Both walls (north and south bank) have extensive areas of deterioration. On the south bank, there are a number of deep voids formed by concrete freeze/thaw mechanisms and associated erosion processes. While these are mainly at low water level, there is also extensive spalling at vertical joints and fractures, vegetation growth, and high waterline erosion.

PHOTO 4.2.5-1
TYPICAL CONDITION OF SOUTH BANK RETAINING WALL

The north bank retaining wall also has deep voids caused by combined freeze/thaw and erosion mechanisms acting on the concrete. In addition, there is a large area of the face that appears to have deep freeze/thaw deterioration which is causing the surface concrete to spall from the wall.

PHOTO 4.2.5-2
NORTH EMBANKMENT RETAINING WALL

4.2.6 Conclusions and Recommendations – Structures Condition

In general, the concrete was found to be in good condition but the following issues were noted and require attention:

Talbot River Dam

North Gravity Bulkhead Drainage Channel: The condition of the concrete at the outlet end should be monitored and eventually repaired within the next 10 years.

Lock 39

Walls Within and Upstream of Lock: Spalled/deteriorated areas should be repaired within the next 2 to 4 years.

Walls Downstream of Lock: Monitor the spalled/deteriorated areas, especially the spalled and fractured area in the vertical face near a construction joint immediately downstream of the foot of the stairway, with an expectation to repair within the next 10 years.

Culvert

Inlet and Outlet and Adjacent Retaining Walls: Repair the deteriorated concrete within the next 3 to 5 years.

4.3 FLOW CONTROL EQUIPMENT CONDITION AND FUNCTIONAL ASSESSMENT

4.3.1 General

The assessment of the mechanical equipment is based on:

- visual inspections carried out on November 28, 2012 and September 24, 2013;
- a stoplog sluice equipment functional test carried out on September 24, 2013;
- observations of lock operations made on September 24, 2013;
- review of the completed operations questionnaires;
- available documentation such as drawings, manuals, etc; and
- interviews and correspondence with PCA staff.

The objective of the functional test was to determine if the equipment is in good operating condition. For the log sluice equipment functional test and the lock operations assessment, the equipment was operated by PCA staff. Both events were witnessed and recorded by KGS Group.

A full flow test where all the stoplogs are removed from one sluice under high flow conditions could not be carried out because of PCA water management constraints. Thus, the adequacy of the flow control equipment to remove all stoplogs under high flow conditions could not be confirmed.

4.3.2 Condition Assessment – Log Sluice

4.3.2.1 Equipment Description

The mechanical equipment at the Talbot River Dam consists of a set of two traveling, manually operated crab winches, two stoplog sluices with 10 wooden logs in each sluice and one steel half-stoplog (see Photo 4.3.2-1). The winches travel on a set of rails spanning across the two sluices.

PHOTO 4.3.2-1
TALBOT RIVER DAM DECK SHOWING WINCHES, SLUICE AND RAILS

The dam was built in 1908 and the two crab winches are over 100 years old. Their design lifting capacity is unknown. Each winch consists of a cast steel frame mounted on four wheels. A gear train drives a winch (see Photo 4.3.2-2). The gear train has a set of two open, unguarded, spur type pinion/gear assemblies. The winch drum holds a wire rope fastened to a lifting hook that engages the log ends. The winches are operated by removable hand cranks which are not dedicated to this dam but are shared with Talbot Dam.

PHOTO 4.3.2-2
CRAB WINCH

The winch frames are secured to the dam deck with linkages on the frames that are connected with pins to eyes embedded into the concrete piers. The pins are pad locked in place to safeguard against vandalism (see Photo 4.3.2-3). The pins are chained to the winch frames to prevent losing them when the pins are removed during moving of the winch frames between sluices.

PHOTO 4.3.2-3
LOCK FOR CRAB WINCH

The wooden stop logs range in age from 5 years to 30 years old. They are 356 mm wide x 305 mm high x 8.13 m long and have a steel U-bolt installed at each end for engagement and handling (see Photo 4.3.2-4). The single steel half-stoplog is 150 mm x 150 mm in cross section x 8.13 m long and has lifting bars welded into the ends of its structure (see Photo 4.3.2-5).

PHOTO 4.3.2-4
WOODEN STOPLOG END FITTING

PHOTO 4.3.2-5
STEEL HALF-STOPLOG

Sluice gains are concrete with steel liners on the upstream and downstream faces.

Sluice crossers are placed across the sluice opening to allow lowering of a log on to the deck after it is raised out of the sluice. The crossers are chained to the winch frames to prevent loss into the sluice during log operations and to prevent removal of the crossers by vandals.

Each of the two sluice openings in the dam deck is covered using an out-of-service wooden stoplog. Each log is supported from the deck using three 14 mm thick steel support brackets (see Photo 4.3.2-6).

PHOTO 4.3.2-6
STOPLOG SLUICE COVER AND SUPPORT BRACKET

Electrical equipment at the Talbot River Dam associated with the dam operation is limited to a water level gauge. The gauge system was installed in 2004. The gauge is mounted on the upstream face of the dam and powered by a single phase electrical service. Water level readings are automatically recorded hourly and sent via hard wire to the Peterborough headquarters. Readings are also e-mailed once a day to the water control engineer and to the Northern Sector manager.

Portable equipment used at the Talbot River Dam is shared with the Talbot Dam and is stored at the Lock 38 control building. Pike poles are used to guide the winch lifting hooks down the

sluice gains to engage the stoplog end fittings. Peavies are used to manipulate the stoplogs on the dam deck. A hydraulic jack and wooden jacking blocks are available to compress the stoplogs once installed in the sluice. Jacking is typically not required and not performed at this dam except when installing stoplogs for over the winter.

There is no formal written maintenance program or records of maintenance performed for the equipment associated with the Talbot River Dam. Operators lubricate the winches periodically. The automated water level gauge system is virtually maintenance free and is calibrated every two years. Operators typically identify what needs to be repaired or requires maintenance and Northern Sector maintenance staff perform repairs in the fall of each year.

4.3.2.2 Inspection Results

The winch frames appear to be structurally sound and their protective coating is mostly intact. They are well secured and pad locked to the deck. The teeth on the gears have minor wear and are not lubricated (see Photo 4.3.2-7). Some gears are not fully guarded. Gear teeth break occasionally but mostly due to vandalism. Shaft journals are well lubricated. The wire ropes, rope end fittings and drums are all in good condition but the wire rope is not lubricated (see Photo 4.3.2-2). Winches operated well during the September 2013 functional testing with no unusual noise or excessive effort. Suppliers of the winches no longer exist. Replacement parts need to be custom made. However, due to the ruggedness and simplicity of the winches, the chance of an unexpected component failure is very low and PCA does stock some spare parts.

PHOTO 4.3.2-7
PINION AND SPUR GEAR

The stoplogs are 5 to 30 years old. There were 11 wooden stoplogs on the deck during the November 2012 inspection. All appeared in good condition and free of rot. The ends of one of the logs were slightly damaged. The steel U-bolts were all solidly attached to the ends of the logs. There is one spare log at the dam and five spares for the region are stored at the Kirkfield shop.

The steel half-stoplog was structurally sound and well coated.

As noted in Section 4.2.2, the gains appeared to be in good condition except in one location. The south gain of the north sluice is missing some concrete adjacent to the downstream steel liner.

Sluice crossers are in good condition. Some protective coating is missing in wear areas but this is of no concern.

Sluice cover stoplogs are in good condition. The log support brackets are in good condition except for protective coating missing in some areas.

The rails are in good condition and have stops at their ends. However, they are not adequately secured to the deck and not well aligned. In some areas, there is 2 to 2.5 m between rail clips (1 m maximum is recommended), some rail clips are loose, some clips are not clamping on the rail and some clips are missing (see Photo 4.3.2-8 and 4.3.2-9). Also the rails are misaligned over the north sluice (see Photo 4.3.2-10).

PHOTO 4.3.2-8
RAIL CLIP NOT CLAMPING RAIL

PHOTO 4.3.2-9
MISSING RAIL CLIPS

PHOTO 4.3.2-10
MISALIGNED RAIL OVER NORTH SLUICE

The water level gauging system appears to be in good condition and is well secured to the upstream face of the dam. The enclosure access door is locked. The gauge is calibrated every two years by the water control engineer and requires very little maintenance. The system is reliable and in good functioning condition. However, in cold weather it freezes and at that time provides inaccurate data.

Winch handles and peavies used during the September 2013 equipment functional test functioned well and are in good condition. Pike poles, the log jack and wooden jacking blocks were not used during the test but those inspected in storage at the Lock 38 control building appeared to be in good condition. The jack exhibited no oil leaks and PCA staff reported that there are no known operating problems with the jack or any jacking ancillary equipment.

4.3.3 Functional Test Description and Results

Functionality of the stoplog sluice equipment at Talbot River Dam was tested by removing two logs from the south sluice at the dam and then re-installing them. The steel half stoplog was not used during the tests. Detailed test record sheets are included in Appendix E.

The water level upstream of the dam, at the start of the log removal and log installation test, was 230.427 m and remained the same throughout the testing. The weather conditions were sunny and the temperature was 14 °C. Both tests considered the full sequence of operations including:

- loading portable equipment on trucks at the Kirkfield shop;
- travelling to site including stopping at Lock 38 on the way to pick up additional portable equipment and the winch crank handles;
- site preparation;
- monitoring for persons upstream and downstream of the dam;
- log removal and installation (two logs from one of the two sluices);
- restoring the site to secure conditions;
- returning portable equipment to Lock 38 storage on the way back to the shop;
- travelling back to the shop; and
- returning portable equipment to storage at the shop.

The log removal operation went well and without incident. There is a potential slip and fall hazard when walking from the truck to the dam through a ditch that has very steep embankments, especially if the ground is wet. All portable and permanent equipment performed well. There is a concern of a potential pinch hazard due to gears not being fully guarded.

Based on the functional test completed with two logs, the total estimated time required to remove two wooden stoplogs in daylight and good summer weather conditions would be 57 minutes. The breakdown of the 57 minutes is as follows:

- 17 minutes to obtain portable equipment from the Kirkfield shop and to travel to Lock 38;
- 3 minutes to obtain additional portable equipment from Lock 38 and to drive to the dam;
- 17 minutes working at the site which includes 5 minutes to warn kayakers/boaters downstream and upstream of the dam of the pending log sluice operations; and
- 3 + 17 minutes to return portable equipment to storage at Lock 38 and to travel back to the Kirkfield shop.

Average time for individual log removal is 2.5 minutes per log for the top two logs in the sluice. Note that removal time per log will increase with progressively deeper logs. There was little water overtopping the two logs removed and pike poles were not required to engage lifting hooks into the ends of the logs. However, pike poles would likely be required as more logs are removed and the water depth overtopping the logs in the sluice increases. This will increase removal time per log. Also, the extra lifting distance will add 5 to 10 seconds to each progressively deeper log being removed as well.

The log installation operation also went well. All portable and permanent equipment performed well. As with the log removal operation, there is a concern of a potential pinch hazard due to gears not being fully guarded. Based on the functional test completed with two logs being installed, the total estimated time required to install two wooden stoplogs in daylight and good summer weather conditions would be 52 minutes. Breakdown of the 52 minutes is the same as for the removal operation except there were no people upstream or downstream of the dam to warn of pending sluice operations. Average time for individual log installation is 2.5 minutes per log for the top two logs in the sluice. Log installation time would not increase notably for logs deeper in the sluice since the winches are not used in their conventional manner to lower the logs during installation. Logs are lowered just below deck level and then the winch drum latches are released and the log drops into the sluice under gravity with the winch free-wheeling.

No jacking down of the stoplogs was carried out during the functional test. Based on discussions with the Operators, jacking is not required during the boating season and is only performed when adjusting the number of logs for winter set up.

4.3.4 Lock 39 Assessment

4.3.4.1 Equipment Description

Lock 39 was built in 1908. It is a navigational lock and has two closure gates to vary the water level in the lock chamber to match the upper and lower approach channels. The closure gates are the double-leaf mitre type and are made of horizontal wooden timbers framed by two vertical wooden members at either end. The horizontal timbers for each gate leaf are secured together with eight vertical steel tie rods and one diagonal tie rod (see Photo 4.3.3-1).

PHOTO 4.3.3-1
LOCK 39 DOWNSTREAM GATES

Both upstream gate leaves also have vertical wooden members near the bottom of the leaves on the downstream side (see Photo 4.3.3-2).

PHOTO 4.3.3-2
LOCK 39 UPSTREAM GATE LEAVES WITH
VERTICAL WOODEN MEMBERS NEAR GATE BOTTOM

Each gate leaf is supported by a gudgeon at the top and rotates on the gudgeon pin and a pintle bearing at the bottom. The gudgeon is adjustable and is secured to the concrete walls of the lock with two horizontal steel tensioning rods.

Each gate leaf is operated by its respective hand operated actuator. Each actuator turns a wire rope sheave located below grade. The wire rope in turn drives a gate actuating arm to open and close the gate (see Photo 4.3.3-3).

PHOTO 4.3.3-3
LOCK 39 UPSTREAM GATE ACTUATOR

Each gate leaf has a horizontally mounted butterfly valve located near the bottom of the gate leaf for draining or filling of the lock (see Photo 4.3.3-1). A manual actuator for each valve is located on the top of the gate leaf and is accessed on the gate leaf walkways. The valve actuator consists of two partially guarded, parallel pinion/gear sets mounted on a steel frame. The bottom pinion/gear set has a pinion that engages a vertical rack connected to a vertical steel rod which raises and lowers to open and close the valve (see Photo 4.3.3-4).

PHOTO 4.3.3-4
MANUAL VALVE ACTUATOR

There is no electrical equipment used to operate Lock 39 other than lighting.

4.3.4.2 Inspection Results

The upstream gate at Lock 39 was replaced 10 years ago and the downstream gate was replaced 30 years ago. The upstream and downstream gates are both in good condition. There is no leakage between the timbers on the upstream gate leaves and very minor leakage through the timbers in one of the leaves of the downstream gate. Based on visual inspection, the diagonal metal tie rods are in good condition. Vertical tie rods could not be accessed for inspection and their condition is unknown. Only the top of the gudgeon is visible but looks to be in good condition and had evidence of recent lubrication. Gudgeon diagonal tensioning rods are secure and in good condition. The pintle bearings at the base of the gate leaves could not be accessed for visual inspection but all gate leaves operate smoothly without any unusual noise or vibration indicating that the gudgeon pins and pintle bearings are in good operating condition. There is no leakage at the centre, bottom and outer seals of all gate leaves indicating that the seals everywhere are in good condition.

The gate actuators are in fair condition. The stands and actuator hand wheels are well coated with paint. There is excessive play between the hand wheel hub and the square actuator drive

shaft. This is accelerating wear at this interface and also allows the hand wheels and their extensions to tip off-horizontal presenting an ergonomic problem for average to tall people. Also, while operating the gate actuators, the Operators must repeatedly walk close to the edge of the deck of the lock. Hand wheels are not locked when Operators are off site and could easily be removed by vandals. The gate actuating arms are in good condition and generally well coated. Wire ropes are in good condition. Sheaves were not accessible for visual inspection but operate well. The actuators do, however, require a high level of force to get the gates moving from a stopped position. The lock gate leaves are chained and pad locked together overnight to prevent gate actuation by vandals but are not locked during the day when Operators are not present.

The two lock filling valves in the upstream gate were refurbished 10 years ago and two draining valves in the downstream gate were refurbished 30 years ago. Valves in the downstream gate and all four valve actuators are in good condition. However, the valves in the upstream gate leak badly even when a high force is exerted by the Operators at the valve actuators (see Photo 4.3.3-5). All coatings are in good condition. Valve actuator gears and pinions are well lubricated. Gears and pinions are not fully guarded. Supports for the valve actuators, operating shafts and valves are sound. Valve actuators operate well and valves move smoothly. The potential for vandalism exists as there are no means to pad lock the valve actuators to prevent undesired valve operation when Operators are not present. Also, even though handles for all valve actuators are removed and securely stored by staff overnight, they can easily be removed by vandals during the day when Operators are not present.

PHOTO 4.3.3-5
LEAKING FILL VALVES IN UPSTREAM GATE

4.3.5 Conclusions and Recommendations – Mechanical Equipment

Witnessing the flow control equipment functional test provided sufficient evidence to confirm that PCA staff are capable of operating the equipment to remove and install stoplogs. However, not all stoplogs were removed during the test, nor was it confirmed that the flow control equipment is adequate to remove all stoplogs under high flow conditions.

Witnessing the operation of the lock gates and lock fill valves provided sufficient evidence to confirm that PCA staff are capable of operating the equipment and that the equipment is adequate. The only concern noted by the Operators is that the actuators require a high level of force to get the gates moving from a stopped position. It was observed that Operators must repeatedly walk close to the edge of the lock deck while operating the gate actuators posing a potential fall hazard into the lock.

The mechanical equipment at the dam and lock are generally in good condition but some remedial work is required and a documented maintenance program needs to be implemented. A summary list of recommended remedial actions is provided in Section 9.0.

4.4 GEOTECHNICAL ASPECTS – EARTH STRUCTURES AND FOUNDATIONS

In this report, “right” and “left” is referenced to looking downstream.

The geotechnical components of the site inspection for Lock 39, Talbot River Dam and Talbot Canal site included earthfill structures, dam and lock foundations, headpond and tailrace shorelines and abutments. There are four main earthfill structures associated with the site. These include the Right (North) and Left (South) Earthfill Embankments located at Talbot River Dam and the Northwest and Southeast Canal Earthfill Embankments located on either side of the Talbot Canal (refer to Figure 1.2-1 shown in Section 1.2). The earthfill associated with Lock 39 is primarily associated with the backfill of the lock retaining walls that blend into the abutments, and there are no significant earthfill structures associated with water retaining components.

An evaluation and assessment of the geotechnical performance relative to slope stability, seepage, concrete-to-bedrock interfaces, retaining wall loading / support, and settlement / deformation has been completed based on the following:

- review of available and relevant background information;
- detailed visual site inspection of the dam structures, foundations, and abutments; and
- KGS Group 2013 geotechnical investigation program (see Appendix A).

4.4.1 Description of structures

4.4.1.1 Right (North) Earthfill Embankments

The Right Earthfill Embankment was constructed of an earthfill section with a concrete core wall and an additional 14 m long end section with sheet pile core near the right (north) abutment at the river shoreline. The plan and typical cross-sections of the embankment are shown on Drawings G01 and G02, respectively and are provided in Appendix B. Key details of the Right (North) Wing Earthfill Embankment are based on the as built drawing (DWG No: T22-118a66, Simcoe Balsam Lake Division), background information and topographic survey from November 27 and 28, 2012 are summarized below:

- The concrete core wall is up to 6.2 m high and the width of the concrete core wall is approximately 0.9 m at the top and 1.5 m at the base. The wall was apparently founded on the bedrock based on background information (DWG No: T22-118a66, Simcoe Balsam Lake Division), although there are no details confirming the base condition of the wall and foundation preparation. The background drawings also show an approximately 0.9 m thick clay puddle placed at the base of the upstream side against the core wall that presumably acts to minimize the seepage through the foundation contact of the core wall. The top of the concrete core wall is at El. 230.85 m based on the background information, which is consistent with the elevation measured from the topographical survey completed at the exposed core wall area near the north abutment.
- The crest elevation of the embankment was surveyed between El. 231.10 m and EL.231.35 m and the crest width is approximately 3.5 m.
- The embankment is approximately 130 m long and up to 4.5 m high at the downstream side above the surrounding ground surface.
- The upstream and downstream slopes of the embankment are at approximately 2.5H: 1V and 1.5H: 1V respectively based on the topographic survey. The upstream slope below the water is unknown, however the as built drawing shows the upstream slope is at approximately 2H: 1V.
- Riprap provided for erosion protection at the upstream side.
- The north abutment of the embankment blended well into the existing ground surface at the shoreline, while the concrete core wall is tied into the Talbot River Dam structure at the south abutment.

4.4.1.2 Left (South) Earthfill Embankment

The Left (South) Earthfill Embankment extends into the Northwest Canal Earthfill embankment of the Talbot Canal and consists of an earthfill section with a 12 m long concrete core wall extending south from the concrete dam structure. The plan and typical cross-sections of the embankment are shown on Drawings G01 and G02, respectively (see Appendix B). Key details of the Left (South) Wing Earthfill Embankment based on the as built drawing (DWG No: T22-118a66, Simcoe Balsam Lake Division), background information and topographic survey from November 27 and 28, 2012 are summarized below:

- The concrete core wall is up to 6.2 m high and the width of the concrete core wall is approximately 0.9 m at the top and 1.5 m at the base of the foundation. Consistent with the Right Embankment, the wall was apparently founded on the bedrock and a 0.9 m thick clay puddle was placed at the upstream side of the core wall. The top of the concrete core wall is at El. 230.85 m based on the background information.
- The crest elevation of the embankment was surveyed between El. 231.15 m and EL.231.30 m and the crest width is approximately 4.0 m near the Talbot River Dam structure. The embankment has wider crest width (up to 10 m) away from the dam structure.
- The embankment is approximately 12 m long and is low in height at the downstream side (1.8 to 2.2 m) above the surrounding ground surface.
- The upstream and downstream slopes of the embankment are at approximately 2.5H: 1V and 1.5H: 1V respectively based on the topographic survey. The upstream slope below the water is unknown; however, the as built drawing shows the upstream slope is at approximately 2H: 1V.
- Riprap provided at the upstream side for erosion protection.
- The south end of the embankment (abutment) blended well into the Northwest Canal Earthfill embankment while the concrete core wall is tied into the Talbot River Dam structure at the north end.

4.4.1.3 Northwest and Southeast Canal Earthfill Embankments

The Northwest and Southeast Earthfill Embankments of the canal were constructed of an earthfill section with a single gabion basket installed near the mid portion of the upstream (canal side) slopes. The plan and typical cross-sections of the embankment are shown in Drawings G03 and G04 to G06, respectively (see Appendix B). Key details of the canal embankment based on the background information and topographic survey are summarized as follows:

- A single gabion basket (900 X 900 mm) was installed along the canal near the mid portion of the upstream (canal side) slope.
- Granular fill (19 mm crushed stone) was placed behind and likely at the base of the gabion baskets.
- No geotextile separator was observed between the gabion basket and the backfill.

- Riprap erosion protection was provided on the lower slope area below the water up to the gabion basket.
- The crest elevation of the embankment was surveyed at El. 231.15 m to EL.231.35 m and the crest width is approximately 3.0 m.
- The embankment is approximately 1300 m long, with typical slope heights of 1.5 to 3.2 m and up to 4.2 m high at the downstream side (approximately between STA. 0+580 to STA. 0+640- near the concrete structure approximately at the middle of the canal) above the surrounding ground surface.
- The upstream and downstream slopes of the embankment are at approximately 1.75 to 3.0H: 1V and 1.75 to 2.5 H: 1V respectively.

The original PWGSC Terms of Reference for this DSR noted that a steel sheet pile wall was installed along the crest of the embankments towards the upstream edge of both canal embankments. However, no background documentation or drawings were available to substantiate the existence of the steel sheet pile wall nor could any PCA personnel confirm that sheet piles were installed in this location. As part of the 2013 geotechnical investigation, test pitting was performed by KGS Group at four select locations along the canal embankments to try and identify whether or not the sheet piles were installed. Note that similar test pitting was completed at the canal embankment just upstream of Lock 40 where seepage and erosion was observed in the 2013 summer. At this location near Lock 40, the sheet piles were found near the upstream edge of the crest. The water level was high during the test pitting between lock 39 and the Talbot River Dam (approximately 0.8 m below the crest), and the test pitting depth and extent were limited to above the water level to ensure that there would be no seepage or erosion, particularly since the location and elevation of the sheet piles were not yet known. Sheet piles were not encountered within the investigated depth and extent of the test pits.

4.4.1.4 Earthfill Backfill of Lock 39 Walls

There are two earthfill structures associated with the Lock 39 structure, including the backfill on both sides of the concrete lock walls that extend downstream to the canal. The earthfill associated with Lock 39 is primarily associated with the backfill of the lock retaining walls. There are no significant earthfill structures associated with water retaining components. The plan and typical cross-sections of the embankment are shown in Drawings G03 and G05, respectively

(see Appendix B). It should be noted that the earthfill embankment has a wider crest width (12 to 15 m).

4.4.2 Site Inspection

The inspection followed the terms of reference and criteria specified within the Directive for Dam Safety Program of Parks Canada Dams and Water-Retaining Structures. During the geotechnical inspection, the following tasks were performed in support of the evaluation of the existing physical conditions of the structures:

- Inspection of the earthfill embankments.
- Inspection of visible bedrock foundations.
- Inspection of the slopes of shorelines, with consideration of stability, seepage and erosion protection.
- Inspection of the abutments.

Some of the key observations made during the site inspection are provided below. The inspection checklists containing the geotechnical observations provide more detailed information and are included in the Dam Safety General Inspection (DSGI) forms in Appendix C. Some key observations on the geotechnical components from the site inspection are summarized in the following sections.

At the time of the inspection, there was some minor snow cover on the embankments which possibly could have obscured some deficiencies. However, subsequent to the site inspection, our geotechnical engineer was on site in June 2013 for the geotechnical drilling program and the observations noted below during the DSR site inspection are consistent with what was observed in June 2013.

4.4.2.1 Right (North) Earthfill Embankment

Based on visual inspection, the Right Earthfill Embankment was generally in fair condition geotechnically. There was evidence of previous movement, although there did not appear to be any recent significant slope movements, erosion or seepage that would suggest significant

concerns related to the slope stability and safety of the structures. Some key observations from the site inspection included the following:

- Some uneven areas were observed at the crest but there was no evidence of surface cracking, significant settlement or depression.
- The concrete core wall was exposed on the upstream side due to erosion and/ or local shallow sloughing of the upper portion of the slope. The condition of the slope below water is unknown. The exposed length of core wall was approximately 6 m long and located approximately 115 to 121 m from the North Abutment of the Spillway (Photo 4.4.2-1).
- Some minor erosion was observed at the upstream edge of the crest throughout the dam, causing misalignment of the crest edge. More significant erosion was observed at the exposed area of the core wall as noted above. No visual misalignment was observed on the downstream side.
- No signs of any active or historic slope failures or movements were observed at the upstream and downstream slopes with the exception of the area of the exposed concrete core wall and minor erosion at the edge of the crest as noted above at the upstream slope.
- Primarily finer to larger size riprap (typically 75 mm to 450 mm) was observed at the upstream side. Some erosion was observed above the riprap, and some silt infill was also observed within the riprap. The upper slope above the riprap appeared steeper due to loss of material from erosion.
- The abutment contacts between the concrete and earthfill at the Talbot River Dam as well as the earthfill and shoreline at the north end were in good condition. There was no evidence of separation or seepage.
- Some minor water flow and stagnant water was observed within the ditch along the toe of the dam (Photo 4.4.2-2). The source of the water flow was not apparent, however it may have been associated with snow melting at the time of inspection. There was no visual evidence of seepage through the dam or piping / internal erosion, although dense vegetation and snow partially obscured inspection.
- Minor surficial erosion and small gullying was observed approximately 10 m north of the Talbot River Dam at the downstream side (approx. 0.45 m wide by 0.2 m deep (Photo 4.4.2-3).

- Grass and low shrub vegetation covered the downstream slope and crest providing protection against surficial erosion. There was some tree growth at/ near the crest of the upstream and the downstream sides.

PHOTO 4.4.2-1
UPSTREAM SLOPE OF RIGHT (NORTH) WING EARTHFILL EMBANKMENT
(Note significant loss of upper portion of the slope and exposed concrete core wall.)

PHOTO 4.4.2-2
DOWNSTREAM DITCH AT TOE OF RIGHT (NORTH) WING EARTHFILL EMBANKMENT
(Note minor water flow and stagnant water observed in base of ditch.)

PHOTO 4.4.2-3
DOWNSTREAM SLOPE OF RIGHT (NORTH) WING EARTHFILL EMBANKMENT
(Note minor erosion and small gullying.)

4.4.2.2 Left (south) Earthfill Embankment

Based on the visual inspection, the Left Earthfill Embankment was generally in good condition geotechnically. There were no slope movements, significant erosion or seepage that would suggest significant concerns related to the slope stability and safety of the structures. Some key observations from the site inspection include the following:

- Some uneven areas were observed at the crest but there was no evidence of surface cracking, significant settlement or depression.
- Some minor erosion was observed at the upstream edge of the crest above the riprap throughout the dam, causing misalignment of the crest edge. No visual misalignment was observed at downstream side.
- No signs of any active or historic slope failures or movements were observed at the upstream and downstream slopes.
- Primarily finer to larger size riprap (typically 75 mm to 450 mm) was observed at the upstream side. Some erosion was observed above the riprap, and some silt infill was also observed within the riprap.
- The abutment contacts between the concrete and earthfill at the Talbot River Dam as well as the earthfill and the Northwest Canal Earthfill embankment were in good condition. There was no evidence of separation or seepage.
- Grass and low shrub vegetation covered the downstream slope and crest, providing surficial erosion protection. There was some tree growth at/ near the crest of the upstream and the downstream sides.

4.4.2.3 Northwest and Southeast Canal Earthfill Embankments

Based on the visual inspection, the Northwest and Southeast Earthfill Embankments were generally in fair to poor condition geotechnically, in recognition of the significant erosion and notable loss of the upper portion of the upstream slope. Rotated, damaged and deformed gabion baskets were also observed in both embankments. However, there were no signs of deep seated slope failures, significant slope movements, nor seepage that would suggest immediate concerns related to the slope stability and safety of the structures. Some key observations from the site inspection included the following:

- Some uneven areas were observed at the crest but there was no evidence of surface cracking, significant settlement or depression. Some depressions and a few minor sinkholes were observed at the southeast earthfill embankment between STA 0+ 050 and STA 0+450 (Drawings G03, Appendix B). The depressions were local in extent, and did not appear to be associated with significant dam settlement or slope movement (Photo 4.4.2-4).

PHOTO 4.4.2-4
CREST OF SOUTHEAST CANAL EARTHFILL EMBANKMENT
(Note sink hole near STA 0+ 250.)

- A few disturbances in the Southeast Canal Earthfill embankment were observed between STA 0+050 to 0+250. These disturbances appear to have been man-made, with a trench along the crest of the dyke and the excavated material piled near the toe of the slope. It is speculated that the disturbances observed were made for farm-land irrigation purposes, although this has not been confirmed (Photo 4.4.2-5).

PHOTO 4.4.2-5
CREST OF SOUTHEAST CANAL EARTHFILL EMBANKMENT
(Note apparent man-made cut across embankment near STA 0+250.)

- It was generally observed that the life expectancy of the gabion baskets has expired. The majority of the gabion baskets were corroded causing exposure of the inner rockfill. However most of the baskets generally retained their shape without significant collapse of the rockfill within the gabions.
- The gabion baskets were frequently observed to be rotated and damaged. Some longitudinal misalignment was also noted in some areas.
- Significant erosion and loss of the upper portion of the upstream slope behind the gabion baskets were observed at the southeast earthfill embankment between STA 0+375 and STA 0+425 causing misalignment of the crest edge. No visual misalignment was observed at the downstream side.
- Some loss of material (19 mm clear crushed stone) was frequently observed behind the gabions throughout both embankments (Photo 4.4.2-6).

PHOTO 4.4.2-6
SOUTHEAST CANAL EARTHFILL EMBANKMENT LOOKING DOWNSTREAM FROM STA 0+200
(Note loss of material behind the gabions, typical gabion rotation, corrosion damage and misalignment of baskets.)

- Significant loss of the upper portion of the slope behind the gabions, approximately 10 m in length, was observed at the Northwest Canal Earthfill embankment near STA 0+150 and was associated with erosion (Photo 4.4.2-7).

PHOTO 4.4.2-7
UPSTREAM SLOPE OF NORTHWEST CANAL EARTHFILL EMBANKMENT
(Note significant erosion of slope.)

- Similar loss of the upper portion of the slope behind the gabions was also observed at the southeast earthfill embankment between STA 0+375 and STA 0+425 and between STA 0+750 and STA 0+800 (approximate). This loss of material also appeared to be associated with erosion.
- No signs of any active or historic deep seated slope failures or movements were observed at the upstream and downstream slopes.
- The abutment contacts between the concrete structure and earthfill embankment near the mid-length of the canal (STA 0+600) were generally in good condition and there was no evidence of separation or seepage in the earthfill. However, gabion basket misalignment and rotation at the abutments to the concrete structure were observed and some erosion around the ends of the concrete structure was observed at both embankments (Photos 4.4.2-8 and 4.4.2-9).

PHOTO 4.4.2-8
UPSTREAM SLOPE OF NORTHWEST CANAL EARTHFILL EMBANKMENT AT ABUTMENT TO CONCRETE STRUCTURE (WEST END) NEAR STA 0+ 600

PHOTO 4.4.2-9
UPSTREAM SLOPE OF SOUTHEAST CANAL EARTHFILL EMBANKMENT AT ABUTMENT TO CONCRETE STRUCTURE (WEST END) NEAR STA 0+ 600

- Primarily grass and low shrub vegetation covered the downstream slope and crest, providing surficial erosion protection. In some areas some tree growth was observed at upstream slope and within the gabion baskets.

4.4.2.4 Foundations

The background information suggested that Talbot River Dam (and particularly the concrete core wall) was founded on competent bedrock, although the bedrock/ concrete contact interface was not visible. There were no signs of distress of the structure that would suggest any concerns related to the foundation support.

The background information revealed that the Lock 39 structure was founded on end-bearing timber piles founded on the underlying bedrock or competent soil. There were no signs of any significant settlement or distress that would suggest any concerns related to the foundation conditions based on the visual inspection of the Lock 39 structure.

4.4.2.5 Headpond and Tailrace Shorelines

Based on visual inspection, the headpond and tailrace shorelines were in good condition. There were no noticeable historic or active slope failures/ movements or significant erosion that would suggest significant concerns related to the slope stability and safety of the structures. Generally, riprap provided erosion protection along shorelines. There was some minor erosion observed above the riprap at the headpond shorelines upstream from the dam but it did not appear to be a threat to the stability of the slopes.

4.4.2.6 Dam Abutments at the Talbot River Dam

Based on the visual inspection, the dam abutments at the concrete spillway of Talbot River Dam were in good condition geotechnically. There was no evidence of slope movement, erosion or seepage that would suggest significant concerns related to the slope stability and safety of the structures. Some key observations from the site inspection included the following:

- The abutments are tied into the concrete core wall of the earthfill embankment.
- Primarily finer size (75 to 250 mm with gravel) riprap was visible at the surface of the upstream side of the north abutment (Photo 4.4.2-10). No evidence of significant erosion within the riprap was observed but some erosion was observed above the riprap.
- A stacked stone wall was constructed along the face of the downstream slope at both abutments. The stone wall appeared intact at the abutments, although the shape of the stones indicate possible movement / settlement in the past, particularly in the lower half of the wall near the concrete structure at north abutment (Photo 4.4.2-11). The movement did not appear recent.
- There was some tree growth on the upstream slope and the downstream slope within the stacked stone.

PHOTO 4.4.2-10
NORTH ABUTMENT OF THE CONCRETE STRUCTURE AT THE DAM (UPSTREAM SIDE)

PHOTO 4.4.2-11
NORTH ABUTMENT OF THE CONCRETE STRUCTURE AT DAM (DOWNSTREAM SIDE)
(Note some tree growth within the stacked stone wall.)

4.4.2.7 Lock 39

There are two earthfill structures associated with the Lock 39 structure, including the backfill on both sides of the concrete lock walls that extend downstream to the canal. There is little to no water retained by these earthfill embankments extending to the sides of the lock structure. There are also earthfill slopes transitioning from the lock area downstream to the canal dykes that provide access beyond the limit of the lock structure, although these are not water retaining components of the structure. Overall, the earthfill embankments were generally in good condition geotechnically, based on the visual inspection. There were no slope movements, significant erosion or seepage that would suggest significant concerns related to the slope stability and safety of the structures. It should be noted that the earthfill embankment has a wider crest width (12 to 15 m). Any shallow sloughing and shallow slope movement at the downstream slope will not immediately impact the dam performance due to this wider crest width.

4.4.3 Foundation Material Properties

The key geotechnical design parameters for the site are the frictional contact at the concrete to bedrock interface, as well as the shear strength (i.e. internal angle of friction, Ø') of the natural and earthfill materials within the earthfill embankments. These properties are discussed in more detail in the following sections.

4.4.3.1 Bedrock to Concrete Shear Strength

Background Information for Talbot River Dam

Review of the available background information revealed that there were no field investigations or laboratory testing to determine the shear strength parameters at the concrete–to-bedrock interface. During the site inspection, the bedrock–to-concrete contact was not visible at the dam. The cross sections of the concrete dam on the available construction drawing (DWG No: T22-118a 66, Simcoe Balsam Lake Division) showed that the foundation surface was uneven and that the structures appeared to be keyed into the bedrock. This would imply that there was some excavation and preparation of the bedrock surface as part of the construction. This would likely have consisted of removal of all overburden and loose rock material, as well as possible

blasting, which is consistent with typical construction methods. However, there were no construction photographs or documents available for review under this DSR to verify the preparations performed on the foundations.

Review of Contact Shear Strength

Some general comments regarding the shear strength parameters of the Talbot River Dam include the following:

- There was no background documentation available that showed the foundation preparation performed during construction (e.g. construction reports, photographs, etc.). Therefore, the actual conditions at the base of the concrete structures could not be verified. It is usually good construction practice that preparation of the foundation includes removal and cleaning of all overburden and loose rock material, as well as possible blasting along portions of the dam base. This would result in a reasonably rough surface with good frictional contact. However, this could not be confirmed for this review.
- Caution must be exercised when considering the use of cohesion in the contact shear strength as there may be weak zones or discontinuities at or near the contact resulting in an unbonded or partially bonded interface. In addition, CDA 2007 and OMNR 2011 dam safety guidelines recommend that cohesion should not be included within the analyses unless there is adequate test information or other justifications to warrant its use.
- There are a number of factors that affect the contact shear strength (e.g. actual roughness of the contact surface, presence of joints within the bedrock below the contact, infill along the contact that could reduce the friction, etc.). Further discussion on the potential range in contact shear strength that could be representative for the Talbot River Dam structure is provided below.

Review of published information on bedrock to concrete contact conditions was performed as part of the contact shear strength assessment, including generally accepted methods to estimate representative shear strength along the contact typically used as part of dam analyses (Ref. 31 to 34). Some general considerations include:

- The long term reliability of any concrete to bedrock bond is difficult to predict as it can be impacted over time by environmental effects such as seepage, thermal stresses, chemical

weathering, etc. Both CDA and OMNR guidelines recognize these difficulties, and recommend that stability evaluations be performed without reliance on bonding or cohesion as a first step. If stability criteria are not satisfied, then the use of cohesion could be considered, although it must be supported by relevant information such as field investigation and testing. Furthermore, OMNR only permits the use of field tested cohesion if the results of the stability analysis demonstrate that the structure has a safety factor in excess of 1.0 in the absence of a cohesive bond.

- A small amount of cohesion can significantly affect the available shear resistance, particularly for low to medium height dams, which is the case at the Talbot River Dam. For partially bonded surfaces, the location of the bonding can also impact the stability calculations (i.e. higher safety factors are calculated when the bonded portion is towards the upstream portion of the dam). In addition, there has been significant testing performed on bedrock to concrete interfaces, particularly for dam sites owned and operated by Ontario Power Generation (OPG) with similar foundation and construction conditions to the Talbot River Dam. The data from this testing has shown considerable variations in shear strength (both friction and tension/cohesion). This variation can be partly attributed to difficulties in completing small-scale testing used to represent field conditions, but also represents likely variations between the different sites and across a dam base.

The selection of the shear strength parameters requires care, and conservative values are normally selected given the difficulty in accurately determining a shear strength that would be representative across the entire contact surface. Typical methods for estimating the shear strength along the contact (see Refs. 31 to 34) consist of determination of a representative basic friction angle (approaching the residual frictional contact of the bedrock to concrete materials), with inclusion of a measure of the roughness of the contact to reflect increased resistance to sliding. The selection of the roughness coefficient to represent the field conditions is somewhat subjective, and again can be variable across the actual contact surface.

Possible Range of Contact Strength for Talbot River Dam

The basic friction angle of limestone bedrock to concrete similar to that found at Talbot River Dam site typically varies between 27° and 37° (based on published information and previous testing). The factor to represent the increased resistance to sliding associated with the contact

roughness is dependent on the roughness of the surface, the applicable normal stresses, and the compressive strength of the bedrock. Methods to estimate the various factors are provided in referenced literature, and care must be taken when considering the large-scale effects on joint roughness. If the bedrock surface is relatively smooth, it would not provide a high level of interlocking and roughness. In addition, possible jointing within the bedrock below the concrete must be factored into consideration of the potential sliding block failure. There is no construction documentation, field investigations, or detailed bedrock joint mapping to confirm the actual conditions of the foundation contact. Another general comment of note is that tests on small scale samples have been found to overestimate the actual field strengths, although this has not been conclusive. This is important when considering the use of published information for the contract shear strength which is mostly based on small-scale core samples.

Assuming a relatively smooth to moderately rough surface (Joint Roughness Coefficient of 5 to 10, a representative in-situ effective friction angle (Ø') could range from the low 30°'s to possibly as high as 45°, depending on the assumed basic friction angle (estimated at 27 to 37° for the Talbot River Dam Structure).

As a result of these uncertainties and in the absence of construction documentation, reasonably achieved lower bound shear strength of friction angle of 36° with zero cohesion was used for the stability analysis of the concrete structures. It is possible that a higher friction angle and some cohesion could be applicable; however, it cannot be confirmed with the available information.

4.4.3.2 Earthfill Embankment Material Shear Strength

A review of the existing documentation has shown that there was limited to no information on the earthfill materials used as backfill for the embankments and concrete walls. As a result, a geotechnical investigation program was undertaken to identify the conditions for incorporation into the DSR. Note that the investigation program was not considered extensive and investigation activities that could potentially be required for design of any remedial work were not included in this program. The 2013 site field investigation report is annexed in Appendix A.
The 2013 site investigation program was performed to obtain the geotechnical data required to determine the general embankment material types and their engineering properties,

embankment foundation conditions and to install monitoring wells and vibrating wire piezometers to measure the piezometric levels within the embankments.

Representative engineering properties of the fill and foundation soils for the earthfill embankments are presented on Table 4.4-1. The shear strength parameters of the embankment fill were estimated from in-situ Standard Penetration Tests (SPT), laboratory index testing using empirical correlations and the interpretation of the advanced laboratory testing results, as well as previous experience with similar soil materials. Ranges of representative upper and lower bounds of these estimated shear strengths have been provided to represent the variations observed from the investigations.

TABLE 4.4-1
EFFECTIVE SHEAR STRENGTH PARAMETERS FOR EARTHFILL EMBANKMENTS

Material	Representative Lower and Upper Bounds, Φ'	Consistency based on SPT	Cohesion, c' (kPa)	Estimated Unit Weight, γ_{sat} (kN/m^3)
North and South Embankments of Talbot River Dam				
Sandy Silt to Silty Sand Fill	30° to 32°	Loose to Compact	0	19.5
Clayey Silt to Silty Clay Fill	27° to 30°	Soft to Firm	0	18.0
Sandy Silt Till	38° to 40°	Dense to Very Dense	0	21.0
North of Lock 39 Embankments				
Clayey Silt to Silty Clay Fill	27° to 30°	Soft to Very Soft	0	18.0
Sandy Silt Till	38° to 40°	Compact	0	21.0
Northwest and Southeast Canal Embankments of Talbot Canal				
Clayey Silt to Silty Clay Fill	27° to 30°	Soft to Very Soft	0	18.0
Clayey Silt to Silty Clay	27° to 30°	Soft to Very Soft	0	18.0
Sandy Silt Till	38° to 40°	Very Dense	0	21.0

4.4.4 SURVEILLANCE AND MONITORING

The surveillance and monitoring of the dam performance to date has primarily been through visual inspections to determine whether there are any significant changes to the dam conditions

that could threaten its operation or integrity. However, there are no records of regular inspections documenting the observed conditions. The site inspections have not identified any significant concerns related to the geotechnical performance of the structures with the exception of the concrete core wall exposed at the Right Wing Earthfill embankments at the Talbot River Dam, and the poor condition of the gabions, man-made disturbances and significant erosion on the canal embankments. However, a more formal inspection and documentation process is preferred so that the observed conditions can be better documented, and potential changes to the structure performance could be better identified.

There was no geotechnical instrumentation monitoring the ongoing dam performance prior to the initiation of this DSR. As part of this study and corresponding investigation program, three monitoring wells (standpipe piezometers) and two vibrating wire piezometers were installed between June 6 and 8, 2013 at the earthfill embankments to obtain groundwater measurements. The vibrating wire piezometers were connected to an automated data logger to continuously monitor the piezometric levels within the embankment fill while the standpipes require manual readings of water level. The location of the piezometers is provided in the Drawings G01 and G03 (see Appendix B), and summarized below.

- Vibrating wire piezometer at the Right Wing Earthfill Embankment at Talbot River Dam within the sandy silt till.
- Vibrating wire piezometer at the North of Lock 39 within the embankment fill north of Lock 39-VW.
- Standpipe piezometer at the Right Wing Earthfill Embankment at Talbot River Dam within the bedrock.
- Standpipe piezometer at the north side of Lock 39 within the foundation soil.
- Standpipe piezometer at the Northwest Canal Embankment at Talbot Canal within the foundation soil.

4.4.5 GROUNDWATER CONDITIONS

After installation, standpipe readings were monitored two times in June 2013 and September 2013. The monitoring zone and the results of the groundwater level monitoring are summarized

on Table 4.4-2. The standpipe readings indicated little change in the groundwater levels within the foundation soils was less than 0.6 m during the period from June 2013 to September 2013.

From June 6, 2013 to September 12, 2013 the vibrating wire piezometers were set up to collect data every 5 minutes in order to monitor groundwater levels within the embankment fill or foundation soils. The monitoring data from the installed VW piezometers are summarized in Figures 4.4.5-1 and 4.4.5-2 together with the upstream reservoir levels and Talbot Canal levels. VW monitoring data from north of Lock 39 was reviewed to determine any groundwater fluctuation during the lock operation. Some lock operation times between June and September 2013 were gathered from the PCA site log book, although the water levels are not recorded with time. The typical lock operation consists of the lowering and refilling of the approximate 6.0 m water level over a 20 to 30 minute period. The review of piezometer data revealed that the groundwater levels during the lock operations have shown only approximately 25 to 50 mm groundwater fluctuation, indicating that there is a very slight to no impact on the piezometric levels within the embankment during the lock operation. As shown in Figures 4.4.5-1 and 4.4.5-2, the piezometric levels measured in the Right Wing Earthfill Embankment of Talbot River Dam varied between El. 227.9 m to El. 228.2 m during the monitoring period and between El. 227.8 m to El. 228.4 m at the embankment north of Lock 39.

TABLE 4.4-2
GROUNDWATER MONITORING DATA – STANDPIPES READINGS

Date	Groundwater Elevation (m)		
	Talbot River Dam	Lock 39	Talbot Canal
June 15, 2013	227.36	227.53	229.47
September 12, 2013	227.96	227.12	-
September 13, 2013			229.34

FIGURE 4.4.5.-1
GROUNDWATER LEVEL MEASUREMENTS - NORTH EMBANKMENT OF TALBOT RIVER DAM

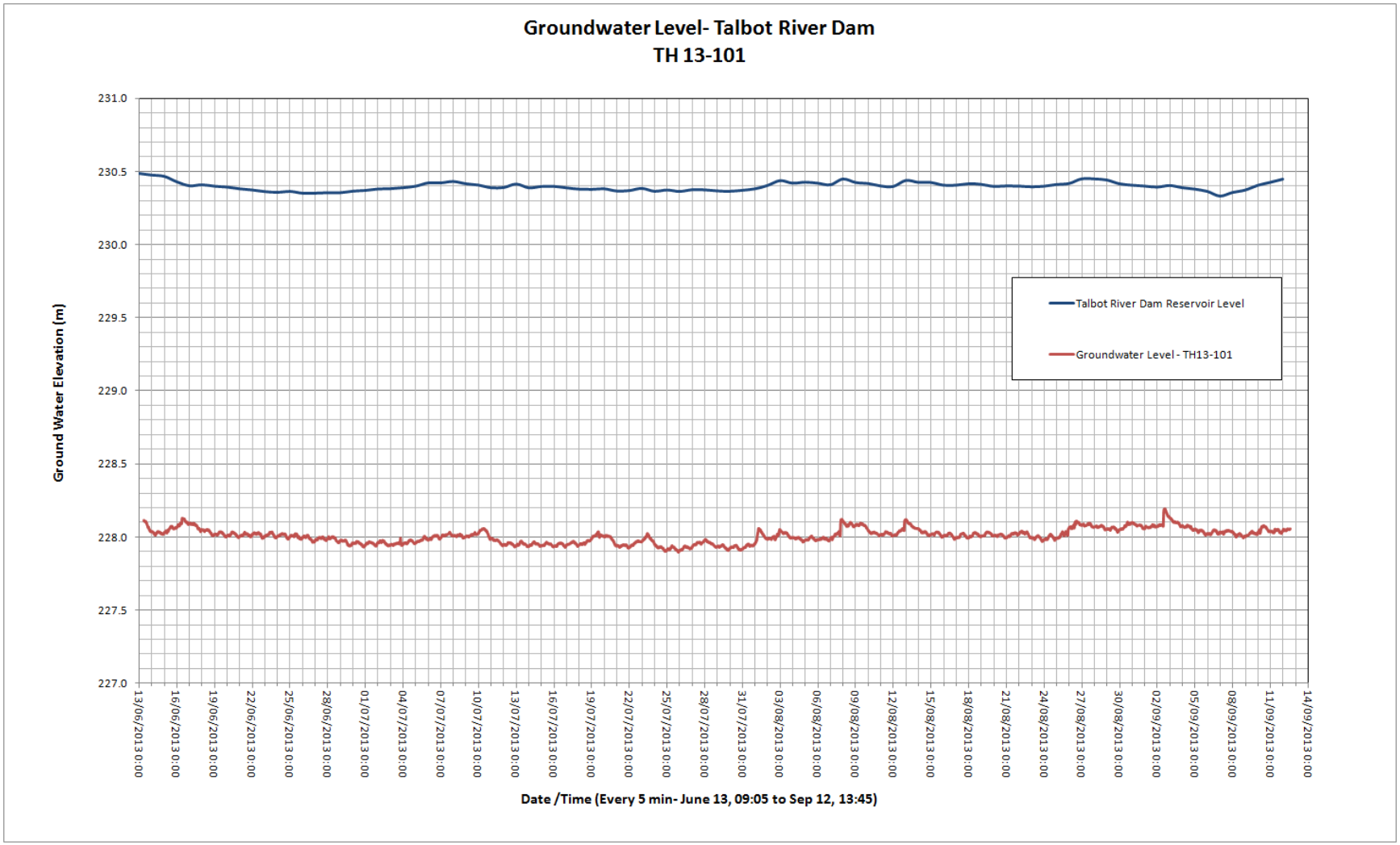
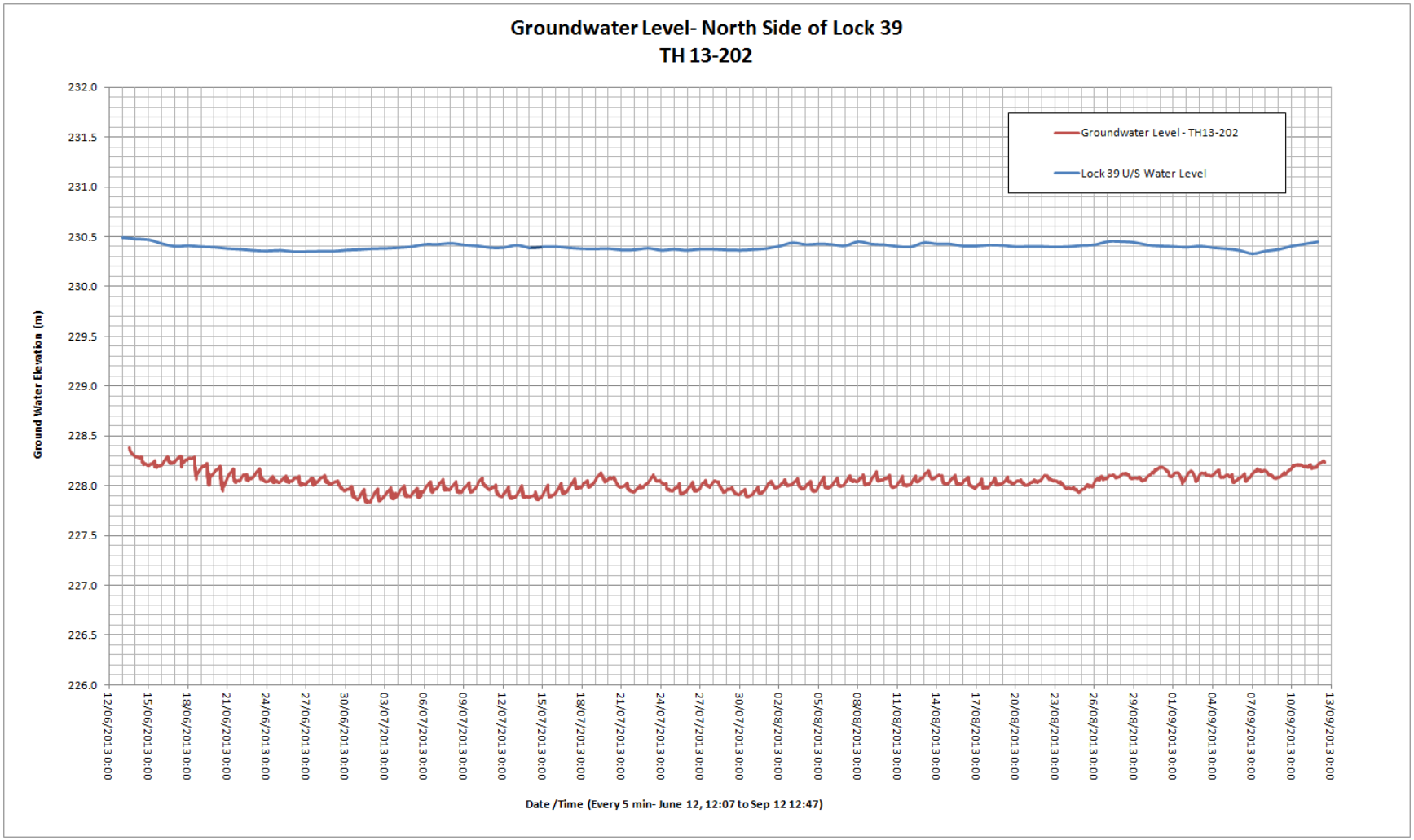


FIGURE 4.4.5-2
GROUNDWATER LEVEL MEASUREMENTS - NORTH EMBANKMENT OF LOCK 39.
(Note: That Water Level Fluctuation with Lock Operation Not Recorded By PWGSC and Not Shown)



4.4.6 SEEPAGE AND DRAINAGE

In the Right and Left Wing Earthfill Embankments at Talbot River Dam, the concrete core walls function as a barrier to control seepage. Based on the site inspection and the available background information, there are no formal measures such as a foundation drainage system (filter) to collect seepage. However, a ditch was located at the toe of the slope which also collects the surface runoff water from the nearby road. Some minor water flow and stagnant water was observed within the ditch along the toe of the dam. The source of the water flow was not apparent, although possible sources may be either seepage through the dam or snow melting at the time of inspection. There was no visual evidence of seepage through the dam or piping / internal erosion; however, dense vegetation and snow partially obscured inspection.

In the Lock 39 Earthfill Embankments, the lock concrete walls function as a barrier to retain the water in the lock chamber and control seepage. The Lock 39 structure background drawings show a granular zone immediately behind the lock walls which would act as seepage collection and control, although drainage and discharge details are not apparent on the drawings. The embankment has generally functioned adequately. There was no evidence of active seepage observed on the slopes and the toe areas of the embankments to the sides of the lock walls during the site inspection, although these embankments are relatively low in height (typically less than 3 m).

In the canal earthfill embankments, it appeared that clayey silt to silty clay type embankment fill and foundations soils control the seepage through the embankment. Based on the site inspection and the available background information, there are no formal measures such as a foundation drainage system (filter) to collect seepage. There was no evidence of active seepage observed on the downstream slope and the toe areas of the embankments during the site inspection.

A ditch was located at the toe of the slope which also collects the surface runoff water from the nearby road. Minor water flow and stagnant water was observed within the ditch along the toe of the dam which could be associated with snow melting at the time of inspection or seepage through the embankments. There was no visual evidence of seepage through the dam or piping / internal erosion, although dense vegetation and snow partially obscured inspection.

In general, control of the surface drainage over the earthfill embankments is good. Grass and vegetation cover provides erosion protection for the crest and the downstream slope of the earthfill embankments. The upstream slope of the Right and Left Earthfill embankments were protected from erosion by rockfill riprap. Primarily finer to larger size riprap (typically 75 mm to 450 mm) was observed at the upstream side of the both embankments and the upstream side of the north abutment (Talbot River Dam). Some erosion was observed above the riprap. Some silt infill was also observed within the riprap. These observations are not major deficiencies and do not pose a threat to the performance of the dam in the short term. However, the condition of the crests requires monitoring, particularly following strong wind events to ensure there are no threats to the dam with continued erosion.

The upstream slope of the canal embankments had rockfill riprap and the gabion basket to provide erosion protection. It was generally observed that the life expectancy of the gabion baskets has expired. The majority of the gabion baskets were corroded, misaligned and rotated, although most of the baskets generally retained their shape without significant collapse of the rockfill within the gabions. Significant erosion and loss of the upper portion of the upstream slope were also observed in the canal embankments. Again, these conditions did not appear to pose a threat to the performance of the canal embankments in short term, however ongoing monitoring is required before any remedial works are undertaken to ensure there are no threats to the integrity of the canal slopes.

4.4.7 Foundation Support and Settlement

All the embankment structures were constructed approximately 100 years ago and all primary settlement within the embankment and the foundation soils will have already occurred. The Talbot River Dam embankments are founded on compact to very dense till underlain by bedrock and the foundation soils are not likely to settle. Talbot Canal embankments are founded on firm to stiff clayey silt to silty clay underlain by dense till. Foundation soils are not likely to settle under existing conditions. There were also no signs of any significant settlement observed during the field inspection.

The background information revealed that the Lock 39 structure was founded on end-bearing timber piles founded on the underlying bedrock or compact to dense sandy silt till. There were

no signs of any significant settlement or distress that would suggest any concerns related to the foundation conditions of the lock walls based on the visual inspection.

Correctly treated timber piles can maintain their structural integrity almost indefinitely. Typical environments where degradation is a concern exist when the pile is exposed to alternate wetting and drying cycles or located above the water table. Groundwater monitoring at Lock 39 indicates that the timber piles are below water table. Based on the above facts, no degradation of the timber piles are anticipated at the site.

No significant future settlement that could impact the embankment support and lock walls is expected over the remaining life of the structures under existing conditions.

4.4.8 Liquefaction Potential

The potential for liquefaction during earthquake events depends on the soil type, the relative density or void ratio of the soil, the initial confining pressure and the intensity and duration of the ground motion. Soils most susceptible to liquefaction are uniformly graded (versus well graded) and loose fine sands and silts. These more susceptible soils are generally not found within the embankment materials or the foundation soils beneath the earthfill embankments, and the risk of liquefaction is considered low. The loose to compact sandy silt and silty sand layers observed below water at the downstream slope of the Right Earthfill Embankment at Talbot River Dam was an exception, and may susceptible to liquefaction. Based on the preliminary liquefaction analyses, with the design seismic events being of relatively low intensity (PGA value of 0.05g), the potential for liquefaction of the above mentioned loose to compact layers within the earthfill embankments is unlikely. Considering the Talbot River Dam embankments were constructed with a concrete core wall, minor movement of the loose layers due to the liquefaction is unlikely to impact the dam performance, particularly in short term.

The Right and Left Earthfill Embankments and Lock 39 embankments are founded on compact to very dense glacial till that consists mostly of well-graded sand, silt, clay, gravels, cobbles and boulders underlain by competent bedrock. The Northwest and Southeast Canal Earthfill Embankments are founded on clayey silt to silty clay. These foundation materials are not susceptible to weakening or liquefaction under seismic loading.

4.4.9 Vegetation

4.4.9.1 General Considerations

A review of the vegetation conditions at the lock and dam structures has been completed to evaluate the potential impacts that the existing growth could have on their performance, as well as the ability for effective monitoring. The evaluation methodology generally followed that identified in the FEMA 534 / September 2005 technical manual on plant impacts to dams (Ref. 37), in accordance with the requirements of the PWGSC scope of work / project brief.

As noted in the manual, it is generally accepted that woody tree growth on dam structures is not desirable and could lead to detrimental impacts on dam performance. It is also accepted that these impacts are dependent on the location of the trees on the embankments and structures as well as the type and extent of growth. In contrast to tree growth, dam engineers generally agree that a healthy, dense stand of low-growing grass is a desirable condition on earthfill dams to help protect embankment slopes against erosion.

Some of the more significant dam safety problems caused by tree and woody vegetation growth include the following:

- Fallen or uprooted trees could cause damage to the earthfill embankment and dam structures. This is particularly a concern with embankments having narrow or reduced sections / widths.
- Decaying roots may create seepage paths that could lead to internal erosion and piping of earthfills or foundation materials. Also, roots could wedge into joints or cracks in bedrock foundations or concrete structures, which could further open the joints and increase the seepage or piping potential.
- Tree and vegetation growth can cause interference with effective dam safety monitoring, inspection and maintenance, particularly where the cover is dense.
- Root growth can cause blockage of underdrain systems, resulting in potentially higher piezometric levels within the structures or foundations. This is less relevant for the embankments at these sites as there are no specific drainage systems with the exception of the Lock 39 walls.

- Cracking, uplifting or displacing concrete structures and other facilities can occur with root penetration.
- Tree growth can provide shelter for burrowing animals, which could lead to increased rodent population and activity.
- Root growth results in loosening of earth materials, which is of particular concern with tree uprooting and root decay over time.
- Tree growth can hinder establishment of more desirable vegetation cover such as grasses, which can ultimately lead to increased erosion.

General comments on the location of tree growth within an earthfill embankment and the relative level of concern are discussed below. Note that the typical concerns for each of the different zones are identified, however careful inspection and review of the tree growth over the entire dam is important to ensure all potential concerns are properly identified and appropriate remedial measures are implemented if warranted. Tree growth in the zones that are typically of lower concern can still lead to performance issues that could threaten dam performance or the required remedial works.

- ***Upstream Slope / Crest –*** The upstream slope and edge of the crest is one of the more critical zones relative to potential impacts from tree growth, particularly on embankments with narrow crest widths. A key dam safety concern associated with tree growth within this upstream zone is related to potential damage of the embankment with falling trees and disturbance of the fill materials, which could increase the potential for erosion.
- ***Crest / Upper Portion of Downstream Slope –*** Through the crest and upper approximately one third of the downstream slope, the potential impacts associated with woody tree growth are typically of lowest concern. However, this is the area that would typically show evidence of tension cracks or failure scarps associated with slope instability, and dense vegetation growth could impede inspection and identification of such problems.
- ***Downstream Lower Slope / Toe Area –*** This is typically the most critical area of the embankment relative to tree growth. The saturation and seepage lines associated with flow through the dam will be exiting the embankment at this lower portion of the downstream slope, and the presence of trees could result in a higher piezometric surface. The risk of this is lower with the concrete core walls at the Talbot River Dam. Any disturbance caused

by tree growth (soil loosening, voids from decaying roots or fallen trees, etc.) can significantly increase the potential for seepage and piping.

4.4.9.2 Site Conditions

The site inspection considered the type and location of tree growth relative to the guidelines provided FEMA manual (Ref. 37). Some tree and vegetation growth was observed on the upstream edge of the crest, the crest and the downstream slope and toe area of the earthfill embankments. No dead trees were observed obviously. However, there were a few trees that appeared distressed. Some specific observations of note:

- The trees on the Lock 39 Embankments crests were large and mature (typical trunk diameter of 0.2 to 0.45 m), and typically set back from the edge of the lock wall by 5 m or more. These trees were spaced fairly far apart and add to the park-like landscaping for the lock. They did not interfere with inspection.
- Smaller trees and brush growth on the Right and Left Embankments of the Talbot River Dam (typical trunk diameter up to 0.25 m) were observed at/ near the crest of the upstream and the downstream sides.
- The smaller tree growth on the abutments of the Right and Left Embankments of the Talbot River Dam (typical trunk diameter up to 0.2 m) was observed on the upstream slope, as well as the downstream slopes within the stacked stone at the abutments to the concrete Spillway.
- Some tree growth at / near the crest edge on both the upstream and downstream sides near the Talbot River Dam (Approx. STA 0+000 to 0+100).
- Smaller tree and brush growth on the Northwest Canal Embankment (typical trunk diameter up to 0.4 m) was observed at / near the crest edge on both the upstream and downstream sides and within the gabions, particularly near the Talbot River Dam (Approx. STA 0+000 to 0+100).
- The smaller tree and brush growth on the Southeast Canal Earthfill Embankment (typical trunk diameter up to 0.4 m) observed at/near the crest edge of the both upstream and downstream sides and within the gabions, particularly between STA 0+500 and STA 0+850.

PHOTO 4.4.9-1
FEW TREES AT THE CREST OF THE NORTH SIDE OF LOCK 39 EARTHFILL EMBANKMENT

PHOTO 4.4.9-2
SOME SMALLER TREES AT/ NEAR CREST EDGE OF U/S AND D/S SLOPES OF RIGHT EARTHFILL EMBANKMENT OF THE TALBOT RIVER DAM
(Note the typical trunk diameter was up to 0.2 m.)

PHOTO 4.4.9-3
A FEW SMALLER TREES WITHIN THE STACKED STONE AT THE ABUTMENT OF TALBOT RIVER DAM AT THE SPILLWAY
(Note the typical trunk diameter was up to 0.2 m.)

PHOTO 4.4.9-4
A FEW SMALLER TREES WITHIN THE STACKED STONE AT THE ABUTMENT OF TALBOT RIVER DAM AT THE SPILLWAY
(Note typical trunk diameter was up to 0.2 m.)

PHOTO 4.4.9-5

SOME SMALLER TREES AT/ NEAR CREST EDGE OF U/S AND D/S SLOPES OF SOUTHEAST CANAL EARTHFILL EMBANKMENT

(Note Typical Trunk Diameter was up to 0.2 M.)

PHOTO 4.4.9-6

SOME TREE GROWTH ON TOE OF D/S SLOPE OF NORTHWEST CANAL EARTHFILL EMBANKMENT. LOOKING D/S FROM APPROXIMATELY STA 0+ 750

4.4.9.3 Assessment / Potential Impacts to the Structures

Currently, the tree growth does not appear to be an immediate threat to the dam performance but is a concern in the medium and long term. No dead trees or significantly distressed or uprooted trees were observed. However, there are potential detrimental impacts that could occur from the growth, as discussed below:

- The trees on the Lock 39 earthfill embankment crest are currently not of major concern. There is potential for root penetration into joints within the concrete lock wall or the drainage systems behind the lock walls. However, given the few number of trees and relatively large spacing, the growth here is not considered a significant threat to the structure and removal is not necessary at this time. Ongoing inspection for detrimental impacts needs to continue. In the event that the trees die or become uprooted, then repairs will be necessary.
- The smaller tree and brush growth on the upper portion of the downstream slope of the Left and Right Earthfill Embankments at Talbot River Dam is in a less critical area relative to embankment performance, particularly as the concrete core wall provides the water seepage barrier. Ongoing tree growth in this vicinity is unlikely to have a significant detrimental impact in the short term to the performance of the embankments, although removal of the tree growth and prevention measures for future growth would be preferred. Ongoing monitoring is required however, particularly to identify potential dead or fallen trees in the future as this could cause disturbance of the soils and ultimately loss of support for the core wall. It is unlikely that the loss of any specific tree would threaten the support for the core wall, and repairs could be initiated if required.
- The smaller tree and brush growth on the upper portion of the upstream slope of the Talbot River Dam embankments is of a higher concern. There is a risk of increased erosion with loosened soils, particularly with dead or fallen trees. Although the riprap appears to be intact within or behind the trees, there is risk of erosion with loosened soil or future fallen trees. However, it is unlikely that significant damage to the embankments or breach would occur associated with tree affects, as a local loss of material with erosion would not cause catastrophic loss of support for the wall. In the event that fallen trees are observed, repairs should be initiated in a timely fashion. Note that the removal of the smaller tree and brush growth in particular is preferred; however, this would require significant reconstruction efforts to re-establish the upper slope and riprap.

- The few smaller trees within the stacked stone at the north and south abutments of Talbot River Dam to the Spillway may disturb the stability of the stacked stone, particularly with fallen or dead trees. Note that the stone wall appeared intact at the time of the inspection, although the shape of the walls suggested possible movement / settlement in the past, particularly in the lower half of the wall near the concrete structure at north abutment. Removal of the trees and their root systems is preferred for both abutments, although this would require reconstruction of the stone walls. Careful construction methods would be necessary to ensure that there would be no loss of support for the concrete core wall. In the short term, monitoring on the abutments is warranted for damage, particularly with fallen trees after high wind events. In the event of tree loss, loss of support for the core wall is not likely, and repairs could then be initiated.
- The smaller tree and brush growth on the upper portion of the upstream slope of the canal embankments is of a higher concern due to narrow crest width of the embankments. There is a risk of increased erosion with loosened soils, particularly with dead or fallen trees. In the event that fallen trees are observed, repairs should be initiated in a timely fashion. Note that the removal of the smaller tree and brush growth in particular is preferred; however, this would require significant reconstruction efforts to re-establish the upper slope.
- The smaller tree and brush growth on the upper portion of the downstream slope of the canal embankments is in a less critical area and is typically a lower concern relative to embankment performance. However, if there is any shallow sloughing or movement of the embankment, the vegetation growth could impede the identification of such problem. Ongoing tree growth in this vicinity is unlikely to have a significant detrimental impact to the performance of the embankments, although removal of the tree growth and prevention measures for future growth would be preferred. Ongoing monitoring is required however, particularly to identify potential dead or fallen trees in the future as this could cause disturbance of the soils.
- The tree and brush growth on the downstream lower slope and toe area of the canal embankments is typically is the most critical area and is a higher concern relative to the embankment performance. The saturation and seepage lines associated with the flow exiting at the lower portion of the slope. However, the monitored water level at the canal embankment has shown some head loss across the clayey silt to silty clay embankment fill which appears to be providing a reasonable head loss and control the seepage and as no wet conditions or seepage discharge were observed at the downstream slope and the toe

areas during the inspection and the soil investigation. There is a risk of piping and internal erosion with loosened soils, particularly with dead or fallen trees. In the event that fallen trees are observed, repairs should be initiated in a timely fashion. Note that the removal of the tree and brush growth in particular is preferred; however, this would require significant reconstruction efforts to re-establish the slope and toe.

It is recommended that PCA be proactive and carry out the vegetation removal suggested for the embankments and the abutments within the next 2 years. For any trees having a stump diameter less than 300 mm, they can be cut flush with the ground level and the stump and roots left in place. Although removal of all stumps, rootballs and root systems would provide a better and more conservative approach, it is not considered necessary at this location. For trees with a stump diameter greater than 300 mm, the stumps, root ball and roots greater that 15 mm in diameter should be removed. In addition, all loose soil around the root ball cavity should be removed, and replaced with well compacted competent soil, similar in nature to the surrounding material in relatively thin (200 mm thick) lifts. Careful construction methods are necessary to ensure that there is no risk to the structures or support of the core wall during the work.

Visual monitoring of all vegetation should continue, with particular attention during and after storm or high wind events as falling trees could cause damage to the earthfill embankments and dam structures. If any distress is observed, including material loss or shallow slope movement/ failure, then temporary repair or protection measures of the embankment should be undertaken on an as required basis.

4.4.10 Geotechnical Conclusions and Recommendations

In general, the geotechnical performance of the Talbot River Dam and Lock 39 structures has been fair to good, with no major concerns or problems identified. The geotechnical performance of the Talbot Canal Embankments has been poor to fair with some concerns identified. Some of the key observations and findings include the following:

- There were no signs of erosion, settlement, back-cutting and / or undermining of the foundation of the Talbot River Dam and Lock 39 structure.
- The headpond and tailrace shorelines were in good condition overall, based on the visual inspection.

- The dam abutments at Talbot River Dam were found to be in good condition geotechnically with no evidence of slope movement, erosion or seepage.
- Lock 39 earthfill embankments have performed well geotechnically. There were no slope movements, significant erosion or seepage that would suggest significant concerns related to the slope stability and safety of the structures. It should be noted that the earthfill embankment has a wider crest width (12 to 15 m). Any shallow sloughing and shallow slope movement at the downstream slope will not immediately impact the dam performance due to this wider crest width.
- Right Earthfill Embankment at Talbot River Dam has performed well with no concerns observed related to deep seated slope failures, seepage or settlement with the exception of the exposed concrete core wall due to shallow sloughing or erosion at the upper portion of the slope.
- The concrete core wall exposed area currently does not appear to be jeopardizing the stability of the embankment in the short term. However, increased visual monitoring of this section should be performed, particularly during storm or high wind events to monitor for changes in conditions. If additional distress or material loss is observed, then temporary protection measures such as placement of riprap material or fill buttressing may be required. This can be determined on an as-required basis. Some remedial work is expected to be required within the next 10 years. This repair may require a bathymetric survey and detailed stability analyses.
- Left Earthfill Embankment at Talbot River Dam has performed well with no concerns observed related to deep seated slope failures, seepage or settlement. The downstream (dry side) slopes of the Northwest and Southeast Canal Earthfill Embankments have remained functional and effective in containing the canal water. There have been no significant instability or deficiencies related to their performance observed. However, loss of the upper slope, deterioration of the gabion baskets, and erosion of the inner (wet) slope has been ongoing.
- The upstream (wet side) canal slopes are generally in poor to fair condition due to loss of the upper slope, deterioration of the gabion baskets, and significant erosion of the inner (wet) slope. Ongoing erosion on upstream slope (wet side) is anticipated to be a relatively slow process as there are no significant waves except the waves associated with boat traffic. The overall risk of breach of the canal embankment is relatively low considering the relatively low height of the embankments. However, the embankment cannot be considered

to remain stable over the long-term without repair. Repair to the canal slope is expected to be required within the next 10 years. Inspection and monitoring of the canal embankments should be increased until the repairs are initiated. A few disturbances in the Southeast Canal Embankment were also observed between STA. 0+050 and STA. 0+250. These disturbances appear to have been man-made. Their condition should be monitored.

- Clearing of the trees and vegetation on the embankments and abutments of the dam is recommended to be carried out as specified in Section 4.4.9.3. Clearing the trees and vegetation will not only prevent damage to the embankments and abutments but will improve the ability for inspection of the slopes for potential movement.

5.0 DAM CLASSIFICATION REVIEW

In accordance with the PCA Directive are to be classified following a consistent set of rules that are described in the directive. This classification, in turn, allows the selection of design and maintenance guidelines to be followed throughout the life cycle of the structures.

As part of the dam safety review for the Talbot River Dam, Portage Lock 39 and associated structures, a preliminary dam hazard classification was obtained during the initial stages of the study based on a general review of the structures, their impoundment and the infrastructure that exists downstream. Following this review, a more rigorous assessment was conducted using the results of dam breach simulations carried out with a hydrodynamic numerical model. This section of the report describes the methodology and findings that support the proposed dam hazard classification, while other hydrotechnical aspects of the Dam Safety Review are addressed in Section 6.0.

5.1 PRELIMINARY DAM CLASSIFICATION

It was indicated to KGS Group, at the onset of the project, that an interim hazard classification of **HIGH A** had been previously adopted for the Talbot River Dam. There was no interim classification for Portage Lock 39 or for the dykes between the dam and the lock. As requested in the terms from this project, prior to any dam breach analysis, KGS Group provided a preliminary hazard classification for the structures. A cursory review concluded that there could be loss of life resulting from a breach of these, although it would most likely be in the range of 1 to 10. Therefore the proposed preliminary hazard classification was:

- HIGH A for the Talbot River Dam. Due to the presence of residences in the anticipated flood zone loss of life resulting from a breach of the dam cannot be discounted. However, these would be limited by the small volume of water stored by the dam and the limited number of residences in the flooded area.
- HIGH A for Portage Lock 39. The population that would be affected by the breach of the lock would be minimal but loss of life cannot be entirely discounted.
- HIGH A for the dyke between the dam and Portage Lock 39. There are some residences located next to the dyke and these could be impacted directly if a breach were to occur at

those locations. However, as the distance from the dykes increase, the severity of the flood would substantially reduce, with less likelihood of loss of life.

The hazard classification of these structures was subsequently reviewed with a more rigorous analysis, as indicated in the following sections.

5.2 DAM BREAK ANALYSIS AND INUNDATION MAPPING

The basis for a rigorous analysis of dam breach downstream hazards and incremental consequences is the deployment of a numerical hydrodynamic model. This model is used to evaluate the hydraulic conditions that would occur in the river valley and floodplain under various flood and non-flood scenarios, with and without the breach of the dam. The results of the hydrodynamic model, combined with the topographic data and the available database of buildings and features in the area downstream of the dam, allow estimation of the flood extents and the damages and consequences that can be attributed to the failure of the dam. This approach was followed in the assessment of the hazard classification for the Talbot River Dam and Portage Lock 39. Simulations of a breach at the dykes between the dam and the lock were not carried out for classification of the dyke. Instead, the possible consequences of a breach of the dykes were assessed in general and using the results from the simulations done for the other structures.

5.2.1 Dam Breach Characteristics

To simulate the dam breach of the structures investigated in this study, a number of parameters were selected to define the breach final geometry and developing time. Rather than a precise prediction, these parameters constitute reasonable assumptions, commensurate with the level of effort and accuracy required for dam hazard classification.

The guidelines provided in the document Selecting and Accommodating Inflow Design Floods for Dams (Ref 15) from FERC were used to select the dam breach parameters for the Talbot River Dam. Appendix II-A of those guidelines recommends for concrete dams to adopt a breach width that is equal or less than one-half of the crest length. Considering that the length of the

concrete structure at the Talbot River Dam is approximately 48 m, a hypothetical dam breach with a width of nearly one half of that length could include:

- The sluiceway structure with the piers (22.6 m), or
- One gravity wall (either the north one or the south one) with the adjacent spillway and sluiceway up to the central pier (25.3 m),

The latter case was used. It was assumed that the sluiceway logs would be in place for the sunny-day scenario and that they would be removed during a flood.

Since the foundation of the dam is on rock, it was assumed that it would not be eroded significantly during the outflow in the case of a breach. It was also assumed that the entire concrete blocks in the area of the breach would be removed, so the breach would have vertical side slopes. A time of breach development of 0.2 hours was adopted, which is consistent with the values suggested in Ref 15. The dam breach parameters used for the simulation of a breach of the concrete structure at the Talbot River Dam are summarized in Table 5.2-1.

TABLE 5.2-1
PARAMETERS USED TO SIMULATE A BREACH OF THE CONCRETE STRUCTURE AT THE TALBOT RIVER DAM

Type Of Dam	Concrete - Gravity
Crest Elevation (m)	232.02 (deck level)
Reservoir Level at time of breach (m)	230.44 (sunny day) – 232.02 (flood)
Breach Bottom Elevation (m)	225.16 (assumed bedrock level)
Dam Height (m)	6.86 (from the deck)
Breach top width (m)	25.3
Breach width at bottom (m)	25.3
Breach development time (h)	0.2
Breach side slope (Z)	Vertical

It must be recognized that a breach of this size, would be the maximum value indicated in FERC (1993). However, it is in line with other criteria used by some agencies in Canada. For instance, the guidelines used by OPG (2006) recommend a breach size of 2 to 3 times the dam height. This criterion applied to the Talbot River Dam corresponds to a breach width of approximately 20.5m to 27.4 m. There is no certainty on the actual size of a hypothetical breach but failure of

one half or even the full length have occurred in concrete gravity dams, as reported in Veale and Davinson (2013). The implications of adopting a smaller dam breach are discussed in Section 5.4.1.

If the failure is on the embankment, the recommended breach size in Ref 15 corresponds to a trapezoidal breach with an average width of up to 5 times the height of the dam and 1 to 1 side slopes. The crest of the embankment is at El. 231.26 m, the foundation level is estimated to be at El. 225.16 m, for a 6.1 m height. The average breach width was then estimated as 30.5 m. Table 5.2-2 shows the corresponding breach parameters.

TABLE 5.2-2
PARAMETERS USED TO SIMULATE A BREACH OF THE EARTH EMBANKMENT AT THE TALBOT RIVER DAM

Type of Dam	Earth Embankment
Crest Elevation (m)	231.26 (crest level)
Reservoir Level at time of breach (m)	230.44 (sunny day) – 231.26 (flood)
Breach Bottom Elevation (m)	225.16 (assumed bedrock level)
Dam Height (m)	6.10 (from the crest)
Breach top width (m)	36.6
Breach width at bottom (m)	24.4
Breach development time (h)	1
Breach side slope (Z)	1:1

The breach of Portage Lock 39 was simulated as a sudden collapse of the lock gates. For modelling purposes the bottom of the breach was adopted as the sill level upstream of the lock gates and the breach developing time was limited to 0.2 hours as a short time that still allowed stable simulations with the dam break model. The lock was assumed to be fully closed at the time of the breach, both for sunny-day and flood conditions, with all flows being passed through the Talbot River Dam. The crest of the wing walls at Lock 39 is at El. 231.24 m. The top of the lock gates has been estimated as El. 230.78 m. The proposed dam breach parameters for Lock 39 are shown in Table 5.2-3.

TABLE 5.2-3
PARAMETERS USED TO SIMULATE A BREACH AT LOCK 39

Type of Dam	Stacked Timber Gates
Crest Elevation (m)	230.78 (top lock gates)
Reservoir Level at time of breach (m)	230.44 (sunny day) – 231.24 (flood)
Breach Bottom Elevation (m)	228.4 (sill level)
Dam Height (m)	2.38
Breach top width (m)	10.0
Breach width at bottom (m)	10.0
Breach development time (h)	0.2
Breach side slope (Z)	Vertical

For a sunny-day failure, the dam breach was assumed to occur with the reservoir at full supply level (FSL), adopted to be El. 230.44 m as indicated in a KGS Group's memorandum to discuss breach parameters and assumptions, issued in 2012 (Ref. 16). The river flow for the sunny-day dam breach simulation was assumed to be a constant value of 2 m^3/s, which is approximately the average of the Talbot Dam outflows, obtained from records from September 1986 to October 1988. Separate simulations were done for sunny-day breach of the Talbot River Dam and sunny-day breach of Portage Lock 39.

For the analysis of a dam breach during a flood, conducted to select the Inflow Design Flood (IDF), the procedure recommended in the FERC guidelines (Ref. 15) was adopted. It consists of assuming that the dam would be theoretically modified to pass any inflow floods that are lower than the simulated event (i.e. the dam is just at the point of overtopping at the peak of the simulated flood). This approach allows identifying the flood scenario that, when marginally exceeded, generates the consequences that justify adopting that event as the IDF. The conditions used in the simulations conducted to select the IDF were, therefore, adjusted so that the peak flood level matched the crest level and that the dam breach occurred at the time at which that level was reached. The adopted crest levels for each structure are those shown in Tables 5.2.1 to 5.2.3.

Separate dam breach simulations were carried out to analyze the breach of each structure at the site (Talbot River Dam concrete structure, Talbot River Dam earth embankment and Portage Lock 39). In each case, the structure under study was assumed to be the only one to fail, with the other two structures staying in place. It is possible that in a real scenario more than

one structure fails; but the assumed condition represents the worst case for each of the structures individually and it was, therefore, considered appropriate for the purpose of dam hazard classification.

To allow the analysis of the dam breach consequences that can be attributed to a potential failure at the Talbot River Dam or at Portage Lock 39, it was assumed that the Talbot Dam, located upstream, would stay in place in all cases. For the locks, that are located in the Talbot Canal downstream of Portage Lock 39, it was assumed that these would breach in any scenario in which these would be overtopped by 0.6 m or more. The breach of these locks was assumed to be a complete collapse of the gates with bottom elevation at the upstream sills.

The following floods were evaluated using the method previously described:

- Flood one-third of the way between the 1,000-year flood and the PMF
- Flood two-thirds of the way between the 1,000-year flood and the PMF
- PMF

To allow evaluation of dam break incremental consequences, four simulations were done for each flood scenario: no dam breach, breach of the concrete structure at the Talbot River Dam, breach of the Talbot River Dam earth embankment and breach of Lock 39.

Once the IDF was selected for the site, additional simulations were conducted to model the IDF using the following conditions, as requested in the RFP:

- Initial water level in the reservoir at FSL
- Configuration of discharge weirs based on operation procedures and capability of the operating crews

The simulation of the IDF with these assumptions were carried out for two scenarios: safe passage of the flood and breach of the corresponding structure (dam or lock) at the peak of the flood.

A dam break simulation program was not used to classify the earth embankments (north and south) along the Talbot Canal. This structure was classified based on a general review of the conditions that surround it and considering the results of the simulations done for the other structures.

5.2.2 Hydrodynamic Routing of the Flood Wave

A numerical hydrodynamic model was developed using the USACE software HEC-RAS Version 4.1.0. This software is widely used in the dam safety field and is an accepted basis for estimating effects of a dam failure. The HEC-GEO RAS tool was used to obtain the geometric information, directly in HEC-RAS format, from a digital elevation model (DEM) available from the Ontario Ministry of Natural Resources Water Resource Information Program (WRIP).

KGS Group used the current data available from the Ontario Land Initiative to locate buildings in the flood zone. That spatial dataset is only accurate in location to the normal limits associated with 1:20,000 scale maps, and does not contain information on building type or age. This data was supplemented with buildings identified in aerial photographs and during the site visit by KGS Group.

For the purpose of dam break modeling, KGS Group prepared models of the river reach from immediately downstream of the Bolsover Dam to the discharge of the Talbot River and Talbot Canal at Lake Simcoe, approximately 13 km downstream. The model was prepared using over 110 cross sections obtained through HEC-GEO RAS. Additional cross sections were interpolated at intervals that ranged from 50 m to 250 m.

The boundary conditions for the model included:

- the inflow hydrographs at the upstream end of the model; and
- a constant water level of El. 219.0 m (approximately average level of Lake Simcoe) adopted as downstream boundary condition for the model.

The volume of Lake Simcoe is so large that the lake levels would not change in response to inflows either from natural floods or from a breach of the Talbot River Dam or Lock 39.

Representative Manning's n-values were used to simulate the resistance to flow along the river bed and banks. Manning's n-values of 0.03 and 0.07 were adopted for the river channel and for the overbank areas, respectively. These were assigned based on the conditions observed during the site visit and on the available aerial imagery from Google Earth.

The following components were included in the model:

- Talbot River cross sections from downstream of the Bolsover Dam to the Talbot Dam.
- The structures at the Talbot Dam, based on dimensions obtained from available drawings and survey data.
- Talbot River cross sections from downstream of the Talbot Dam to the Talbot River Dam.
- The structures at the Talbot River Dam and the earth embankment, based on dimensions obtained from available drawings and survey data.
- Talbot River cross sections from downstream of the Talbot River Dam to the Talbot Canal and from the Talbot Canal to Lake Simcoe.
- Talbot Canal cross sections from the Talbot River Dam to Lake Simcoe. The entrance of the canal at the Talbot River Dam forebay was represented as a lateral structure in HEC-RAS.
- Lock 39, based on dimensions available from drawings and survey data,
- Lock 40 and Lock 41, based on dimensions available from the existing drawings,

The results of the simulations are provided in the following tables for a dam breach in sunny day conditions:

- Table 5.2-4 for the sunny-day failure of the Talbot River Dam concrete structures
- Table 5.2-5 for the sunny-day failure of the Talbot River Dam earth embankment
- Table 5.2-6 for the sunny-day failure of the Portage Lock 39

The results for a dam breach during the flood one-third of the way between the 1,000-year flood and the PMF are provided in the following tables:

- Table 5.2-7 for the breach of the Talbot River Dam concrete structures during the flood one-third of the way between the 1,000-year flood and the PMF

- Table 5.2-8 for the breach of the Talbot River Dam earth embankment during the flood one-third of the way between the 1,000-year flood and the PMF
- Table 5.2-9 for the breach of the Portage Lock 39 during the flood one-third of the way between the 1,000-year flood and the PMF

The results for a dam breach during the flood two-thirds of the way between the 1,000-year flood and the PMF are provided in the following tables:

- Table 5.2-10 for the breach of the Talbot River Dam concrete structures during the flood two-thirds of the way between the 1,000-year flood and the PMF
- Table 5.2-11 for the breach of the Talbot River Dam earth embankment during the flood two-thirds of the way between the 1,000-year flood and the PMF
- Table 5.2-12 for the breach of the Portage Lock 39 during the flood two-thirds of the way between the 1,000-year flood and the PMF

The results for a dam breach during the PMF are provided in the following tables:

- Table 5.2-13 for the breach of the Talbot River Dam concrete structures during the PMF.
- Table 5.2-14 for the breach of the Talbot River Dam earth embankment during the PMF.
- Table 5.2-15 for the breach of the Portage Lock 39 during the PMF.

TABLE 5.2-4
RESULTS FROM SIMULATION OF SUNNY DAY FAILURE OF THE SLUICEWAY STRUCTURE AT THE TALBOT RIVER DAM

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[1] (h)	Wave Arrival Time[1] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	2	234.8		NA	0.0	2	234.8
Upstream of Talbot Dam	0.0	11008.4	2	234.8		NA	0.0	2	234.8
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	2	230.5	0.0	NA	0.0	2	230.5
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	90	230.5	0.0	NA	0.0	2	230.5
0.5 km downstream of Lock 38	0.5	10500.7	150	230.5	0.0	NA	0.0	2	230.5
Upstream of Talbot River Dam	1.1	9864.4	450	230.5	0.0	NA	0.0	2	230.5
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	450	228.6	0.3	0.0	2.8	2	225.5
0.5 km from Talbot River Dam	1.6	9366.4	360	228.0	0.4	0.1	2.5	2	225.3
0.9 km from Talbot River Dam	2.0	9031.8	310	227.9	0.4	0.1	1.8	2	225.1
2.2 km from Talbot River Dam	3.3	7683.9	260	224.6	0.7	0.3	2.1	2	221.7
3.9 km from Talbot River Dam	5.0	5977.5	160	223.7	1.0	0.5	1.9	2	221.2
5.2 km from Talbot River Dam	6.3	4723.1	140	222.7	1.4	0.6	1.3	2	220.1
Gamebridge near Hazel St.	7.0	4007.5	130	222.5	1.5	0.7	1.4	2	220.0
Gamebridge bridge at Ramara Road 51	7.3	3651.0	120	222.2	1.5	0.8	1.5	2	219.9
Bridge at Hwy 12	8.6	2442.6	120	221.1	1.8	1.0	1.4	2	219.1
Near Furniss Drive	10.0	1010.1	100	220.4	2.3	1.1	1.1	2	219.0
Talbot River near confluence with canal	10.9	62.8	100	219.1	2.4	1.1	1.1	2	219.0
Trent Talbot Marina	5.9	1084.2	40	219.1	2.4	1.1	0.4	1	219.0
CN Rail bridge	6.3	652.1	40	219.1	2.4	1.1	0.4	1	219.0
Bridge at Ramara Road 47	6.8	156.9	40	219.1	2.5	1.2	0.4	1	219.0
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	0	230.4	0.0	NA	0.1	0	230.4
Upstream of Portage Lock 39	2.4	4567.0	0	230.4	0.0	NA	0.1	0	230.4
Downstream of Portage Lock 39	2.5	4528.0	0	226.2	0.0	NA	0.2	0	226.2
Upstream of Lock 40	3.2	3797.0	0	226.2	0.0	NA	0.2	0	226.2
Downstream of Lock 40	3.2	3758.0	0	221.9	0.0	NA	0.1	0	221.9
Upstream of Lock 41	4.3	2712.0	0	221.9	0.0	NA	0.1	0	221.9
Downstream of Lock 41	4.3	2673.0	0	219.1	2.5	1.3	0.1	0	219.0
Confluence with the Talbot River	5.7	1282.7	60	219.1	2.4	1.2	0.9	1	219.0
CN Rail bridge	5.9	1090.2	60	219.1	2.4	1.2	0.6	1	219.0
Bridge at Ramara Road 47	6.6	405.9	60	219.0	2.5	1.2	0.6	1	219.0

[1] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

TABLE 5.2-5
RESULTS FROM SIMULATION OF SUNNY DAY FAILURE OF THE EARTH EMBANKMENT AT THE TALBOT RIVER DAM

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[1/] (h)	Wave Arrival Time[1/] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	2	234.8		NA	0.0	2	234.8
Upstream of Talbot Dam	0.0	11008.4	2	234.8		NA	0.0	2	234.8
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	2	230.5	0.0	NA	0.0	2	230.5
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	40	230.5	0.0	NA	0.0	2	230.5
0.5 km downstream of Lock 38	0.5	10500.7	80	230.5	0.0	NA	0.0	2	230.5
Upstream of Talbot River Dam	1.1	9864.4	300	230.5	0.0	NA	0.0	2	230.5
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	300	228.4	1.0	0.1	2.6	2	225.5
0.5 km from Talbot River Dam	1.6	9366.4	280	227.9	1.1	0.2	2.4	2	225.3
0.9 km from Talbot River Dam	2.0	9031.8	260	227.7	1.1	0.2	1.7	2	225.1
2.2 km from Talbot River Dam	3.3	7683.9	220	224.5	1.3	0.5	2.0	2	221.7
3.9 km from Talbot River Dam	5.0	5977.5	160	223.7	1.6	0.9	1.8	2	221.2
5.2 km from Talbot River Dam	6.3	4723.1	140	222.7	2.0	1.1	1.3	2	220.1
Gamebridge near Hazel St.	7.0	4007.5	130	222.5	2.1	1.2	1.4	2	220.0
Gamebridge bridge at Ramara Road 51	7.3	3651.0	120	222.2	2.1	1.2	1.4	2	219.9
Bridge at Hwy 12	8.6	2442.6	120	221.0	2.3	1.5	1.4	2	219.1
Near Furniss Drive	10.0	1010.1	100	220.4	2.9	1.6	1.1	2	219.0
Talbot River near confluence with canal	10.9	62.8	100	219.1	3.0	1.1	1.0	2	219.0
Trent Talbot Marina	5.9	1084.2	40	219.1	3.0	1.7	0.4	1	219.0
CN Rail bridge	6.3	652.1	30	219.1	3.0	1.8	0.4	1	219.0
Bridge at Ramara Road 47	6.8	156.9	30	219.1	3.1	1.8	0.4	1	219.0
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	0	230.4	0.0	NA	0.1	0	230.4
Upstream of Portage Lock 39	2.4	4567.0	0	230.4	0.0	NA	0.1	0	230.4
Downstream of Portage Lock 39	2.5	4528.0	0	226.2	0.0	NA	0.2	0	226.2
Upstream of Lock 40	3.2	3797.0	0	226.2	0.0	NA	0.2	0	226.2
Downstream of Lock 40	3.2	3758.0	0	221.9	0.0	NA	0.1	0	221.9
Upstream of Lock 41	4.3	2712.0	0	221.9	0.0	NA	0.1	0	221.9
Downstream of Lock 41	4.3	2673.0	0	219.1	3.0	1.8	0.1	0	219.0
Confluence with the Talbot River	5.7	1282.7	60	219.1	3.0	1.7	0.9	1	219.0
CN Rail bridge	5.9	1090.2	60	219.1	3.0	1.7	0.6	1	219.0
Bridge at Ramara Road 47	6.6	405.9	60	219.0	3.1	1.7	0.6	1	219.0

[1/] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

TABLE 5.2-6
RESULTS FROM SIMULATION OF SUNNY DAY FAILURE OF PORTAGE LOCK 39

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[/1/] (h)	Wave Arrival Time[/1/] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	2	234.8		NA	0.0	2	234.8
Upstream of Talbot Dam	0.0	11008.4	2	234.8		NA	0.0	2	234.8
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	2	230.5	0.0	NA	0.0	2	230.5
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	2	230.5	0.0	NA	0.0	2	230.5
0.5 km downstream of Lock 38	0.5	10500.7	2	230.5	0.0	NA	0.0	2	230.5
Upstream of Talbot River Dam	1.1	9864.4	2	230.5	0.0	NA	0.0	2	230.5
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	2	225.5	0.0	NA	0.3	2	225.5
0.5 km from Talbot River Dam	1.6	9366.4	2	225.3	0.0	NA	0.3	2	225.3
0.9 km from Talbot River Dam	2.0	9031.8	2	225.1	0.0	NA	0.4	2	225.1
2.2 km from Talbot River Dam	3.3	7683.9	2	221.7	0.0	NA	0.3	2	221.7
3.9 km from Talbot River Dam	5.0	5977.5	2	221.2	0.0	NA	0.5	2	221.2
5.2 km from Talbot River Dam	6.3	4723.1	2	220.1	0.0	NA	0.2	2	220.1
Gamebridge near Hazel St.	7.0	4007.5	2	220.0	0.0	NA	0.2	2	220.0
Gamebridge bridge at Ramara Road 51	7.3	3651.0	2	219.9	0.0	NA	0.2	2	219.9
Bridge at Hwy 12	8.6	2442.6	3	219.2	1.0	0.8	0.2	2	219.1
Near Furniss Drive	10.0	1010.1	4	219.1	0.8	0.6	-0.1	2	219.0
Talbot River near confluence with canal	10.9	62.8	10	219.1	0.7	1.1	0.0	2	219.0
Trent Talbot Marina	5.9	1084.2	40	219.1	0.7	1.7	0.2	1	219.0
CN Rail bridge	6.3	652.1	30	219.0	0.7	1.8	0.3	1	219.0
Bridge at Ramara Road 47	6.8	156.9	30	219.0	1.1	1.8	0.0	1	219.0
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	30	230.4	0.0	0.1	0.1	0	230.4
Upstream of Portage Lock 39	2.4	4567.0	30	230.4	0.0	0.1	0.1	0	230.4
Downstream of Portage Lock 39	2.5	4528.0	30	227.4	0.4	0.1	1.2	0	226.2
Upstream of Lock 40	3.2	3797.0	40	227.3	0.4	0.1	0.9	0	226.2
Downstream of Lock 40	3.2	3758.0	40	223.2	0.6	0.1	1.0	0	221.9
Upstream of Lock 41	4.3	2712.0	70	223.1	0.6	0.3	0.9	0	221.9
Downstream of Lock 41	4.3	2673.0	70	219.5	0.8	0.3	2.3	0	219.0
Confluence with the Talbot River	5.7	1282.7	40	219.1	0.7	0.5	0.3	1	219.0
CN Rail bridge	5.9	1090.2	40	219.1	0.7	0.5	0.3	1	219.0
Bridge at Ramara Road 47	6.6	405.9	40	219.0	0.7	0.5	0.2	1	219.0

[/1/] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

TABLE 5.2-7
RESULTS FROM SIMULATION OF A BREACH OF THE SLUICEWAY STRUCTURE AT THE TALBOT RIVER DAM DURING THE FLOOD 1/3 OF THE WAY BETWEEN THE 1,000-YR FLOOD AND THE PMF

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[/1/] (h)	Wave Arrival Time[/1/] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	210	235.6		NA	0.3	210	235.6
Upstream of Talbot Dam	0.0	11008.4	210	235.6		NA	0.3	210	235.6
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	220	232.0	0.0	NA	0.8	210	232.0
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	300	232.0	0.0	NA	0.5	210	232.0
0.5 km downstream of Lock 38	0.5	10500.7	360	232.0	0.0	NA	0.2	210	232.0
Upstream of Talbot River Dam	1.1	9864.4	810	232.0	0.0	NA	0.1	170	232.0
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	810	229.5	0.3	0.0	3.3	170	227.9
0.5 km from Talbot River Dam	1.6	9366.4	670	228.8	0.3	0.0	3.9	170	227.5
0.9 km from Talbot River Dam	2.0	9031.8	630	228.8	0.4	0.0	1.7	170	227.3
2.2 km from Talbot River Dam	3.3	7683.9	550	225.8	0.7	0.0	2.6	170	224.6
3.9 km from Talbot River Dam	5.0	5977.5	390	225.1	0.9	0.2	2.4	170	223.9
5.2 km from Talbot River Dam	6.3	4723.1	360	224.0	1.1	0.2	2.3	170	223.1
Gamebridge near Hazel St.	7.0	4007.5	350	223.5	1.2	0.3	2.5	170	222.8
Gamebridge bridge at Ramara Road 51	7.3	3651.0	350	223.3	1.3	0.3	2.2	170	222.5
Bridge at Hwy 12	8.6	2442.6	340	222.0	1.5	0.3	2.6	170	221.4
Near Furniss Drive	10.0	1010.1	320	220.9	2.0	0.5	1.8	170	220.6
Talbot River near confluence with canal	10.9	62.8	310	219.7	2.3	0.8	2.5	170	219.4
Trent Talbot Marina	5.9	1084.2	120	219.5	2.4	0.9	1.1	80	219.3
CN Rail bridge	6.3	652.1	120	219.3	2.5	1.0	1.4	80	219.1
Bridge at Ramara Road 47	6.8	156.9	120	219.1	2.8	1.1	1.2	80	219.1
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	50	231.9	0.0	0.0	0.6	50	231.9
Upstream of Portage Lock 39	2.4	4567.0	50	231.6	0.0	0.0	1.3	50	231.6
Downstream of Portage Lock 39	2.5	4528.0	50	227.6	0.0	0.0	1.6	50	227.6
Upstream of Lock 40	3.2	3797.0	50	227.4	0.0	0.0	1.4	50	227.4
Downstream of Lock 40	3.2	3758.0	50	223.4	0.0	0.0	1.5	50	223.4
Upstream of Lock 41	4.3	2712.0	70	223.2	0.0	0.2	1.3	70	223.2
Downstream of Lock 41	4.3	2673.0	70	219.7	2.3	0.8	2.3	70	219.7
Confluence with the Talbot River	5.7	1282.7	190	219.7	2.3	0.8	2.0	130	219.4
CN Rail bridge	5.9	1090.2	190	219.5	2.3	0.8	2.1	130	219.3
Bridge at Ramara Road 47	6.6	405.9	190	219.1	2.4	1.0	1.7	130	219.1

[/1/] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

TABLE 5.2-8
RESULTS FROM SIMULATION OF A BREACH OF THE EARTH EMBANKMENT AT THE TALBOT RIVER DAM DURING THE FLOOD 1/3 OF THE WAY BETWEEN THE 1,000-YR FLOOD AND THE PMF

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[/1/] (h)	Wave Arrival Time[/1/] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	210	235.6		NA	0.3	210	235.6
Upstream of Talbot Dam	0.0	11008.4	210	235.6		NA	0.3	210	235.6
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	220	231.3	0.0	NA	1.0	210	231.3
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	250	231.3	0.0	NA	0.5	210	231.3
0.5 km downstream of Lock 38	0.5	10500.7	280	231.3	0.0	NA	0.2	210	231.3
Upstream of Talbot River Dam	1.1	9864.4	490	231.3	0.0	NA	0.2	210	231.3
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	490	229.1	1.0	0.0	3.0	210	228.1
0.5 km from Talbot River Dam	1.6	9366.4	490	228.5	0.9	0.1	3.2	210	227.7
0.9 km from Talbot River Dam	2.0	9031.8	510	228.5	0.9	0.1	1.5	210	227.5
2.2 km from Talbot River Dam	3.3	7683.9	470	225.7	1.2	0.2	2.5	210	224.8
3.9 km from Talbot River Dam	5.0	5977.5	360	225.0	1.4	0.3	2.3	210	224.2
5.2 km from Talbot River Dam	6.3	4723.1	340	223.9	1.6	0.4	2.2	210	223.3
Gamebridge near Hazel St.	7.0	4007.5	330	223.4	1.7	0.5	2.4	210	223.0
Gamebridge bridge at Ramara Road 51	7.3	3651.0	330	223.2	1.7	0.5	2.2	210	222.7
Bridge at Hwy 12	8.6	2442.6	320	221.9	2.0	0.6	2.5	210	221.6
Near Furniss Drive	10.0	1010.1	310	220.9	2.4	0.8	1.8	170	220.7
Talbot River near confluence with canal	10.9	62.8	300	219.7	2.7	1.0	2.5	210	219.4
Trent Talbot Marina	5.9	1084.2	120	219.5	2.8	1.0	1.1	80	219.3
CN Rail bridge	6.3	652.1	120	219.3	2.9	1.1	1.4	80	219.1
Bridge at Ramara Road 47	6.8	156.9	120	219.1	3.2	1.3	1.2	80	219.1
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	10	231.3	0.0	0.0	0.1	10	231.3
Upstream of Portage Lock 39	2.4	4567.0	10	231.3	0.0	0.0	0.2	10	231.3
Downstream of Portage Lock 39	2.5	4528.0	10	227.1	0.1	0.1	0.4	10	227.1
Upstream of Lock 40	3.2	3797.0	10	227.1	0.1	0.1	0.3	10	227.1
Downstream of Lock 40	3.2	3758.0	10	222.8	0.1	0.1	0.2	10	222.8
Upstream of Lock 41	4.3	2712.0	10	222.8	0.2	0.2	0.2	10	222.8
Downstream of Lock 41	4.3	2673.0	10	219.6	2.7	1.0	0.1	10	219.4
Confluence with the Talbot River	5.7	1282.7	180	219.7	2.7	1.0	2.0	140	219.4
CN Rail bridge	5.9	1090.2	180	219.5	2.7	1.0	2.1	140	219.3
Bridge at Ramara Road 47	6.6	405.9	180	219.1	2.8	1.0	1.6	140	219.1

[/1/] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

TABLE 5.2-9
RESULTS FROM SIMULATION OF A BREACH OF PORTAGE LOCK 39 DURING THE FLOOD 1/3 OF THE WAY BETWEEN THE 1,000-YR FLOOD AND THE PMF

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[1/] (h)	Wave Arrival Time[1/] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	210	235.6		NA	0.3	210	235.6
Upstream of Talbot Dam	0.0	11008.4	220	235.6		NA	0.3	210	235.6
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	220	231.3	0.0	NA	1.0	210	231.3
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	220	231.3	0.0	NA	0.5	210	231.3
0.5 km downstream of Lock 38	0.5	10500.7	220	231.3	0.0	NA	0.2	210	231.3
Upstream of Talbot River Dam	1.1	9864.4	210	231.3	0.0	NA	0.2	210	231.3
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	210	228.1	0.0	0.0	2.0	210	228.1
0.5 km from Talbot River Dam	1.6	9366.4	210	227.7	0.1	0.1	2.1	210	227.7
0.9 km from Talbot River Dam	2.0	9031.8	210	227.5	0.1	0.2	1.6	210	227.5
2.2 km from Talbot River Dam	3.3	7683.9	210	224.8	0.2	0.3	1.7	210	224.8
3.9 km from Talbot River Dam	5.0	5977.5	210	224.2	0.4	0.5	1.9	210	224.2
5.2 km from Talbot River Dam	6.3	4723.1	210	223.3	0.5	0.6	1.7	210	223.3
Gamebridge near Hazel St.	7.0	4007.5	210	223.0	0.5	0.7	1.8	210	223.0
Gamebridge bridge at Ramara Road 51	7.3	3651.0	210	222.7	0.6	0.7	1.9	210	222.7
Bridge at Hwy 12	8.6	2442.6	210	221.6	0.7	0.8	2.0	210	221.6
Near Furniss Drive	10.0	1010.1	210	220.7	0.0	0.3	1.4	170	220.7
Talbot River near confluence with canal	10.9	62.8	210	219.6	0.6	0.2	1.8	210	219.4
Trent Talbot Marina	5.9	1084.2	110	219.4	0.6	0.2	1.0	80	219.3
CN Rail bridge	6.3	652.1	110	219.2	0.6	0.2	1.3	80	219.1
Bridge at Ramara Road 47	6.8	156.9	110	219.1	1.2	0.2	1.0	80	219.1
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	30	231.3	0.0	0.0	0.1	10	231.3
Upstream of Portage Lock 39	2.4	4567.0	50	231.3	0.0	0.0	0.2	10	231.3
Downstream of Portage Lock 39	2.5	4528.0	50	227.7	0.3	0.0	1.9	10	227.1
Upstream of Lock 40	3.2	3797.0	60	227.4	0.2	0.0	1.6	10	227.1
Downstream of Lock 40	3.2	3758.0	60	223.4	0.4	0.0	1.6	10	222.8
Upstream of Lock 41	4.3	2712.0	70	223.2	0.3	0.0	1.2	10	222.8
Downstream of Lock 41	4.3	2673.0	70	219.8	0.5	0.0	2.3	20	219.4
Confluence with the Talbot River	5.7	1282.7	160	219.6	0.6	0.2	1.9	140	219.4
CN Rail bridge	5.9	1090.2	160	219.4	0.6	0.2	1.9	140	219.3
Bridge at Ramara Road 47	6.6	405.9	160	219.1	0.6	0.2	1.4	140	219.1

[1/] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

TABLE 5.2-10
RESULTS FROM SIMULATION OF A BREACH OF THE SLUICEWAY STRUCTURE AT THE TALBOT RIVER DAM DURING THE FLOOD 2/3 OF THE WAY BETWEEN THE 1,000-YR FLOOD AND THE PMF

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[/1/] (h)	Wave Arrival Time[/1/] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	280	235.7		NA	0.4	280	235.7
Upstream of Talbot Dam	0.0	11008.4	280	235.7		NA	0.4	280	235.7
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	300	232.0	0.0	NA	1.1	280	232.0
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	360	232.0	0.0	NA	0.6	280	232.0
0.5 km downstream of Lock 38	0.5	10500.7	420	232.0	0.0	NA	0.2	280	232.0
Upstream of Talbot River Dam	1.1	9864.4	850	232.0	0.0	NA	0.2	240	232.0
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	850	229.6	0.3	0.0	2.1	240	228.3
0.5 km from Talbot River Dam	1.6	9366.4	720	228.9	0.3	0.0	2.2	240	227.8
0.9 km from Talbot River Dam	2.0	9031.8	680	228.9	0.3	0.0	1.7	240	227.7
2.2 km from Talbot River Dam	3.3	7683.9	600	226.0	0.8	0.1	1.8	240	225.0
3.9 km from Talbot River Dam	5.0	5977.5	430	225.4	0.9	0.2	2.0	230	224.4
5.2 km from Talbot River Dam	6.3	4723.1	420	224.2	1.1	0.2	1.8	230	223.5
Gamebridge near Hazel St.	7.0	4007.5	410	223.6	1.2	0.3	2.0	230	223.1
Gamebridge bridge at Ramara Road 51	7.3	3651.0	400	223.4	1.2	0.3	2.0	230	222.8
Bridge at Hwy 12	8.6	2442.6	380	222.1	1.7	0.4	2.2	230	221.7
Near Furniss Drive	10.0	1010.1	370	221.0	2.2	0.5	1.7	230	220.7
Talbot River near confluence with canal	10.9	62.8	360	219.9	2.4	0.8	2.0	230	219.6
Trent Talbot Marina	5.9	1084.2	150	219.7	2.6	0.9	1.0	110	219.5
CN Rail bridge	6.3	652.1	140	219.4	2.7	1.0	1.3	110	219.2
Bridge at Ramara Road 47	6.8	156.9	140	219.2	3.0	1.1	1.2	110	219.1
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	50	231.9	0.0	0.0	0.6	50	231.9
Upstream of Portage Lock 39	2.4	4567.0	50	231.6	0.0	0.0	1.3	50	231.6
Downstream of Portage Lock 39	2.5	4528.0	50	227.6	0.0	0.0	1.6	50	227.6
Upstream of Lock 40	3.2	3797.0	50	227.4	0.0	0.0	1.4	50	227.4
Downstream of Lock 40	3.2	3758.0	50	223.4	0.0	0.0	1.5	50	223.4
Upstream of Lock 41	4.3	2712.0	70	223.2	0.0	0.4	1.3	70	223.2
Downstream of Lock 41	4.3	2673.0	70	219.8	2.4	0.8	2.2	70	219.8
Confluence with the Talbot River	5.7	1282.7	220	219.9	2.4	0.8	2.0	170	219.6
CN Rail bridge	5.9	1090.2	220	219.7	2.5	0.8	2.0	170	219.4
Bridge at Ramara Road 47	6.6	405.9	220	219.2	2.5	1.0	1.5	170	219.1

[/1/] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

TABLE 5.2-11
RESULTS FROM SIMULATION OF A BREACH OF THE EARTH EMBANKMENT AT THE TALBOT RIVER DAM DURING THE FLOOD 2/3 OF THE WAY BETWEEN THE 1,000-YR FLOOD AND THE PMF

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[/1/] (h)	Wave Arrival Time[/1/] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	280	235.7		NA	0.4	280	235.7
Upstream of Talbot Dam	0.0	11008.4	280	235.7		NA	0.4	280	235.7
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	300	231.3	0.0	NA	1.4	300	231.3
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	310	231.3	0.0	NA	0.7	280	231.3
0.5 km downstream of Lock 38	0.5	10500.7	340	231.3	0.0	NA	0.3	280	231.3
Upstream of Talbot River Dam	1.1	9864.4	540	231.3	0.0	NA	0.3	280	231.3
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	540	229.2	1.0	0.0	2.3	280	228.4
0.5 km from Talbot River Dam	1.6	9366.4	530	228.5	1.0	0.1	2.4	280	228.0
0.9 km from Talbot River Dam	2.0	9031.8	520	228.5	1.0	0.1	1.7	280	227.9
2.2 km from Talbot River Dam	3.3	7683.9	490	225.9	1.2	0.2	1.9	280	225.2
3.9 km from Talbot River Dam	5.0	5977.5	410	225.3	1.3	0.3	2.1	280	224.7
5.2 km from Talbot River Dam	6.3	4723.1	400	224.1	1.5	0.4	2.0	280	223.7
Gamebridge near Hazel St.	7.0	4007.5	390	223.6	1.6	0.5	2.2	280	223.3
Gamebridge bridge at Ramara Road 51	7.3	3651.0	390	223.4	1.6	0.5	2.1	280	223.0
Bridge at Hwy 12	8.6	2442.6	370	222.1	2.0	0.7	2.3	280	221.8
Near Furniss Drive	10.0	1010.1	360	221.0	2.6	0.8	1.8	280	220.8
Talbot River near confluence with canal	10.9	62.8	350	219.9	2.8	1.0	2.4	280	219.6
Trent Talbot Marina	5.9	1084.2	140	219.6	2.9	1.0	1.0	110	219.4
CN Rail bridge	6.3	652.1	140	219.3	3.0	1.1	1.3	110	219.2
Bridge at Ramara Road 47	6.8	156.9	140	219.2	3.3	1.3	1.2	110	219.1
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	10	231.2	0.0	0.0	0.1	10	231.2
Upstream of Portage Lock 39	2.4	4567.0	10	231.2	0.0	0.0	0.2	10	231.2
Downstream of Portage Lock 39	2.5	4528.0	10	227.1	0.1	0.1	0.3	10	227.1
Upstream of Lock 40	3.2	3797.0	10	227.1	0.1	0.1	0.3	10	227.1
Downstream of Lock 40	3.2	3758.0	10	222.8	0.2	0.2	0.2	10	222.8
Upstream of Lock 41	4.3	2712.0	10	222.8	0.3	0.3	0.2	10	222.8
Downstream of Lock 41	4.3	2673.0	10	219.8	2.8	1.0	0.2	10	219.5
Confluence with the Talbot River	5.7	1282.7	210	219.9	2.8	1.0	2.0	170	219.6
CN Rail bridge	5.9	1090.2	210	219.6	2.8	1.0	2.0	170	219.4
Bridge at Ramara Road 47	6.6	405.9	210	219.2	2.9	1.0	1.5	170	219.1

[/1/] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

TABLE 5.2-12
RESULTS FROM SIMULATION OF A BREACH OF PORTAGE LOCK 39 DURING THE FLOOD 2/3 OF THE WAY BETWEEN THE 1,000-YR FLOOD AND THE PMF

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[1/] (h)	Wave Arrival Time[1/] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	280	235.7		NA	0.4	270	235.7
Upstream of Talbot Dam	0.0	11008.4	300	235.7		NA	0.4	240	235.7
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	300	231.3	0.0	NA	1.4	300	231.2
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	290	231.3	0.0	NA	0.7	280	231.3
0.5 km downstream of Lock 38	0.5	10500.7	290	231.3	0.0	NA	0.3	280	231.3
Upstream of Talbot River Dam	1.1	9864.4	280	231.3	0.0	NA	0.3	280	231.3
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	280	228.4	0.1	0.2	2.3	280	228.4
0.5 km from Talbot River Dam	1.6	9366.4	280	228.0	0.1	0.2	2.4	280	228.0
0.9 km from Talbot River Dam	2.0	9031.8	280	227.9	0.1	0.2	1.7	280	227.9
2.2 km from Talbot River Dam	3.3	7683.9	280	225.2	0.3	0.3	1.9	280	225.2
3.9 km from Talbot River Dam	5.0	5977.5	280	224.7	0.4	0.5	2.1	280	224.7
5.2 km from Talbot River Dam	6.3	4723.1	280	223.7	0.5	0.5	2.0	280	223.7
Gamebridge near Hazel St.	7.0	4007.5	280	223.3	0.5	0.6	2.2	280	223.3
Gamebridge bridge at Ramara Road 51	7.3	3651.0	280	223.0	0.6	0.6	2.1	280	223.0
Bridge at Hwy 12	8.6	2442.6	280	221.8	0.7	0.8	2.3	280	221.8
Near Furniss Drive	10.0	1010.1	270	220.8	1.0	1.0	1.8	280	220.8
Talbot River near confluence with canal	10.9	62.8	280	219.8	0.6	0.2	2.4	280	219.6
Trent Talbot Marina	5.9	1084.2	140	219.6	0.6	0.2	1.0	110	219.4
CN Rail bridge	6.3	652.1	140	219.3	0.7	0.2	1.3	110	219.2
Bridge at Ramara Road 47	6.8	156.9	140	219.2	1.3	0.2	1.2	110	219.1
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	30	231.2	0.0	0.0	0.1	10	231.2
Upstream of Portage Lock 39	2.4	4567.0	50	231.2	0.0	0.0	0.2	10	231.2
Downstream of Portage Lock 39	2.5	4528.0	50	227.7	0.3	0.0	0.3	10	227.1
Upstream of Lock 40	3.2	3797.0	60	227.4	0.2	0.0	0.3	10	227.1
Downstream of Lock 40	3.2	3758.0	60	223.4	0.4	0.0	0.2	10	222.8
Upstream of Lock 41	4.3	2712.0	70	223.2	0.3	0.0	0.2	10	222.8
Downstream of Lock 41	4.3	2673.0	70	220.0	0.5	0.0	0.2	20	219.5
Confluence with the Talbot River	5.7	1282.7	190	219.8	0.6	0.2	2.0	170	219.6
CN Rail bridge	5.9	1090.2	190	219.5	0.6	0.2	2.0	170	219.4
Bridge at Ramara Road 47	6.6	405.9	190	219.1	0.6	0.2	1.5	170	219.1

[1/] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

TABLE 5.2-13
RESULTS FROM SIMULATION OF A BREACH OF THE SLUICEWAY STRUCTURE AT THE TALBOT RIVER DAM DURING THE PMF

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[1/] (h)	Wave Arrival Time[1/] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	350	235.8		NA	0.5	350	235.8
Upstream of Talbot Dam	0.0	11008.4	350	235.8		NA	0.5	350	235.8
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	350	232.0	0.0	NA	1.3	350	232.0
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	420	232.0	0.0	NA	0.7	350	232.0
0.5 km downstream of Lock 38	0.5	10500.7	490	232.0	0.0	NA	0.3	350	232.0
Upstream of Talbot River Dam	1.1	9864.4	900	232.0	0.0	NA	0.3	300	232.0
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	900	229.8	0.3	0.0	2.4	300	228.5
0.5 km from Talbot River Dam	1.6	9366.4	800	229.0	0.3	0.0	2.5	300	228.1
0.9 km from Talbot River Dam	2.0	9031.8	730	229.0	0.3	0.0	1.8	300	228.0
2.2 km from Talbot River Dam	3.3	7683.9	660	226.2	0.7	0.0	1.9	300	225.4
3.9 km from Talbot River Dam	5.0	5977.5	490	225.7	1.0	0.2	2.2	300	224.8
5.2 km from Talbot River Dam	6.3	4723.1	470	224.4	1.1	0.2	2.1	300	223.8
Gamebridge near Hazel St.	7.0	4007.5	460	223.7	1.2	0.3	2.3	300	223.3
Gamebridge bridge at Ramara Road 51	7.3	3651.0	460	223.6	1.2	0.3	2.1	300	223.1
Bridge at Hwy 12	8.6	2442.6	440	222.2	1.6	0.4	2.4	300	221.9
Near Furniss Drive	10.0	1010.1	420	221.2	2.3	0.5	1.7	300	220.9
Talbot River near confluence with canal	10.9	62.8	410	220.1	2.5	0.8	2.3	300	219.8
Trent Talbot Marina	5.9	1084.2	170	219.8	2.6	1.0	1.2	140	219.7
CN Rail bridge	6.3	652.1	170	219.4	2.8	1.0	1.6	140	219.4
Bridge at Ramara Road 47	6.8	156.9	170	219.2	3.4	1.2	1.5	140	219.2
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	50	231.9	0.0	0.0	0.6	50	231.9
Upstream of Portage Lock 39	2.4	4567.0	50	231.6	0.0	0.0	1.3	50	231.6
Downstream of Portage Lock 39	2.5	4528.0	50	227.6	0.0	0.0	1.6	50	227.6
Upstream of Lock 40	3.2	3797.0	50	227.4	0.0	0.0	1.4	50	227.4
Downstream of Lock 40	3.2	3758.0	50	223.4	0.0	0.0	1.5	50	223.4
Upstream of Lock 41	4.3	2712.0	70	223.2	0.0	0.4	1.3	70	223.2
Downstream of Lock 41	4.3	2673.0	70	219.9	2.5	0.8	2.1	70	219.9
Confluence with the Talbot River	5.7	1282.7	250	220.1	2.5	0.8	2.1	210	219.8
CN Rail bridge	5.9	1090.2	250	219.8	2.5	0.8	2.3	210	219.6
Bridge at Ramara Road 47	6.6	405.9	250	219.2	2.6	0.8	1.8	210	219.1

[1/] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

TABLE 5.2-14
RESULTS FROM SIMULATION OF A BREACH OF THE EARTH EMBANKMENT AT THE TALBOT RIVER DAM DURING THE FLOOD PMF

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[1/] (h)	Wave Arrival Time[1/] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	350	235.8		NA	0.5	350	235.8
Upstream of Talbot Dam	0.0	11008.4	350	235.8		NA	0.5	350	235.8
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	350	231.3	0.0	NA	1.7	350	231.2
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	380	231.3	0.0	NA	0.9	350	231.2
0.5 km downstream of Lock 38	0.5	10500.7	410	231.3	0.0	NA	0.3	350	231.3
Upstream of Talbot River Dam	1.1	9864.4	590	231.3	0.0	NA	0.3	350	231.3
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	590	229.4	1.0	0.1	2.6	350	228.7
0.5 km from Talbot River Dam	1.6	9366.4	580	228.7	1.0	0.1	2.7	350	228.2
0.9 km from Talbot River Dam	2.0	9031.8	570	228.7	1.0	0.1	1.4	350	228.1
2.2 km from Talbot River Dam	3.3	7683.9	550	226.1	1.1	0.2	2.0	340	225.6
3.9 km from Talbot River Dam	5.0	5977.5	470	225.6	1.4	0.3	2.3	340	225.1
5.2 km from Talbot River Dam	6.3	4723.1	460	224.3	1.5	0.4	2.2	340	224.0
Gamebridge near Hazel St.	7.0	4007.5	450	223.7	1.6	0.5	2.5	340	223.5
Gamebridge bridge at Ramara Road 51	7.3	3651.0	450	223.5	1.6	0.6	2.2	340	223.3
Bridge at Hwy 12	8.6	2442.6	430	222.2	2.0	0.8	2.6	340	222.0
Near Furniss Drive	10.0	1010.1	420	221.2	2.6	0.9	1.7	340	221.0
Talbot River near confluence with canal	10.9	62.8	410	220.1	2.8	1.0	2.6	340	219.9
Trent Talbot Marina	5.9	1084.2	170	219.8	2.9	1.0	1.2	140	219.6
CN Rail bridge	6.3	652.1	170	219.4	3.1	1.1	1.6	140	219.3
Bridge at Ramara Road 47	6.8	156.9	170	219.2	3.6	1.2	1.4	140	219.2
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	10	231.2	0.0	0.0	0.1	10	231.2
Upstream of Portage Lock 39	2.4	4567.0	10	231.2	0.0	0.0	0.2	10	231.2
Downstream of Portage Lock 39	2.5	4528.0	10	227.1	0.1	0.1	0.3	10	227.1
Upstream of Lock 40	3.2	3797.0	10	227.1	0.1	0.1	0.3	10	227.1
Downstream of Lock 40	3.2	3758.0	10	222.8	0.2	0.2	0.2	10	222.8
Upstream of Lock 41	4.3	2712.0	10	222.8	0.3	0.3	0.2	10	222.8
Downstream of Lock 41	4.3	2673.0	10	219.9	2.8	1.0	0.2	10	219.8
Confluence with the Talbot River	5.7	1282.7	240	220.1	2.8	1.0	2.1	210	219.9
CN Rail bridge	5.9	1090.2	240	219.8	2.8	1.0	2.3	210	219.6
Bridge at Ramara Road 47	6.6	405.9	240	219.2	2.9	1.1	1.8	210	219.2

[1/] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

TABLE 5.2-15
RESULTS FROM SIMULATION OF A BREACH OF PORTAGE LOCK 39 DURING THE PMF

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[/1/] (h)	Wave Arrival Time[/1/] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	350	235.8		NA	0.5	320	235.8
Upstream of Talbot Dam	0.0	11008.4	350	235.8		NA	0.5	320	235.8
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	350	231.3	0.0	NA	1.7	350	231.2
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	350	231.3	0.0	NA	0.9	350	231.2
0.5 km downstream of Lock 38	0.5	10500.7	350	231.3	0.0	NA	0.3	350	231.3
Upstream of Talbot River Dam	1.1	9864.4	350	231.3	0.0	NA	0.3	350	231.3
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	350	228.7	0.1	0.2	2.6	350	228.7
0.5 km from Talbot River Dam	1.6	9366.4	350	228.2	0.1	0.2	2.7	350	228.2
0.9 km from Talbot River Dam	2.0	9031.8	350	228.1	0.1	0.2	1.4	350	228.1
2.2 km from Talbot River Dam	3.3	7683.9	340	225.6	0.2	0.3	2.0	340	225.6
3.9 km from Talbot River Dam	5.0	5977.5	340	225.1	0.4	0.4	2.3	340	225.1
5.2 km from Talbot River Dam	6.3	4723.1	340	224.0	0.4	0.5	2.2	340	224.0
Gamebridge near Hazel St.	7.0	4007.5	340	223.5	0.5	0.7	2.5	340	223.5
Gamebridge bridge at Ramara Road 51	7.3	3651.0	340	223.3	0.5	0.7	2.2	340	223.3
Bridge at Hwy 12	8.6	2442.6	340	222.0	0.7	0.8	2.6	340	222.0
Near Furniss Drive	10.0	1010.1	340	221.0	1.1	0.5	1.7	340	221.0
Talbot River near confluence with canal	10.9	62.8	340	220.0	0.6	0.2	2.6	340	219.9
Trent Talbot Marina	5.9	1084.2	170	219.8	0.6	0.2	1.2	140	219.6
CN Rail bridge	6.3	652.1	170	219.4	0.6	0.2	1.6	140	219.3
Bridge at Ramara Road 47	6.8	156.9	170	219.2	1.7	0.2	1.4	140	219.2
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	30	231.2	0.0	0.0	0.1	10	231.2
Upstream of Portage Lock 39	2.4	4567.0	50	231.2	0.0	0.0	0.2	10	231.2
Downstream of Portage Lock 39	2.5	4528.0	50	227.7	0.3	0.0	0.3	10	227.1
Upstream of Lock 40	3.2	3797.0	50	227.4	0.2	0.0	0.3	10	227.1
Downstream of Lock 40	3.2	3758.0	50	223.4	0.4	0.0	0.2	10	222.8
Upstream of Lock 41	4.3	2712.0	70	223.2	0.3	0.0	0.2	10	222.8
Downstream of Lock 41	4.3	2673.0	70	220.1	0.5	0.0	0.2	20	219.8
Confluence with the Talbot River	5.7	1282.7	230	220.0	0.6	0.2	2.1	210	219.9
CN Rail bridge	5.9	1090.2	230	219.7	0.6	0.2	2.3	210	219.6
Bridge at Ramara Road 47	6.6	405.9	230	219.2	0.6	0.2	1.8	210	219.2

[/1/] Estimated times are relative to the time of initiation of the breach of the Talbot Dam for the dam breach case.

5.2.3 Dam Classification Based On Dam Break Analysis

The bases for dam hazard classification are the incremental consequences resulting from a dam failure. These are the damages that the dam breach would cause, above and beyond those resulting from the underlying natural conditions (i.e. if the dam doesn't break), and are evaluated in terms of the following categories:

- Incremental loss of life,
- Incremental economic losses,
- Incremental environmental losses, and
- Incremental cultural losses.

Based on these incremental consequences, dams are classified separately for a breach occurring in normal conditions (sunny-day) and during a flood. Only the latter is used to determine the Inflow Design Flood (IDF), with the former being used to determine seismic requirements for structural considerations. The higher of the two classifications is adopted as the governing case and used to determine the frequency of inspections and dam safety reviews as well as the requirements for emergency preparedness.

The assessments of incremental dam breach consequences provided in this report should not be considered as exact calculations but rather reasonable estimates based on industry guidelines and generally adopted methodologies, as well as engineering judgment. The purpose of these estimates is to compare the consequences of dam failure with the thresholds indicated in the dam classification guidelines applied in this study.

5.2.4 Incremental Loss of Life (ILOL)

There is no precise science to predict the loss of life (LOL) resulting from a dam break. There are, however, reasonable bases for estimation using experiences from actual dam breach cases. One such method, used in this study for LOL estimation, is that developed by W.J. Graham at the United Sates Bureau of Reclamation (USBR) and described in the document "A Procedure for Estimating Loss of Life Caused by Dam Failure" (DSP-99-06) (Ref. 17). This method has become of widespread use in North America for the analysis of incremental consequences resulting from a dam break. It allows estimation of loss of life based on the following factors:

- The number of people occupying the area that could be inundated, referred to as the population at risk (PAR). The PAR includes the total population within the flooded area, regardless of the depth of flooding.
- The amount of warning that can be provided to the people exposed to dangerous flooding.
- The severity (primarily depth and velocity) of the flooding.
- The understanding by the people warning the PAR of the nature of dam breach floods and their knowledge of escape routes.

The latter three factors are used in the method to select a "fatality rate", which is the fraction of the PAR that is estimated to perish.

The area affected by a breach of the Talbot River Dam or Lock 39 includes populated areas immediately downstream of the dam as well as in the Village of Gamebridge, ON, and at the discharge of the Talbot River and Talbot Canal at Lake Simcoe. The distance from the dam to Lake Simcoe is less than 10 km and the dam break flood wave would travel through this area in less than 1 hour. There would be little or no time to warn this population following a dam break.

The type of buildings in the area is predominantly residential, so the time of the day when a dam breach could affect the largest amount of people would be during the night when most of them are at their residences. It was therefore considered conservative to base the calculation of the PAR on a night scenario. The PAR estimate was based on the number of dwellings within the flooded area, obtained by superimposing the dam break model results on the database of buildings, and using the average population for the Township of Brock. According to the data obtained from the 2006 Census, it corresponds to 2.7 people per house. This rate of occupancy was assumed for all residential buildings.

There is a camping site downstream of the Talbot River Dam. Four buildings and one trailer were identified there at the time of the site visit. The site could accommodate additional trailers and tents during camping season. A transient population of 30 people was assumed for this site for a sunny day breach of the Talbot River Dam. To account for other possible recreational users of the waterway, 4 additional people were considered for a total transient population of 34. In a breach of Lock 39 the transient PAR was assumed to be 4 people, consisting on possible users of the canal. The transient population was only included in the ILOL estimation in the case of a sunny-day dam breach. It was considered that during an extreme flood, transient population would not occupy the waterway or the areas susceptible to flooding.

The total population at risk (PAR) was estimated assuming no pre-emptive evacuation of permanent buildings. The total PAR for the scenario of a dam breach during the night is shown in Table 5.2-16 for the Talbot River Dam and Table 5.2-17 for Lock 39. The numbers have been rounded to the nearest higher integer.

TABLE 5.2-16
POPULATION AT RISK (PAR) BREACH OF TALBOT RIVER DAM

Flood Condition		No Dam Breach	Dam Breach
Sunny Day	Permanent PAR	0	62
	Transient PAR	0	34
	Total PAR	**0**	**96**
1/3 between 1,000-year flood and PMF	Permanent PAR	216	286
	Transient PAR	0	0
	Total PAR	**216**	**286**
2/3 between 1,000-year flood and PMF	Permanent PAR	251	311
	Transient PAR	0	0
	Total PAR	**251**	**311**
PMF	Permanent PAR	284	324
	Transient PAR	0	0
	Total PAR	**284**	**324**

TABLE 5.2-17
POPULATION AT RISK (PAR) BREACH OF LOCK 39

Flood Condition		No Dam Breach	Dam Breach
Sunny Day	Permanent PAR	0	78
	Transient PAR	0	4
	Total PAR	**0**	**82**
1/3 between 1,000-year flood and PMF	Permanent PAR	200	213
	Transient PAR	0	0
	Total PAR	**200**	**213**
2/3 between 1,000-year flood and PMF	Permanent PAR	240	248
	Transient PAR	0	0
	Total PAR	**240**	**248**
PMF	Permanent PAR	265	273
	Transient PAR	0	0
	Total PAR	**265**	**273**

To estimate the amount of warning that the population would receive following a dam failure, the time of arrival of the dam-break wave was obtained from the model results for each location downstream of the dam. This wave arrival time is shown in Tables 5.2-4 to 5.2-15. Since there is no permanent staff at the dam site during the night, it was assumed that there would be some intervening time between the dam breach and the release of the warning. The methodology in USBR (Ref. 17) recommends, for this situation, to assume that the warning would be initiated 1 hour, for a night scenario, after the arrival of the dam-break wave to populated areas. In the

case of the Talbot River Dam and Lock 39 there are residences in the vicinity of the dam. It was assumed that the warning would be initiated 1 hour after the dam breach. Since in most cases the dam-break wave would take less than 1 hour to travel through the entire flooded area, this assumption corresponds to no warning for the PAR.

The USBR methodology (Ref. 17) recommends adopting “MEDIUM” severity when the depth of flooding exceeds 3.3 m or the parameter “DV”, which is a combination of depth and velocity, is 4.6 m^2/s or higher. “MEDIUM” flood severity is defined in USBR (Ref. 17) as a condition in which “homes are destroyed but trees or mangled homes remain for people to seek refuge in or on”. “LOW” flow severity is defined as conditions where “no buildings are washed off their foundations”. A value of flood severity was assigned at each building and for each dam breach scenario, based on the depth of flow and the parameter “DV” calculated from the model results. These were “LOW” for all the houses with the exception of a narrow area within 0.5 km of the Talbot River Dam.

The methodology described in USBR (Ref. 17) requires adopting a value for the “flood severity understanding” that relates to the knowledge of the situation that the warning issuers (dam owner or local authorities) have and can successfully convey to the PAR. In this case, and because the warning time was zero, this parameter was not used.

The LOL, in accordance with the USBR Methodology (Ref. 17) consists of estimating the PAR and applying to it a “fatality rate”. This rate is in turn selected based on the parameters previously discussed: flood severity, flood severity understanding and warning time from a table of recommended values that were derived from a database of dam break historical cases. This table, from Ref. 17 is shown in Table 5.2-18.

TABLE 5.2-18
RECOMMENDED FATALITY RATES FROM REF. 17

Recommended Fatality Rates for Estimating Loss of Life Resulting from Dam Failure				
Flood Severity	Warning Time (minutes)	Flood Severity Understanding	Fatality Rate (Fraction of people at risk expected to die)	
			Suggested	Suggested Range
HIGH	no warning	not applicable	0.75	0.3 to 1.00
	15 to 60	Vague	Use the values shown above and apply to the number of people who remain in the dam failure floodplain after warnings are issued. No guidance is provided on how many people will remain in the floodplain.	
	more than 60	Precise		
		Vague		
		Precise		
MEDIUM	no warning	not applicable	0.15	0.03 to 0.35
	15 to 60	Vague	0.04	0.01 to 0.08
		Precise	0.02	0.005 to 0.04
	more than 60	Vague	0.03	0.005 to 0.06
		Precise	0.01	0.002 to 0.02
LOW	no warning	not applicable	0.01	0.0 to 0.02
	15 to 60	Vague	0.007	0.0 to 0.015
		Precise	0.002	0.0 to 0.004
	more than 60	Vague	0.0003	0.0 to 0.0006
		Precise	0.0002	0.0 to 0.0004

The guidelines indicated in USBR (Ref. 17) were used to obtain the LOL corresponding to dam break cases and for areas that would be impacted directly from the dam break flood wave. The fatality rates that were used were in the range of 0.01 to 0.15 and reflected the little or no warning time that would be available after a breach of either the Talbot River Dam or Portage Lock 39. There were some areas in which the model results did not show a direct impact of the dam breach wave; but that would be flooded from overtopping of roads into adjacent lower areas. This would be the case in the areas east of Talbot Rd and south of Regional Rd 50. The flooding in this case would be more gradual and similar to a natural event. These areas were given a fatality rate similar to that used for the estimation of LOL in a no-dam-break case.

For the no-dam break cases a fatality rate of 0.001 was adopted, This is much lower than the values proposed in Ref. 17 for dam break conditions, and it is considered appropriate given that the rise of water levels during the natural flood would be relatively gradual and would take days, as opposed to a dam break flood wave, which would travel across the river reach in approximately one hour.

The USBR methodology does not specifically indicate how to determine the incremental loss of life (ILOL), which is the LOL directly attributable to the dam breach, discounting the LOL due to the natural flood. Different approaches have been used for this. In this study, the following three options were investigated:

1. Discounting the portion of the PAR that is affected by the natural flood, and calculating the ILOL by applying the dam-break fatality rates to the remaining (incremental) PAR. This option is commonly used in the engineering industry due to its simplicity. However, it could be viewed as non-conservative because the portion of the PAR that is discounted from the calculation would still be affected by the dam break flood wave and likely with more severity than the natural flood.
2. Calculating separately the LOL for the dam-breach and the no-dam-breach cases, using for each case the corresponding values of PAR and fatality rates, and then calculating the ILOL as the difference between the two LOL estimations (with and without a dam breach). This option could lead to overestimating the ILOL because it does not take into account that part of the population could be flooded long before the dam breach occurs, which could lead to evacuation.
3. Adjusting Option 2 by subtracting a portion of the population flooded prior to the dam failure. This portion of the PAR is assumed to be evacuated by the time the dam breach flood reaches the area, because the conditions would be such that either they would leave or the authorities would force an evacuation. For the areas flooded downstream of the Talbot River Dam, it was considered reasonable that those flooded by 0.5 m or more by the time of the dam breach would be evacuated. A similar methodology is applied in some jurisdictions in Australia[1]. The approach was considered reasonable for the case of the Talbot River Dam and Portage Lock 39, since there would be a slow rise of the water levels during the natural flood that would precede the dam breach and would allow the evacuation of the flooded population.

[1] The document “Consequence Categories for Dams” (DSC 3A) of 2012 by the New South Wales Dam Safety Committee (DSC), which discusses application of the guidelines issued in 2012 by the Australian National Committee on Large Dams (ANCOLD), indicates a method estimating the ILOL by subtracting from the PAR the population in all buildings flooded before the dam breach occurs. The adjusted PAR is termed “Dambreak PAR”.

The resulting ILOL values are shown in the following tables:

- Table 5.2-19 for a sunny-day breach of the Talbot River Dam.
- Table 5.2-20 for a breach of the Talbot River Dam during the flood 1/3 of the way between the 1000-year flood and the PMF.
- Table 5.2-21 for a breach of the Talbot River Dam during the flood 2/3 of the way between the 1000-year flood and the PMF.
- Table 5.2-22 for a breach of the Talbot River Dam during the PMF.

The numbers in those tables have not been rounded so the values can be used as an index for comparison among scenarios. It must be noted that the concept of a loss of a fraction of a life is entirely theoretical. The tables were prepared based on the results of the simulated dam breach of the concrete structures. The results for a breach of the earth embankment were very similar and are discussed in the following paragraphs.

TABLE 5.2-19
INCREMENTAL LOSS OF LIFE FOR A SUNNY-DAY BREACH OF THE TALBOT RIVER DAM

Option	PAR - Natural Flood + Dam Break	PAR Natural Flood Only	PAR Evacuated for Dam Break	Incremental PAR / Dam Break PAR	LOL dam-break	LOL no-dam-break	ILOL	Notes
1	96	0	N/A	96	N/A	N/A	1.5	ILOL calculated based on the Incremental PAR
2	96	0	N/A	N/A	1.5	0.0	1.5	ILOL calculated as the difference between the LOL obtained with a dam break and the LOL without a dam break
3	96	0	0	96	1.5	0.0	1.5	Assumes evacuation of PAR flooded by 0.5 m or more at the time of the dam break.

TABLE 5.2-20
INCREMENTAL LOSS OF LIFE FOR A BREACH OF THE TALBOT RIVER DAM DURING THE FLOOD 1/3 OF THE WAY BETWEEN THE 1000-YEAR FLOOD AND THE PMF

Option	PAR - Natural Flood + Dam Break	PAR Natural Flood Only	PAR Evacuated for Dam Break	Incremental PAR / Dam Break PAR	LOL dam-break	LOL no-dam-break	ILOL	Notes
1	286	216	N/A	70	N/A	N/A	0.7	ILOL calculated based on the Incremental PAR
2	286	216	N/A	N/A	1.9	0.2	1.7	ILOL calculated as the difference between the LOL obtained with a dam break and the LOL without a dam break
3	286	216	30	256	1.7	0.2	1.5	Assumes evacuation of PAR flooded by 0.5 m or more at the time of the dam break.

TABLE 5.2-21
INCREMENTAL LOSS OF LIFE FOR A BREACH OF THE TALBOT RIVER DAM DURING THE FLOOD 2/3 OF THE WAY BETWEEN THE 1000-YEAR FLOOD AND THE PMF

Option	PAR - Natural Flood + Dam Break	PAR Natural Flood Only	PAR Evacuated for Dam Break	Incremental PAR / Dam Break PAR	LOL dam-break	LOL no-dam-break	ILOL	Notes
1	311	251	N/A	60	N/A	N/A	0.6	ILOL calculated based on the Incremental PAR
2	311	251	N/A	N/A	2.2	0.2	2.0	ILOL calculated as the difference between the LOL obtained with a dam break and the LOL without a dam break
3	311	251	43	268	1.8	0.2	1.6	Assumes evacuation of PAR flooded by 0.5 m or more at the time of the dam break.

TABLE 5.2-22
INCREMENTAL LOSS OF LIFE FOR A BREACH OF THE TALBOT RIVER DAM DURING THE PMF

Option	PAR - Natural Flood + Dam Break	PAR Natural Flood Only	PAR Evacuated for Dam Break	Incremental PAR / Dam Break PAR	LOL dam-break	LOL no-dam-break	ILOL	Notes
1	324	284	N/A	40	N/A	N/A	0.8	ILOL calculated based on the Incremental PAR
2	324	284	N/A	N/A	2.7	0.3	2.4	ILOL calculated as the difference between the LOL obtained with a dam break and the LOL without a dam break
3	324	284	130	194	2.1	0.3	1.8	Assumes evacuation of PAR flooded by 0.5 m or more at the time of the dam break.

The estimated ILOL shown in Tables 5.2-19 to 5.2-22 is in all cases in the range between 1 and 10. The ILOL value obtained for a breach during sunny-day was 1.5. The maximum ILOL value obtained with Option 3 for a breach during a flood is 1.8. These values were rounded up to recognize that the concept of the loss of a portion of a life is purely academic. Consequently, an ILOL of 2 was adopted as the basis for the selection of the IDF.

The ILOL values obtained for a breach of the earth embankment at the Talbot River Dam were very similar to those obtained for a breach of the concrete structure. For instance, with Method 3, the results were:

- 1.5 ILOL for a sunny-day breach.
- 1.2 ILOL or a breach during the PMF.

For the purpose of classifying the dam, the results obtained assuming a breach of the concrete structure were used because they were slightly higher.

Similarly to the analysis done for the Talbot River Dam, the ILOL values were estimated for a breach of Portage Lock 39. The results are shown in the following tables:

- Table 5.2-23 for a sunny-day breach of the Lock 39.
- Table 5.2-24 for a breach of the Lock 39 during the flood 1/3 of the way between the 1000-year flood and the PMF.
- Table 5.2-25 for a breach of the Lock 39 during the flood 2/3 of the way between the 1000-year flood and the PMF.
- Table 5.2-26 for a breach of the Lock 39 during the PMF.

TABLE 5.2-23
INCREMENTAL LOSS OF LIFE FOR A SUNNY-DAY BREACH OF LOCK 39

Option	PAR - Natural Flood + Dam Break	PAR Natural Flood Only	PAR Evacuated for Dam Break	Incremental PAR / Dam Break PAR	LOL dam-break	LOL no-dam-break	ILOL	Notes
1	82	0	N/A	82	N/A	N/A	0.9	ILOL calculated based on the Incremental PAR
2	82	0	N/A	N/A	0.9	0.0	0.9	ILOL calculated as the difference between the LOL obtained with a dam break and the LOL without a dam break
3	82	0	0	82	0.9	0.0	0.9	Assumes evacuation of PAR flooded by 0.5 m or more at the time of the dam break.

TABLE 5.2-24
INCREMENTAL LOSS OF LIFE FOR A BREACH OF LOCK 39 DURING THE FLOOD 1/3 OF THE WAY BETWEEN THE 1000-YEAR FLOOD AND THE PMF

Option	PAR - Natural Flood + Dam Break	PAR Natural Flood Only	PAR Evacuated for Dam Break	Incremental PAR / Dam Break PAR	LOL dam-break	LOL no-dam-break	ILOL	Notes
1	213	200	N/A	13	N/A	N/A	0.1	ILOL calculated based on the Incremental PAR
2	213	200	N/A	N/A	1.3	0.2	1.1	ILOL calculated as the difference between the LOL obtained with a dam break and the LOL without a dam break
3	213	200	35	178	1.2	0.2	1.0	Assumes evacuation of PAR flooded by 0.5 m or more at the time of the dam break.

TABLE 5.2-25
INCREMENTAL LOSS OF LIFE FOR A BREACH OF LOCK 39 DURING THE FLOOD 2/3 OF THE WAY BETWEEN THE 1000-YEAR FLOOD AND THE PMF

Option	PAR - Natural Flood + Dam Break	PAR Natural Flood Only	PAR Evacuated for Dam Break	Incremental PAR / Dam Break PAR	LOL dam-break	LOL no-dam-break	ILOL	Notes
1	248	240	N/A	8	N/A	N/A	0.1	ILOL calculated based on the Incremental PAR
2	248	240	N/A	N/A	1.5	0.2	1.3	ILOL calculated as the difference between the LOL obtained with a dam break and the LOL without a dam break
3	248	240	65	183	1.1	0.2	0.9	Assumes evacuation of PAR flooded by 0.5 m or more at the time of the dam break.

TABLE 5.2-26
INCREMENTAL LOSS OF LIFE FOR A BREACH OF LOCK 39 DURING THE PMF

Option	PAR - Natural Flood + Dam Break	PAR Natural Flood Only	PAR Evacuated for Dam Break	Incremental PAR / Dam Break PAR	LOL dam-break	LOL no-dam-break	ILOL	Notes
1	273	265	N/A	8	N/A	N/A	0.1	ILOL calculated based on the Incremental PAR
2	273	265	N/A	N/A	1.6	0.3	1.3	ILOL calculated as the difference between the LOL obtained with a dam break and the LOL without a dam break
3	273	265	146	127	1.1	0.3	0.8	Assumes evacuation of PAR flooded by 0.5 m or more at the time of the dam break.

The estimated ILOL shown in Tables 5.2-23 to 5.2-26 is in all cases approximately 1.

Recognizing that the LOL estimation is not exact, the methodology indicated in Ref. 17 includes an evaluation of the possible uncertainties in the calculations. The following considerations were made in that regard:

- There is an element of uncertainty that is inherent to the numerical modelling used as a basis to estimate the ILOL. The preparation of the model included use of several data sources and adoption of a number of parameters. The data was verified to the extent possible and it was found consistent. The assumptions made in the analysis were conservative but reasonable.
- There is uncertainty in the determination of the dam breach size and development time. The selected breach width is the maximum recommended in the guidelines that were applied (Ref. 15) and is considered reasonably conservative. Adopting a smaller dam breach size would still result in an ILOL of 1 or more for these structures. Furthermore, the analysis considered either a breach at the earth embankment or the concrete structure at the Talbot River Dam site, with very similar results in both cases.
- There is uncertainty in the determination of the PAR. The analysis was based on the average population in the flooded area, according to Census data, and building information that was verified to the extent practical. The assumptions made for population are considered adequate for the analysis.
- There is uncertainty with respect to the time of the day at which the dam breach could occur. The ILOL estimations were in all cases in the range of 1 to 10, and were made assuming a dam breach occurring at night. An estimate for a day scenario would most likely require reducing the PAR. However, the resulting ILOL would still be in the range of 1 to 10.

- There is uncertainty in the ability to precisely determine the fatality rate for a dam break. The suggested fatality rate values, provided in USBR (Ref. 17), were used to develop tables 5.2-19 to 5.2-26. In addition to suggested values, the reference also provides a suggested range that can be used for testing sensitivity of the results. A review of the ILOL estimates for a dam break at the Talbot River Dam during the PMF was carried out using the minimum and maximum values in that range. With the minimum fatality rate, using Option 3 as defined previously, the resulting ILOL was almost zero. With the maximum fatality rate and Option 3, the resulting ILOL was 3.8. In the case of the Talbot River Dam, and also Lock 39, there are no factors that suggest using either the minimum or the maximum values in the range of fatality rates. The suggested value, which is near the middle of the range, is considered adequate. However, the ILOL values obtained with minimum and maximum fatality rates are still in the range of less than 10.

The results obtained in tables 5.2-19 to 5.2-26 are considered conservative but reasonable, and provide a range of ILOL (between 1 and 10) that is consistent through the analysis of uncertainties. The proposed value to be used for IDF selection is an ILOL of 2, rounding up the maximum obtained for a breach of the Talbot River Dam during an extreme flood. The ILOL attributable to a breach of Lock 39 is approximately 1.

A breach of the North of South dykes along the Talbot Canal from the forebay of the Talbot River Dam to Portage Lock 39 was not specifically modelled to estimate incremental consequences. It was observed in the site visit and from aerial photographs that there are houses within 20 m of the South Dykes at least at 2 locations. The North Dyke also has 2 locations with nearby inhabitable buildings, in this case within 30 m of the dyke. A breach of these dykes at any of these locations would directly impact the houses due to their proximity, with little intervening time and with flooding conditions that would correspond to LOW flood severity, as described Ref. 17. Therefore, ILOL resulting from a breach of these dykes cannot be fully discarded. The impact of such a breach would be reduced as the flood wave travels away from the dykes along the floodplain, were other buildings are located. Due to the limited number of buildings directly impacted, an ILOL of 1 has been adopted for a breach of either the North Dyke or the South Dyke.

5.2.5 Incremental Third Party Economic Losses

As other aspects of dam classification, the estimation of incremental third-party economic losses is not intended to be precise but rather to allow ranking the dam among the hazard categories. The Parks Canada Directive indicates a HIGH hazard category for dams resulting in incremental third party losses of $13.7 Million ($12 Million Dollars of the year 2008) or more.

Third party losses associated to a breach at the Talbot River Dam and Lock 39 would include:

- Damages to residential buildings and to recreational sites.
- Damages to bridges and road infrastructure.
- Other damages that are almost certain to occur but difficult to quantify (KGS Group has normally used a 20% contingency for these damages).

Single family houses are the predominant type of building in the area that could be affected by a breach of the Talbot River Dam or Lock 39. The 2006 Census data indicates for the Township of Brock an average value of dwellings of approximately $260,000. A review of available real estate listings at the time of the study indicated that this could be a conservative estimate for the area of the study, so it was adopted without any adjustments of values since 2006. The buildings at those locations where the water level was estimated to exceed the first floor elevation were assumed to incur damages of $390,000 (150% of the adopted value of dwellings) to include repairs to structures, as well as replacement of affected contents and living costs for the residents during the period of flooding. From inspections of the typical structures in the affected areas, KGS Group estimated that the average first floor elevation is approximately 1 m above the ground level.

For the camp site, downstream of the Talbot River Dam, a combined estimate was used to include the permanent buildings at the site (four cabins and a trailer were found at the time of the visit) as well as the properties that campers could bring for temporary habitation. A gross estimate of $1,000,000 for a breach of the dam in sunny-day and $500,000 for a breach during a flood (assuming no temporary campers) was applied.

At locations where the model results indicated overtopping of local roads, a repair cost of $20,000 per km was applied. This value is based on typical repair costs to overtopped roads in the Red River Valley flood of 1997 in Manitoba, inflated based on the increase in Consumer Price Index since 1997. This cost does not include full replacement of roads but rather repair of damages that would be incurred due to temporary overtopping. The model results suggest that there would be minimal overtopping of roads. There are 8 bridges between the Talbot River Dam and Lake Simcoe, including road and railway bridges. The model results indicated that only the bridge located on Regional Rd 50, immediately downstream of the Talbot River Dam, would be overtopped by the flood resulting from the breach of the Talbot River Dam. Overtopping of the bridge deck was assumed to cause irreparable damage and a replacement cost of $10,000 per square metre of bridge deck area was applied, based on bridge construction costs.

Calculation of the damages listed above, with the addition of a 20% provision for damages not accounted for, are provided in Table 5.2-27 for a sunny-day breach of the Talbot River Dam and Table 5.2-28 for a breach of the dam during the PMF. This case was selected as representative of a flood scenario because, of the three analyzed floods, it resulted in the highest incremental damages to third party property. The values corresponding to a breach of the concrete structures were selected because they are more conservative than for a breach of the earth embankment at the dam site. The total incremental third party damages for these two scenarios are less than the $13.7 Million threshold for adopting a HIGH hazard category. The third party economic losses associated to a breach of the Lock 39, for sunny day and flood conditions, would also be below the threshold for adopting a HIGH hazard category.

TABLE 5.2-27
ECONOMIC DAMAGES TO THIRD PARTIES FROM A SUNNY-DAY BREACH OF THE TALBOT RIVER DAM

DESCRIPTION		Unit	Unit Price	Amount Affected			Cost		
				No Dam Breach	Dam Breach	Incremental	No Dam Breach	Dam Breach	Incremental
Private Property	Houses	Item	$ 390,000	0	4	4	$ -	$ 1,560,000	$ 1,560,000
	Camp	Global	$ 1,000,000	0	1	1	$ -	$ 1,000,000	$ 1,000,000
Infrastructure	Road System Repairs	km	$ 20,000	0	1.0	1.0	$ -	$ 20,000	$ 20,000
	Bridges	m²	$ 10,000	0	180	180	$ -	$ 1,800,000	$ 1,800,000
SUBTOTAL									$ 4,380,000
CONTINGENCIES			20%						$ 880,000
TOTAL THIRD PARTY ECONOMIC LOSSES									**$ 5,260,000**

TABLE 5.2-28
ECONOMIC DAMAGES TO THIRD PARTIES RESULTING FROM A BREACH OF THE TALBOT RIVER DAM DURING THE PMF

DESCRIPTION		Unit	Unit Price	Amount Affected			Cost		
				No Dam Breach	Dam Breach	Incremental	No Dam Breach	Dam Breach	Incremental
Private Property	Houses	Item	$ 390,000	11	27	16	$ 4,290,000	$ 10,530,000	$ 6,240,000
	Camp	Global	$ 500,000	0	1	1	$ -	$ 500,000	$ 500,000
Infrastructure	Road System Repairs	km	$ 20,000	10	10	0.0	$ 200,000	$ 200,000	$ -
	Bridges	m²	$ 10,000	0	180	180	$ -	$ 1,800,000	$ 1,800,000
SUBTOTAL									$ 8,540,000
CONTINGENCIES			20%						$ 1,710,000
TOTAL THIRD PARTY ECONOMIC LOSSES									**$ 10,250,000**

5.2.6 Incremental Environmental Damages

A detailed analysis of the extent of the potential environmental damages due to a breach of the Talbot River Dam or Lock 39 has not been carried out. Instead, a general review, commensurate to the level of effort required for dam classification, indicates that there could be some environmental damages caused upstream and downstream of the structures, along the shorelines. These losses are considered to be relatively minor and temporary and not likely to include species identified as threatened or endangered. The incremental environmental damages would not result in a higher dam classification than the ILOL previously discussed.

5.2.7 Incremental Damages to Cultural Assets

There are no known designated cultural facilities or heritage buildings or landmarks in the area that would be affected by a breach of the Talbot River Dam or Lock 39. Incremental damages in this category would not govern the dam classification.

5.2.8 Dam Classification Based on the Parks Canada Directive

The results shown in Section 5.3.1 indicate that a breach of the Talbot River Dam both in sunny-day conditions and during a flood, results in ILOL in the range of 1 to 10. This corresponds to a dam classification of HIGH A in accordance with the PCA Directive. Other categories of incremental damages (third party economic losses, environmental or cultural) do not result in a higher hazard classification.

The results shown in Section 5.3.1 indicate that a breach of the Lock 39 both in sunny-day conditions and during a flood, results in ILOL in the range of 1 to 10. This corresponds to a dam classification of HIGH A in accordance with the PCA Directive. Other categories of incremental damages (third party economic losses, environmental or cultural) do not result in a higher hazard classification.

The analysis performed on the Talbot Canal North and South Dykes indicates that a HIGH A hazard classification is appropriate for these structures, in accordance with the Parks Canada Directive.

Based on the Parks Canada Directive, the proposed classification for the Talbot River Dam, Portage Lock 39 and the Talbot Canal Dykes is, therefore, HIGH A for both sunny-day failure and for a dam breach during a flood.

5.2.9 Dam Classification Based on CDA Dam Safety Guidelines

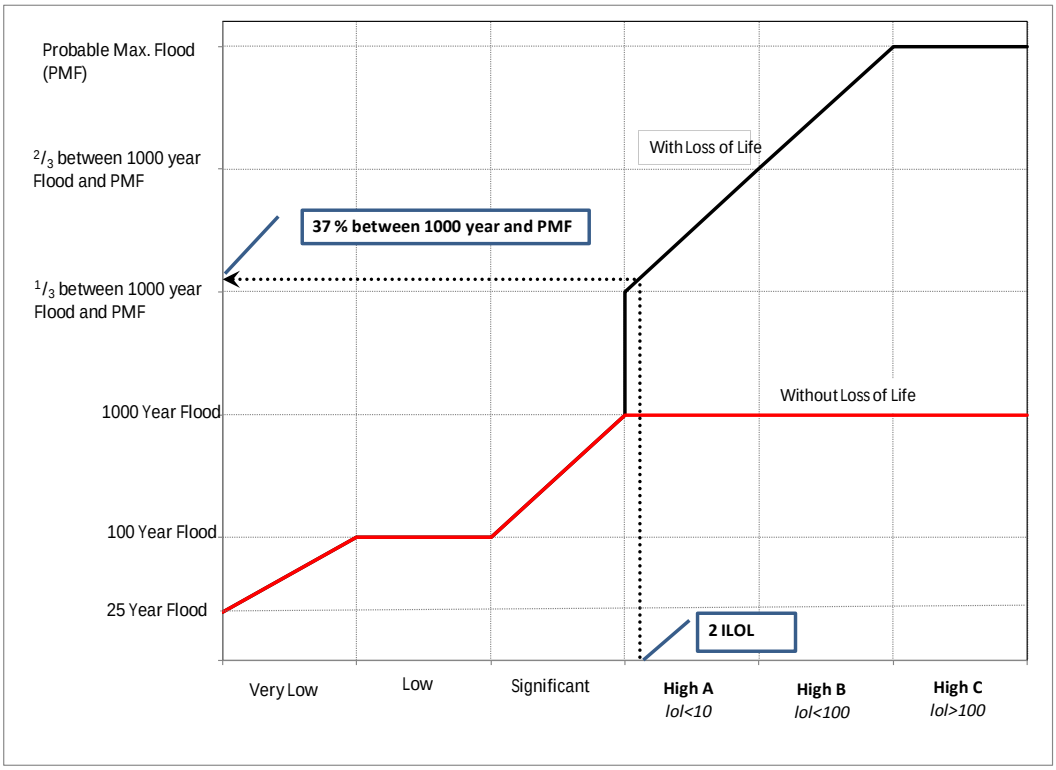
The ILOL values in the range of 1 to 10, for a dam failure in sunny-day and in flood conditions, correspond to a HIGH dam classification in accordance with the 2007 CDA Dam Safety Guidelines. Third party economic, environmental and cultural impacts of a breach of the dam or the lock would not result in a higher category.

Based on the CDA Dam Safety Guidelines, the proposed classification for the Talbot River Dam, Portage Lock 39 and the Talbot Canal Dykes is, therefore, HIGH for both sunny-day failure and for a dam breach during a flood.

5.2.10 Inflow Design Flood (IDF) Selection

Based on the results shown in Sections 5.3.1 to 5.3.4, the governing factor for IDF selection for the Talbot River Dam is the ILOL. A value of 2 was adopted for selection of the IDF. Figure 5.3.7-1 shows the results of applying this ILOL to Figure 7 of the PCA Directive for selection of the IDF. It suggests adopting an IDF that corresponds to 37% of the way between the 1000-year flood and the PMF. This is the proposed IDF for the Talbot River Dam.

FIGURE 5.3.7-1
PCA DIRECTIVE CRITERIA FOR IDF SELECTION AND APPLICATION TO THE TALBOT RIVER DAM



The ILOL obtained for a breach of the Portage Lock 39 during a flood was 1. This would result in an IDF that is lower than that obtained based on a breach of the Talbot River Dam. The latter is the proposed IDF for the site.

5.2.11 Seismic Design Parameters

The selection of the Design Basis Earthquake (DBE), as per the PCA Directive is based on the hazard classification and the estimated ILOL obtained for a dam breach in sunny-day conditions. The results of the analysis correspond to a HIGH A hazard classification (ILOL in the range of 1 to 10) for the Talbot River Dam and for Lock 39. The proposed DBE is therefore the 2,500-year earthquake. Sections 6.3 to 6.5 provide further discussion about the DBE and other loads used in the structural analysis as part of this DSR.

6.0 DAM SAFETY ANALYSIS

The analysis presented in Section 5 allowed the selection of a hazard classification for the Talbot River Dam and for Portage Lock 39 and the corresponding IDF for the site. It was based on the results of dam break simulations and the assessment of dam break incremental consequences for these structures.

In this section of the report, the analyses that supported the hydrologic inputs to the dam break simulations are presented. Also, the hydraulic characteristics of the structures (discharge capacity and available freeboard) are compared to the requirements that derive from the proposed dam classification and IDF.

Other analyses presented in this section of the report are:

- Structural stability analyses of the concrete structures.
- Stability assessment of the West Rockfill Embankment.
- Assessment of the structural adequacy of the log lifting equipment and stoplogs.
- “Other potential failure modes”.

6.1 HYDROLOGY

A hydrologic assessment of the Talbot River was included in the 2012 study by AECOM: “Severn River Watershed Hydro-Technical Study Flood Flows Estimation Study Report” (Ref. 12). In that study, PMF and flows corresponding to extreme flood events were obtained for specific locations within the Severn River Watershed to support subsequent Dam Safety Reviews. The analysis for the Talbot River was done at the location of the Talbot River Dam. The analysis was also considered to be adequate to represent the input flows at the Talbot Dam, located upstream, because:

- the two dams are very close, approximately 1 km apart, with virtually the same watershed area (1% difference);
- there are no major inflows in the reach between the two dams; and
- there is little storage in the reach between the two dams.

In this analysis, the input flows to the hydrodynamic model were applied upstream of the Talbot Dam.

6.1.1 Description of Watershed

The Talbot River flows southwest from its headwaters north of Hwy 45, between Head Lake and Norland, to Lake Simcoe where it discharges. The Talbot River watershed at the Talbot River Dam has an area of approximately 300 km2 (308.4 km2 as indicated in AECOM, 2012 (Ref 18) or 296 km2 as obtained by KGS Group from GIS analyses). It ranges in altitude from El. 301 m to El. 228 m. The majority of the watershed lies in the Canadian Shield and is characterized by thin layers of soil and many rock outcrops.

The watershed includes the Mitchell Lake sub-basin, which discharges into the Talbot River via Grass Creek, which flows are controlled at the Victoria Road Dams. Mitchell Lake also connects to the Talbot River through the Trent Canal. This reach of the canal is operated for navigation purposes and includes the Kirkfield Lift Lock.

6.1.2 PMF Assessment

The PMF for the Talbot River Dam (also applicable to the Talbot Dam) was assessed by AECOM in the study reported in Ref.12, for spring and summer conditions. Three scenarios were considered:

- Summer PMF – based on the maximum runoff generated by the summer Probable Maximum Precipitation (PMP) preceded by an antecedent rainfall corresponding to 50% of the PMP.
- Spring PMF generated by the maximum runoff from the spring PMP, the 1:100 year snow accumulation and the critical temperature sequence for the determination of the snowmelt.
- Spring PMF generated by the runoff from the Probable Maximum Snow Accumulation (PMSA), the critical temperature sequence for the determination of the snowmelt and the 1:100 year spring rainstorm.

The Spring PMF was selected as the maximum of the latter two cases.

The estimates of the PMP and PMSA were, in turn, based on the AECOM study of hydrometeorological conditions for the Severn River watershed (Ref. 11). The study applied the Generalized Regional Method to determine the PMP from depth-area-duration (D-A-D) curves developed by the Centre d'Expertise Hydrique du Quebec and by Ontario Hydro. The two data sets were combined to obtain an enveloping of maximum precipitation values that were then smoothed by AECOM. The methodology was based on the World Meteorological Organization (WMO) Publication 332 (Ref. 23). The D-A-D curves were used to define the Probable Maximum Storms (PMS) for the basins. Temporal and spatial distribution were determined using the method provided in the USACE HMR 52 (Ref. 24). The standard elliptical storm shape with a major to minor axis ratio of 2.5 was used for the PMS storm pattern.

The PMSA was obtained from the maximized season snowpack using a deterministic approach. The largest storms for each year were maximized and the total of all storms was used for the PMSA. The snowmelt was defined by a critical temperature scenario using the WMO (Ref. 23) recommendations.

The PMF was obtained by simulations with the Stream flow Synthesis and Reservoir Routing model (SSARR) developed by the USACE. The model was calibrated and verified using hydrometric data from five Water Survey of Canada (WSC) stations within the Severn River watershed (Ref. 12).

Table 6.1-1 summarizes the key assumptions and controlling parameters used in the analysis of the PMP and PMF, and compares them with those promoted by leading agencies and authorities.

TABLE 6.1-1
SUMMARY OF STANDARDS COMPARED TO PARAMETERS ADOPTED IN PREVIOUS PMP/PMF STUDIES

PMF Component	Criteria Used by Others[1]		Criteria Used by AECOM in PMF Estimation[2]
Principal Storm	All authorities use PMP, but CDA Guidelines require also verification of the PMF spring event using Probable Maximum Snow Accumulation (PMSA) combined with a 1:100 yr rainstorm		Used PMP for the summer event. For the spring event used the maximum of PMP/100-year snowpack combination and 100-year storm combined with PMSA
Antecedent Condition	CDA Guidelines & AGEFA	*In spring*: Melt of 100-yr snowpack with 1:100 yr sequence of air temperatures during and prior to the PMP	For the summer condition used an antecedent storm corresponding to 50 % of the PMP (TVA criteria). For the spring condition used the maximum of two scenarios: 100-year storm preceded by the melt of the PMSA and PMP preceded by the melt of the 100-year snowpack (CDA criteria)
	CDA Guidelines	*Alternate spring scenario*: Maximized snowpack with 1:100 yr rainstorm	
	CDA Guidelines	*In summer*: a 1:100-yr rainstorm	
	AGEFA	10 year – 48 hour storm finishes 5 days prior to the start of the PMP for summer flood event	
	ANS	40% of PMP	
	USACE & USNRCS	Standard Project Flood 5 days prior 1:100 yr flood peaking in reservoir 10 days prior	
	TVA	15-50% of PMP	
	BC Hydro & AGEFA	1:1000-yr snowpack (for maximized snowpack only)	
Rainfall Temporal Distribution	CDA Guidelines & BC Hydro	Distribution of actual storm that is maximized	Used "alternating block method", recommended in WMO and HMR 52 for up to 72 hours and even distribution for the fourth day of the storm.
	ANS	Most critical possible for region Sequence that gives maximum PMF peak	
	USACE & USNRCS & TVA	Average mass curve	
	USBR	Average mass curve with critical arrangement of 6-hr rainfalls	

PMF Component	Criteria Used by Others[1]		Criteria Used by AECOM in PMF Estimation[2]
Loss rates or Initial Soil Moisture Condition	CDA Guidelines	As observed during major flood events	As estimated during calibration of SSARR Model
	ANS	Median soil moisture	
	USACE	Minimum infiltration during runoff	
	USNRCS	Appropriate Curve Number to reflect saturated soil	
	TVA & USBR	Minimum retention loss rates	
	BC Hydro	As estimated by calibrated SSARR Model	
Initial Reservoir Condition	CDA Guidelines	Full supply level in summer; or maximum water level usual in spring	For the summer PMF the maximum operating level was used. For the spring PMF the higher bracket of the normal operating level before the spring runoff
	ANS	Upper level of operating rule	
	OPG	For spring PMF: Minimum operating level	
	OPG	For Summer/Fall PMF: Maximum operating level or maximum historical summer level, whichever is lower	
	USACE	Top of conservation pool OR ½ of flood storage pool	
	USNRCS	1:100 yr flood peaking 10 days earlier	
	TVA	Median at start of PMP storm	
	BC Hydro	Full Supply Level	

Notes:
1. Acronyms:
ANS – American Nuclear Standard
AGEFA – Alberta Guidelines for Extreme Flood Analysis (2003)
CDADSG – Canadian Dam Association Dam Safety Guidelines (2007)
OPG – Ontario Power Generation DS-PRO-06 Revision R02
OMNR – 2011 Ontario Ministry of Natural Resources

The results of the analysis indicated that the maximum estimate of the Spring PMF resulted from combination of the Spring PMP and the melting of the 100-year snow accumulation. This PMF hydrograph was adopted for the dam safety analyses, and it is shown in Figure 6.1.2-1. The Summer PMF is shown in Figure 6.1.2-2 and it has virtually the same peak flow as the Spring PMF.

FIGURE 6.1.2-1
TALBOT RIVER DAM SPRING PMF

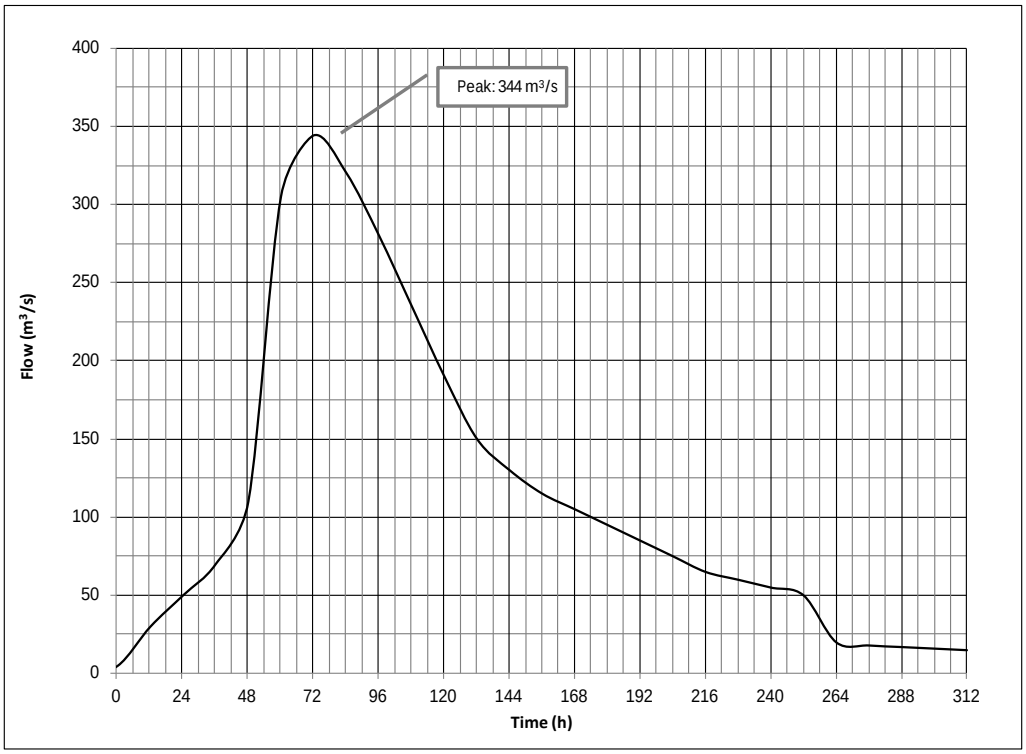
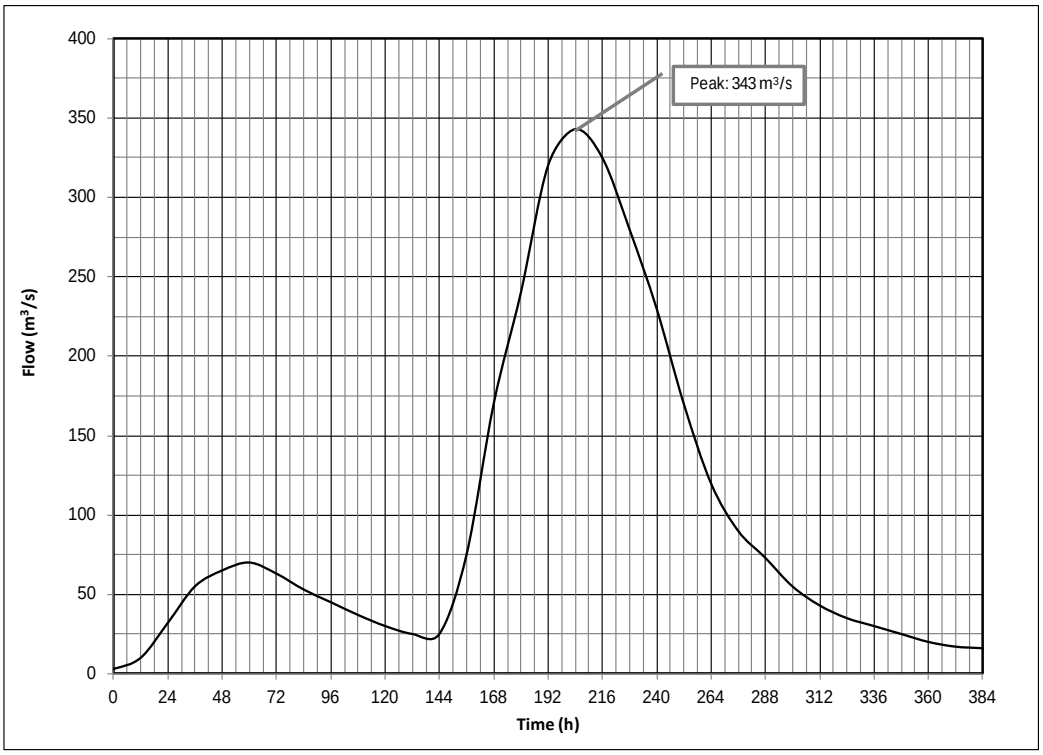
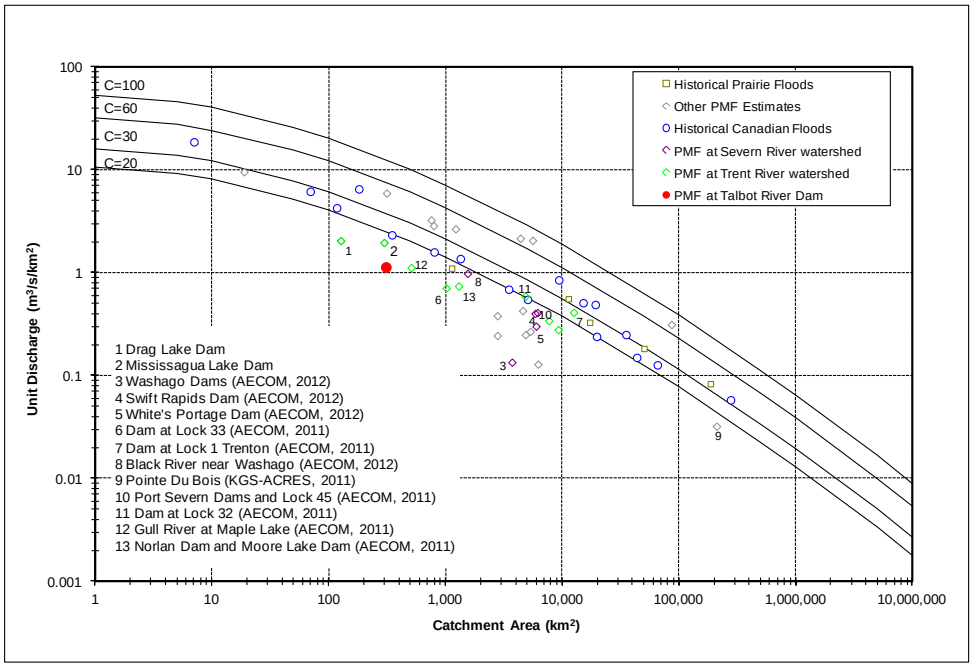


FIGURE 6.1.2-2
TALBOT RIVER DAM SUMMER PMF



To compare the PMF estimates for the Talbot River Dam with other estimates it was plotted on the Creager Diagram. The diagram was originally developed based on data from rivers in the United States (Ref. 25) and is commonly used to provide a graphical comparison of PMF estimates and large floods for different areas on a common basis. The diagram is shown in Figure 6.1.2-3 in SI units and provides a relationship between unit discharge (peak flow per unit area) and total drainage area. The magnitude of the Creager Constant "C" can be used as a measure of the flood-producing characteristics of a region. The comparison indicates that the unit discharge for the Talbot River at the dam site corresponds to a C value of less than 20, which is in the low range based on precedents. It is consistent with other values obtained for sites in the Trent River and Severn River watersheds as part of the studies by AECOM (Ref 12).

FIGURE 6.1.2-3
TALBOT RIVER DAM SPRING PMF PLOTTED IN THE CREAGER DIAGRAM



6.1.3 Flood Frequency Analysis

Flood frequency analyses for the Talbot River were also obtained from the previous hydrologic studies (Ref 12). Data at the Talbot River Dam, provided by Parks Canada, was available only for 13 years from 1936 to 1948. It was recognized in the study that the number of years was too low for estimating the required flood frequency values up to a return period of 100-years. The

flood frequency analysis was therefore extended only to the 50-year return period, and the peak of the 100-year flood was estimated by interpolation between the 50-year flood and the PMF. This methodology is commonly used to obtain flood frequency values exceeding the 100-year return period; but it this case, given the limited data, it was applied also to the estimation of the 100-year flood. Table 6.1-2 shows the peak flows for various return periods proposed by AECOM for the Talbot River Dam.

TABLE 6.1-2
PEAK FLOW VALUES AT THE TALBOT RIVER DAM FOR VARIOUS RETURN PERIODS

Return Period (years)	Peak Flow (m^3/s)
2	30
10	52
20	60
50	70
100	83

6.1.4 Extreme Flood Estimates

To obtain peak flow values for flood events with low frequency (return periods exceeding 100 years), the method proposed in the document “Guidelines on Extreme Flood Analysis” by Alberta Transportation (Ref 20) was used. The Alberta guidelines recommend an interpolation obtained by extending a straight line, drawn on probability/log paper, from the 1 in 100 year value out to the PMF, which is situated at a recurrence interval of 100,000 years. This permits the estimation of the 1 in 1,000 year flood, which is then used to interpolate other flood peaks between the 1 in 1,000 year and the PMF. The method is illustrated in Figure 6.1.4-1. Application of this method to the inflows to the Talbot River Dam resulted in the flood peaks listed in Table 6.1-3. This is the same method used in the AECOM report to obtain the peak of the 100-year flood based on the 50-year flood and the PMF. The peak of the 1:1,000-year flood, obtained in the AECOM report (144 m^3/s) is virtually the same as that shown in Table 6.1-3. The values shown in Table 6.1-3 were adopted for this dam safety analysis. Inflow hydrographs for floods corresponding to these extreme floods were derived by scaling the shape of the PMF hydrograph (Figure 6.1.2-1) based on the ratio of peak flows.

FIGURE 6.1.4-1
ESTIMATION OF THE PEAK OF THE 1,000-YEAR FLOOD BY INTERPOLATION BETWEEN THE RESULTS OF FLOOD FREQUENCY ANALYSES AND THE PMF

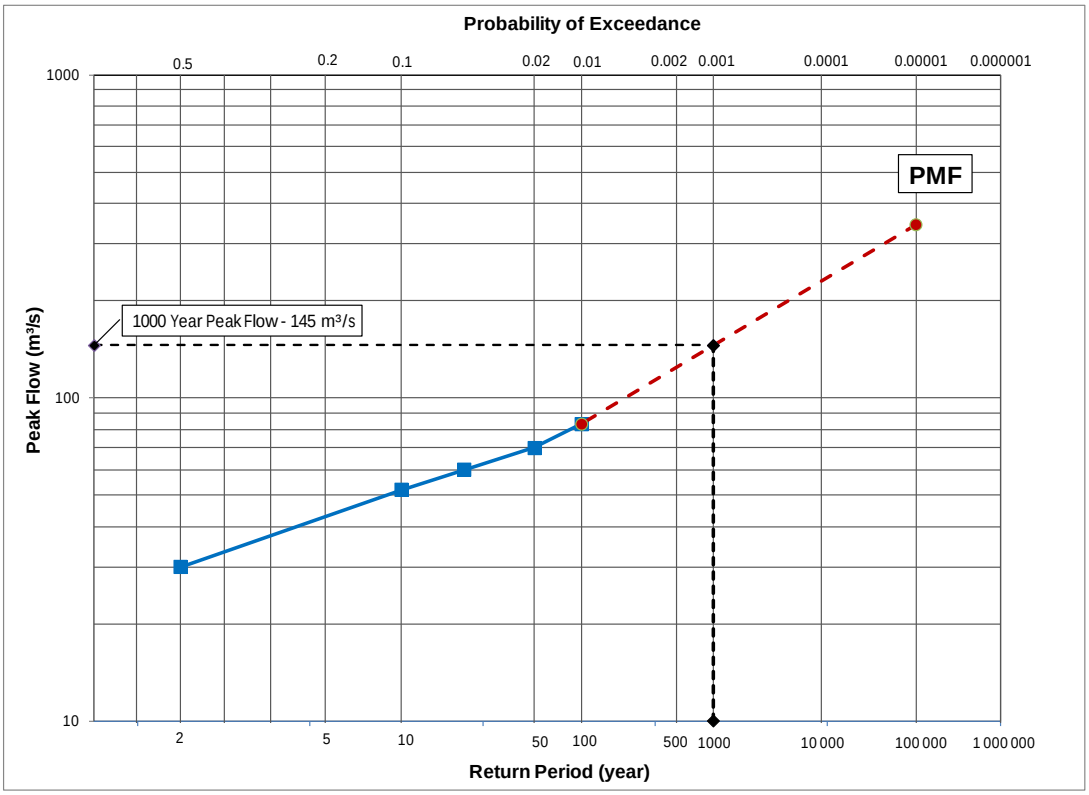


TABLE 6.1-3
EXTREME FLOODS FOR DAM SAFETY ANALYSES

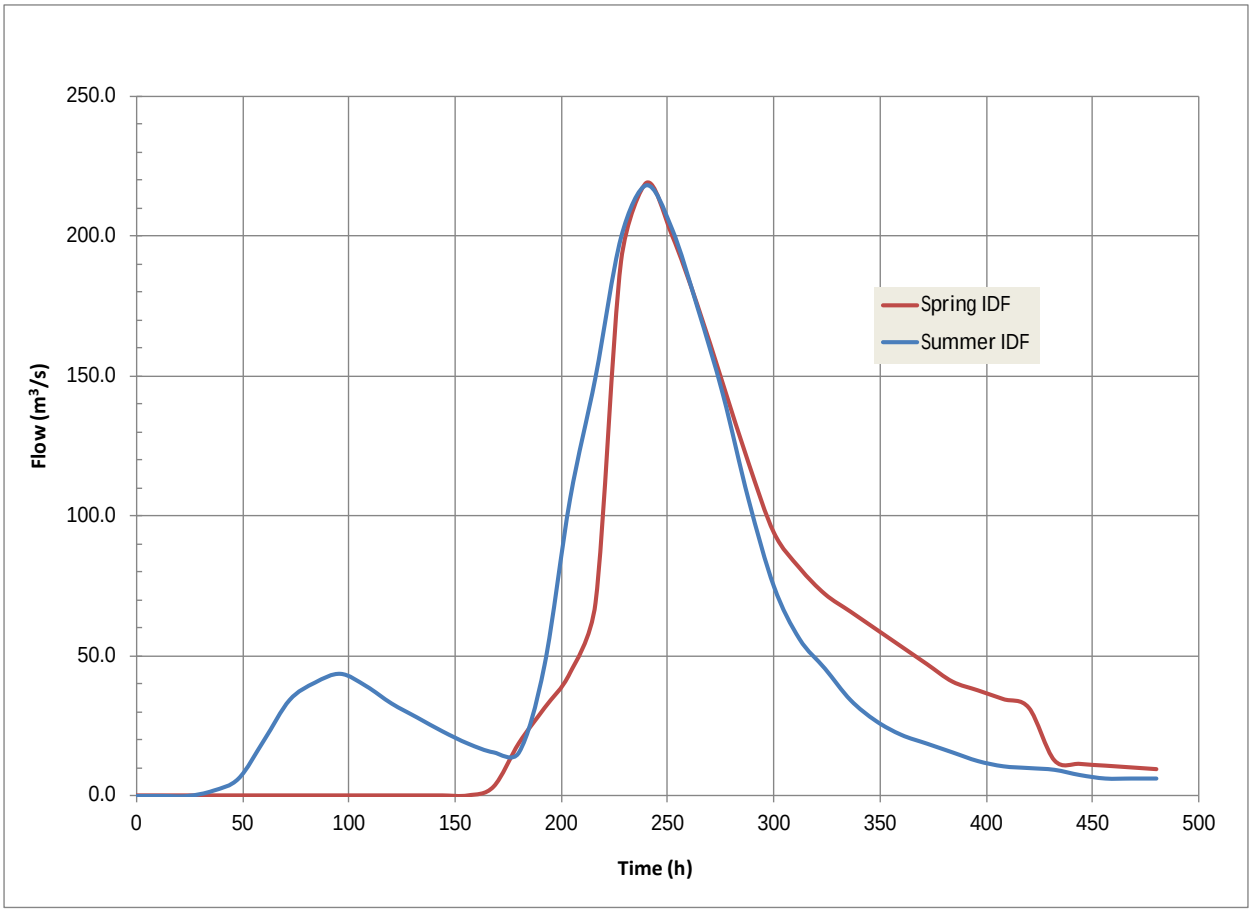
Flood Event	Peak Flow (m^3/s)
1:100 year flood	83
1:1,000 year flood	145
1/3 between 1:1,000 flood and PMF	212
2/3 between 1:1,000 flood and PMF	278
PMF	344

6.2 HYDRAULIC CAPACITY

Based on the results obtained in Section 5 the proposed IDF for the Talbot River Dam site is the flood 37% of the way between the 1,000-Year flood and the PMF. This flood would have a peak inflow of 219 m^3/s, calculated using the spring PMF. The peak of the corresponding summer

event would be virtually the same. The two hydrographs are shown in Figure 6.2-1. The capacity of the dam to safely pass the proposed IDF is discussed in the following sections.

FIGURE 6.2-1
TALBOT RIVER DAM IDF – SPRING AND SUMMER CONDITION



6.2.1 Operational Procedures

A review of the operational procedures for the Talbot River Dam and Lock 39 is provided in Section 7.0 of this report, based on the PCA Standing Orders, the responses to an operations questionnaire and direct witnessing of operations and equipment testing.

6.2.2 Dam Discharge Capacity

The discharge structures at the Talbot River Dam consist of two sluiceway bays and two overflow spillways. The sluiceways are 7.62 m wide and have a sill elevation of El. 227.27 m. The spillways are 6.10 m wide and their sill is at El. 230.30 m. The discharge through the sluiceways is controlled by installing or removing stoplogs. There are 11 wooden stoplogs on each sluiceway bay. The stoplogs are 0.3 m high.

The capacity of the sluiceways and spillways was estimated using the classical weir formula:

$$Q = c \times b \times h^{1.5}$$

Where:

- *Q* is the discharge through the weir,
- *h* is the height of the water level in the forebay over the sill elevation.
- *c* is the discharge coefficient which depends on the shape of the weir and its dimensions with respect to *h*.
- *b* is the open width of the sluiceway or spillway.

The capacity of the spillways was estimated with a discharge coefficient of 1.7 (metric units), that corresponds to a broad crested weir. This was adopted because the length of the weir in the direction of flow (*w*) is relatively large with respect to *h*.

The capacity of the sluiceways was estimated using a variable discharge coefficient, that depends on the ratios *h/w* and *h/p* (*p* is the elevation of the sill over the river bed upstream of the sluiceway). For *h/w* values of 0.4 or less a broad crested weir coefficient of 1.7 (metric units) was used. For larger values of h/w the coefficient ranged from that of a broad crested weir to a sharp crested weir coefficient. For *h/w* values of 1.5 or more a sharp crested weir coefficient was used. The discharge coefficient for a sharp crested weir was obtained using the Rehbock formula (Ref 26). The coefficient corresponds to:

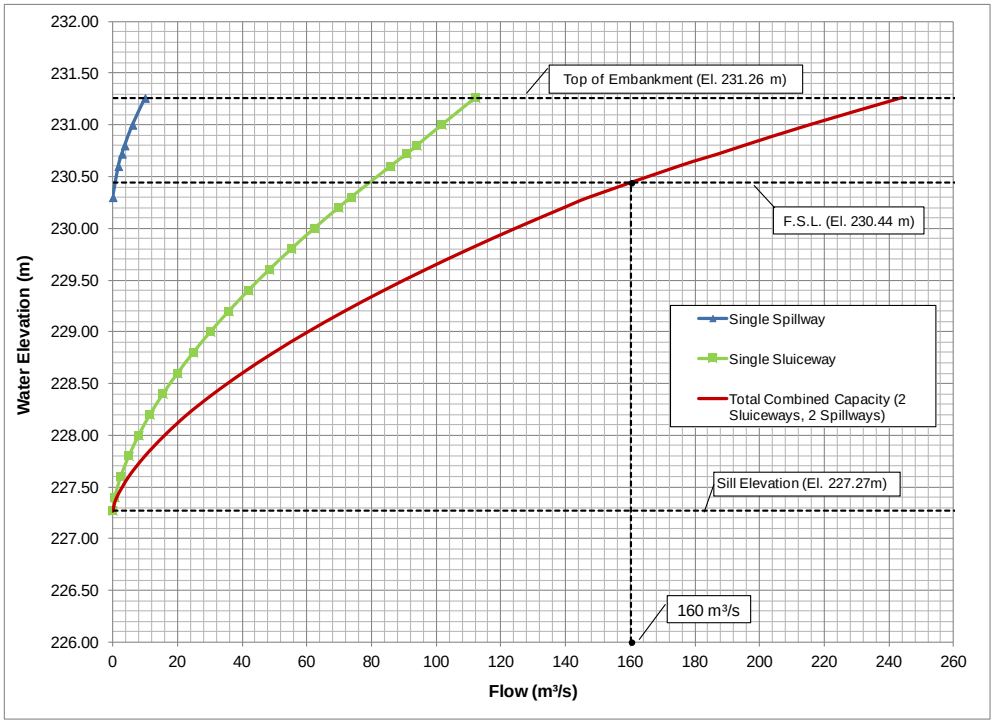
$$c = \frac{2}{3}\sqrt{2g}\left(0.611 + 0.075\left(\frac{h}{p}\right)\right)$$

Where *g* is the acceleration of gravity.

A maximum value of 1.97 (metric units) was adopted, to recognize the limits of application of the Rehbock formula and the fact that the sluiceways do not have a perfect ogee crest. This maximum value is based on measurements made by Ontario Hydro on sluiceway gates in the Lake St. Joseph Diversion (Ref 30).

A rating curve obtained with the assumptions previously described is shown in Figure 6.2.2-1. It shows that with the water at the FSL the combined capacity of the discharge structures would be 160 m^3/s, and that at the top of the earth embankment (El. 231.26 m) the total capacity would be 244 m^3/s. PCA has noted that with the existing flow control equipment, may be limited to being capable to remove a maximum of 8 stoplogs. In that case, 3 stoplogs would remain in each of the two sluiceway bays. The discharge capacity would then be limited to 120 m^3/s if the forebay level is at the FSL and 195 m^3/s if it is at the top of the earth embankment.

FIGURE 6.2.2-1
DISCHARGE CAPACITY RATING CURVE FOR THE TALBOT RIVER DAM ASSUMING SLUICEWAYS FULLY OPEN



6.2.3 Discharge Analysis / Operating Rule Curve

There is no reservoir at the Talbot River Dam. The levels during the navigation season are maintained above a minimum navigational level, reported to be El. 230.34 m (Ref 28). This reference does not indicate a maximum value, which was then assumed to be the adopted FSL: El. 230.44 m.

To evaluate the adequacy of the discharge capacity at the dam, the ability to operate the structures within an appropriate response time must be considered in addition to the theoretical capacity indicated in Section 6.2.2. A functional test was conducted at the dam as part of this study, and is described in Section 4.3.3. Although the test was done in non-flood conditions, it allowed KGS Group to estimate that it would take 1 to 1.5 hours of on-site work to remove all stoplogs from the dam. Factoring the time for mobilization, it is estimated that the operation crew could open the two sluiceways within 2 hours of being requested. This time is considered adequate, provided that the decision to open the structures is made promptly and that there is sufficient personnel to operate this dam in conjunction with other dams in the Talbot River and possibly nearby watersheds. It is important to note that, as previously indicated, experience on the nearby Bolsover Dam suggests that a maximum of 8 stoplogs could be removed. This would limit the total discharge capacity, as indicated in Section 6.2.2.

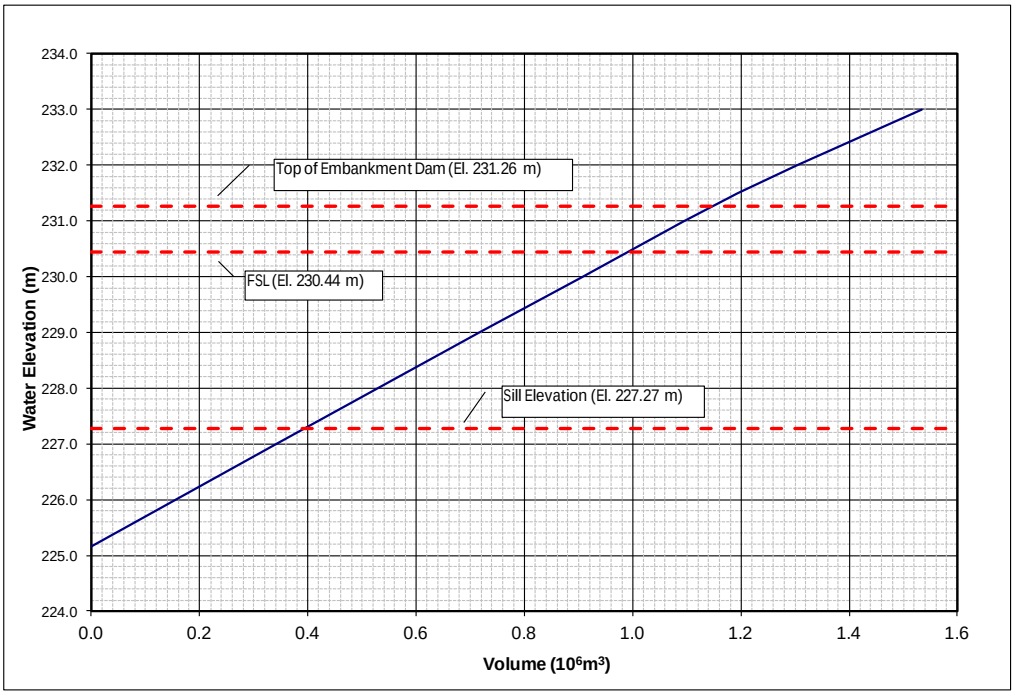
The access to the deck of the dam is from the earth embankment. If during a flood incipient overtopping of the embankment occurs it would not only endanger the stability of the embankment but also impede the access to operate the sluiceway structure. Therefore, it is important the dam is operated early during the flood and that clear instructions, reference levels and emergency measures be determined in advance and documented in an ERP. Another factor that could assist in the operation of the dam during a flood is the forecast of flood volumes and flow peaks. In the case of the Talbot River Dam, there would be information available at Canal Lake and the Bolsover Dam.

6.2.4 Adequacy of Discharge Capacity

Based on the results obtained in Section 5 the proposed IDF for the Talbot River Dam site is the flood 37% of the way between the 1000-year flood and the PMF. This flood would have a peak inflow of 219 m^3/s for a spring flood condition (and it is virtually the same for a summer flood).

A storage curve for the site is provided in Figure 6.2.4-1, obtained from estimates of the surface area of the forebay using a GIS system and with contours generated by the OMNR WRIP DEM.

FIGURE 6.2.4-1
STAGE – STORAGE CURVE FOR THE TALBOT RIVER DAM



The storage volume available from the sill level of the sluiceway to the top of the embankment dam is approximately 0.7 million cubic metres. The storage volume from the FSL to the top of the embankment dam is less than 0.2 million cubic metres. This is very small compared to the volume of the IDF, estimated to be approximately 14 million cubic metres, and effectively results in no attenuation of the flood. The capacity at the dam required to pass the IDF would then be equal to the peak of the inflow flood: 219 m^3/s. Based on the rating curve shown in Figure 6.2.2-1 the required level to pass the IDF, with the sluiceways fully open, would be

El. 231.03 m. This is the level adopted as representative of the passage of the IDF considering all the stoplogs are removed.

Other levels were investigated for various alternative conditions. If three stoplogs remain in place, the water level at the dam would surpass the top of the earth embankment (El. 231.26 m).

6.2.5 Freeboard and Wave Action

Wave action and freeboard requirements were considered for two scenarios as part of the Dam Safety Review:

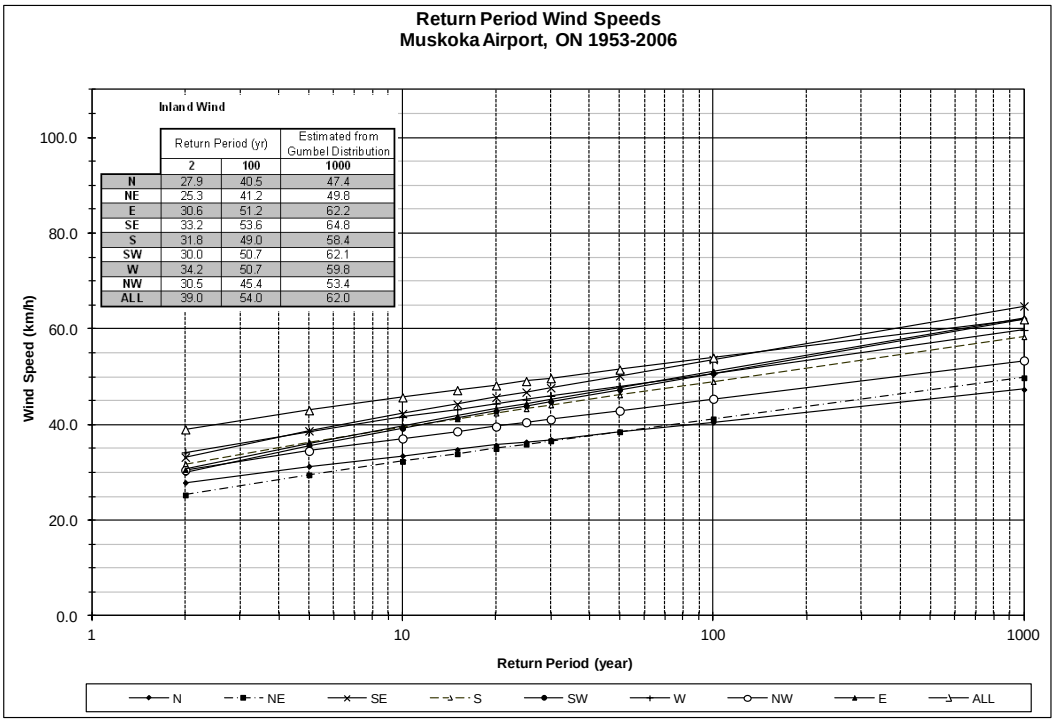
- Non-flood conditions or normal maximum level (FSL is normally adopted), for which the adopted criterion was to verify the freeboard required for preventing overtopping by 95% of the waves during a windstorm with an annual exceedance probability (AEP) of 1 in 1000.
- Flood conditions or IDF level, for which the adopted criterion was to prevent overtopping by 99% of the waves during a windstorm with an annual exceedance probability (AEP) of 1 in 2.

The CDA Dam Safety Guidelines indicate that for concrete dams, which can usually resist overtopping without serious damage, the minimum overtopping can be reduced or overtopping can be allowed, provided that the integrity of the dam and associated structures is not compromised. However, when the crest of the concrete dam is used as an access for the outflow control structures (as it is the case for the Talbot River Dam) the wave splashing during the passage of the IDF should be limited to 1% of the design waves. This was the criteria used for IDF conditions for the concrete structure.

The wind data used for the analysis was obtained from the Environment Canada DSWind database, for the station located at the Muskoka Airport. This station is located 55 km north of the Talbot River Dam. It was selected because of its relative proximity to the site and its long period of record. The DSWind software provided wind speeds for each cardinal and inter-cardinal direction, based on hourly data, for return periods from 2 to 100 years. For the 1000-year return period, wind speeds were extrapolated from the DSWind frequency curves, using a

Gumbel Distribution. The dominant direction for wind generated waves, for the Talbot River Dam is East, due to the location of the dam with respect to the water in the Talbot River. The wind speeds used in the analysis are shown in Figure 6.2.5-1. The wind speeds were multiplied by a factor of 1.2, as recommended in the USACE Shore Protection Manual (Ref. 21), to convert overland wind speeds into overwater wind speeds.

FIGURE 6.2.5-1
WIND DATA AND STATISTICAL ANALYSIS – MUSKOKA AIRPORT



The significant wave height and the wave period for wind generated waves were obtained based on the formulations provided in the USACE Shore Protection Manual, using the wind speeds adopted for each condition (non-flood and IDF). The wave height corresponding to each design wave was obtained assuming the standard Rayleigh distribution of waves. The wave run-up for the dam and the lock was obtained using linear wave theory, assuming full reflection on a smooth vertical wall. The corresponding formulae and charts for calculation were obtained from the USACE Shore Protection Manual (Ref. 21). For the embankment dam, the wave run-up formulations in the Coastal Engineering Manual (Ref 29) were used. For calculation of the wind set-up, the classical Zuider Sea equation was used.

The results from the wave run-up and wind set-up calculations for the Talbot River Dam and for Lock 39 are shown in Table 6.2-1 for non-flood conditions, and indicate that the freeboard at is adequate. For the conditions during passage of the IDF, the required freeboard was calculated and is shown in Table 6.2-2. The table indicates that the concrete sluiceway and Lock 39 structures would provide adequate freeboard but the concrete core wall of the Right and Left Earthfill Embankments would be overtopped.

TABLE 6.2-1
FREEBOARD ANALYSIS FOR THE TALBOT RIVER DAM AND LOCK 39 FOR 1:1000 YEAR WIND AND FSL

	Sluiceway	Embankment	Lock 39
Wind Station	Muskoka	Muskoka	Muskoka
Dam Crest Elevation (m)	231.26	231.26	231.24
Impervious Core Top Elevation (m)	N/A	230.85	N/A
IDF Level (m)	230.44	230.44	230.44
Available Freeboard Crest to IDF Level (m)	0.82	0.82	0.8
Available Freeboard Impervious Core (m)	N/A	0.41	N/A
Wind Direction	East	East	Northeast
1,000-year Wind Speed (km/h)	74.6	74.6	59.8
Effective Fetch (km)	0.3	0.6	0.2
Significant Wave height (m)	0.26	0.37	0.16
Wave Period (s)	1.3	1.6	1.0
Wave Runup at 1% Exceedance (m)	0.45	0.35	0.27
Fetch for wind setup calculations (km)	1.1	1.1	2.5
Wind Setup (m)	0.02	0.02	0.04
Required Freeboard to Crest (m)	0.47	0.37	0.31
Required Freeboard for Impervious Core (m)	N/A	0.02	N/A
Is available freeboard adequate?	**Yes**	**Yes**	**Yes**

TABLE 6.2-2
FREEBOARD ANALYSIS FOR THE TALBOT RIVER DAM AND LOCK 39 FOR 1:2 YEAR WIND AND IDF

	Sluiceway	Embankment	Lock 39
Wind Station	Muskoka	Muskoka	Muskoka
Dam Crest Elevation (m)	231.26	231.26	231.24
Impervious Core Top Elevation (m)	N/A	230.85	N/A
IDF Level (m)	231.03	231.03	231.03
Available Freeboard Crest to IDF Level (m)	0.23	0.23	0.21
Available Freeboard Impervious Core (m)	N/A	-0.18	N/A
Wind Direction	East	East	Northeast

	Sluiceway	Embankment	Lock 39
2-year Wind Speed (km/h)	36.7	36.7	30.4
Effective Fetch (km)	0.3	0.6	0.2
Significant Wave height (m)	0.11	0.15	0.07
Wave Period (s)	1.0	1.2	0.8
Wave Runup at 1% Exceedance (m)	0.23	0.20	0.15
Fetch for wind setup calculations (km)	1.1	1.1	2.5
Wind Setup (m)	0.00	0.00	0.01
Required Freeboard to Crest (m)	0.23	0.20	0.16
Required Freeboard for Impervious Core (m)	N/A	0.00	N/A
Is available freeboard adequate?	**Yes**	**No**	**Yes**

6.2.6 Simulation of Dam Breach During Passage of the IDF

To provide additional information for the analysis of the potential hazard posed by a breach of the Talbot River Dam during the proposed IDF, a dam break scenario was simulated using the following assumptions:

- Initial water level in the reservoir at FSL (El. 230.44 m).
- Inflows to the reservoir corresponding to the proposed IDF in spring conditions (peak flow of 229 m^3/s).
- Discharge capacity corresponding to the two sluiceways fully open and the spillways.

The assumption for the discharge capacity at the dam is based on the size of the structures. As indicated before, it could be reduced if not all the stoplogs can be removed. Two scenarios were simulated, as follows:

- No dam failure, and
- Dam failure at the peak of the flood, when the water levels at the dam are at maximum.

The results of these simulations are shown in Table 6.2-3 and in the inundation maps in Appendices G and H.

TABLE 6.2-3
MODEL RESULTS FOR THE BREACH OF THE TALBOT RIVER DAM DURING THE IDF

Location	Distance from Talbot dam (km)	River Station (HECRAS Model)	Dam Breach					No Breach	
			Peak Flow (m^3/s)	Peak Water Level (m)	Time to Peak Water Level[1] (h)	Wave Arrival Time[1] (h)	Peak Velocity (m/s)	Peak Flow (m^3/s)	Peak Water Level (m)
Downstream of Bolsover Dam	0.0	12278.0	220	235.5		NA	0.4	210	235.5
Upstream of Talbot Dam	0.0	11008.4	210	235.5		NA	0.4	210	235.5
Lock 38 / Talbot Dam									
Downstream of Talbot Dam	0.0	10983.1	220	231.2	0.0	NA	1.1	220	231.2
0.3 km downstream of Lock 38 (Bridge at County Road 50)	0.3	10716.6	260	231.2	0.0	NA	0.6	220	231.2
0.5 km downstream of Lock 38	0.5	10500.7	290	231.2	0.0	NA	0.2	220	231.2
Upstream of Talbot River Dam	1.1	9864.4	520	231.2	0.0	NA	0.2	220	231.2
Talbot River Dam									
Downstream of Talbot River Dam	1.2	9812.9	520	229.2	0.9	0.0	3.1	220	228.2
0.5 km from Talbot River Dam	1.6	9366.4	510	228.5	0.8	0.1	3.2	220	227.7
0.9 km from Talbot River Dam	2.0	9031.8	510	228.4	0.8	0.1	1.5	220	227.6
2.2 km from Talbot River Dam	3.3	7683.9	470	225.7	1.1	0.2	2.5	220	224.9
3.9 km from Talbot River Dam	5.0	5977.5	360	225.0	1.3	0.3	2.3	220	224.3
5.2 km from Talbot River Dam	6.3	4723.1	340	223.9	1.5	0.3	2.2	220	223.4
Gamebridge near Hazel St.	7.0	4007.5	340	223.4	1.6	0.4	2.4	220	223.0
Gamebridge bridge at Ramara Road 51	7.3	3651.0	330	223.2	1.6	0.4	2.2	220	222.8
Bridge at Hwy 12	8.6	2442.6	320	221.9	1.9	0.6	2.5	220	221.6
Near Furniss Drive	10.0	1010.1	310	220.9	2.3	0.7	1.8	220	220.7
Talbot River near confluence with canal	10.9	62.8	300	219.7	2.6	0.8	2.5	220	219.4
Trent Talbot Marina	5.9	1084.2	120	219.5	2.7	0.9	1.1	80	219.3
CN Rail bridge	6.3	652.1	120	219.3	2.8	1.0	1.4	80	219.1
Bridge at Ramara Road 47	6.8	156.9	120	219.1	3.1	1.1	1.2	80	219.1
Canal									
Canal inlet near Talbot River Dam	1.1	5950.0	10	231.2	0.0	0.0	0.1	10	231.2
Upstream of Portage Lock 39	2.4	4567.0	10	231.2	0.0	0.1	0.2	10	231.2
Downstream of Portage Lock 39	2.5	4528.0	10	227.0	0.1	0.1	0.3	10	227.0
Upstream of Lock 40	3.2	3797.0	10	227.0	0.1	0.1	0.3	10	227.0
Downstream of Lock 40	3.2	3758.0	10	222.8	0.0	0.1	0.2	10	222.8
Upstream of Lock 41	4.3	2712.0	30	222.8	0.0	0.2	0.2	10	222.8
Downstream of Lock 41	4.3	2673.0	30	219.6	1.7	0.8	0.1	10	219.4
Confluence with the Talbot River	5.7	1282.7	190	219.7	2.6	0.8	2.0	140	219.4
CN Rail bridge	5.9	1090.2	190	219.5	2.6	0.8	2.1	140	219.3
Bridge at Ramara Road 47	6.6	405.9	190	219.1	2.7	0.8	1.6	140	219.1

6.2.7 Water Impoundment Review

As part of this study, KGS Group was requested to investigate if during passage of the IDF the water levels would be such that they would:

- Overtop the reservoir banks or dykes causing spilling into adjacent lower lands, and/or,
- Flood properties beyond the flood rights.

It was indicated in Section 6.2.4 that the estimated level for passage of the IDF was El. 231.03 m. There would be limited flooding of properties upstream of the dam or around it, at this level. However, if three stoplogs are left in the sluiceways then the level upstream of the dam, during passage of the IDF, would exceed the crest of the embankment and would flood roads

and properties near the dam, and upstream of it. The flooding is shown in Figure I1 in Appendix I.

It is KGS Group's understanding that PCA does not have any flood rights in this area.

6.3 SEISMOLOGY

The approach undertaken for this DSR and endorsed by PWGSC/PCA was to first determine the ground motion values for the seismic hazard evaluation from Geological Survey of Canada (GSC) and perform the associated stability analyses on a preliminary basis (see Sections 6.4 and 6.5). In the event that the stability under earthquake conditions was considered marginal, then a site-specific hazard analysis could be performed (after consultation with PWGSC/PCA) to better estimate the seismic parameters representative for the dam site. Although current practice for seismic hazard evaluation for dam structures as well as 2007 CDA Guidelines recommend determining ground motion design parameters including the PGA value from a site-specific seismic hazard evaluation, the method undertaken herein is considered a reasonable approach to provide an estimate of the earthquake stability conditions, particularly for low seismic areas (see Figures 6.3.1-1 and 6.3.1-2). Detailed evaluation and determination of the site-specific seismic hazard for the Talbot River Dam and Lock 39 is beyond the current scope of work.

The Talbot River Dam and Lock 39 are classified as "High A – High consequence dams" as determined under this DSR and in accordance with the current Parks Canada Agency (PCA) hazard classification. As such, the seismic hazard assessment for the dams should be based on a maximum design earthquake (MDE) with a probability of annual exceedence of at least 1 in 2,500.

6.3.1 PGA Values from Geological Survey of Canada (GSC)

The PGA values for the dam site were obtained from GSC (see Figure 6.3.1-1 and 6.3.1-2), and are provided in Table 6.3-1. The ground motion parameters available from the GSC database were developed for and are limited to the design of building structures. The values are typically not recommended for seismic assessment of dam structures (2007 CDA Guidelines).

TABLE 6.3-1
PGA VALUES FROM GSC DATABASE FOR TALBOT RIVER DAM AND LOCK 39 SITE & ADJUSTED FOR SITE CLASS A & D AND MEAN VALUE

Probability of Exceedance	Return period	Estimated PGA (Site Class "C" & Median Value)	Estimated- Lock 39 Adjusted PGA (Site Class "D" & Mean Value)	Estimated Adjusted PGA- Talbot River Dam (Site Class "A" & Mean Value)
0.000404	1 in 2,500	0.048	0.098	0.053
0.001	1 in 1,000	0.031	0.063	0.034
0.0021	1 in 500	0.021	0.043	0.023
0.01	1 in 100	0.008	0.016	0.84

A fundamental limitation of the ground motion values obtained from the GSC database is that the values are the statistical medians of the data set (i.e. Standard deviation = 0/50th percentile). Whereas, the current practice for seismic hazard evaluation of dam structures and recommendations provided by the 2007 CDA Guidelines is to use the statistical mean values (i.e. standard deviation = 1/84th percentile). It should also be noted that the values provided in Table 6.6 from the GSC database are for "Site Class C –Very Dense Soil/Soft Rock".

The procedures and factors that can be used to adjust the PGA values provided by GSC for the actual soil type and assumed duration are provided in the 2010 National Building Code of Canada (NBCC). The Talbot River Dam is founded on bedrock, and the site is classified as "Site Class A – Hard Rock". Lock 39 is founded on piles driven to hard till / bedrock, and the surrounding foundation soils are considered relatively stiff. As such, the site has been classified as "Site Class D – Stiff Soils". In accordance with the 2010 NBCC Table 4.1.8.4B, the adjustment PGA multiplication factor for "Site Class A" and "Site Class D" is 0.7 and 1.3 respectively. Based on a comparison of PGA mean values and 1 / 84th percentile values for select Canadian sites (in particular Toronto and Ottawa sites) presented in a 2003 paper by Adams and Atkinson (Ref. 38, Table 1) it is reasonable to apply a multiplication factor of 1.5 to adjust the Talbot River Dam and Lock 39 site PGA median values to estimate the mean values. Table 6.3-1 includes the estimated adjusted PGA values for the Talbot River Dam and Lock 39 site.

Under the current dam configuration and with the low seismic parameters representative for the area, the stability of the structures under earthquake conditions was not found to be the critical or controlling case (see Section 6.4 for additional details). The approach undertaken for the

DSR to evaluate the estimated stability conditions under earthquake conditions is considered a reasonable method, and more detailed analyses is not considered necessary at this time.

FIGURE 6.3.1-1
PGA SEISMIC HAZARD MAP FROM GEOLOGICAL SURVEY CANADA (NBCC 2% PROBABILITY OF EXCEEDANCE IN 50 YEARS)

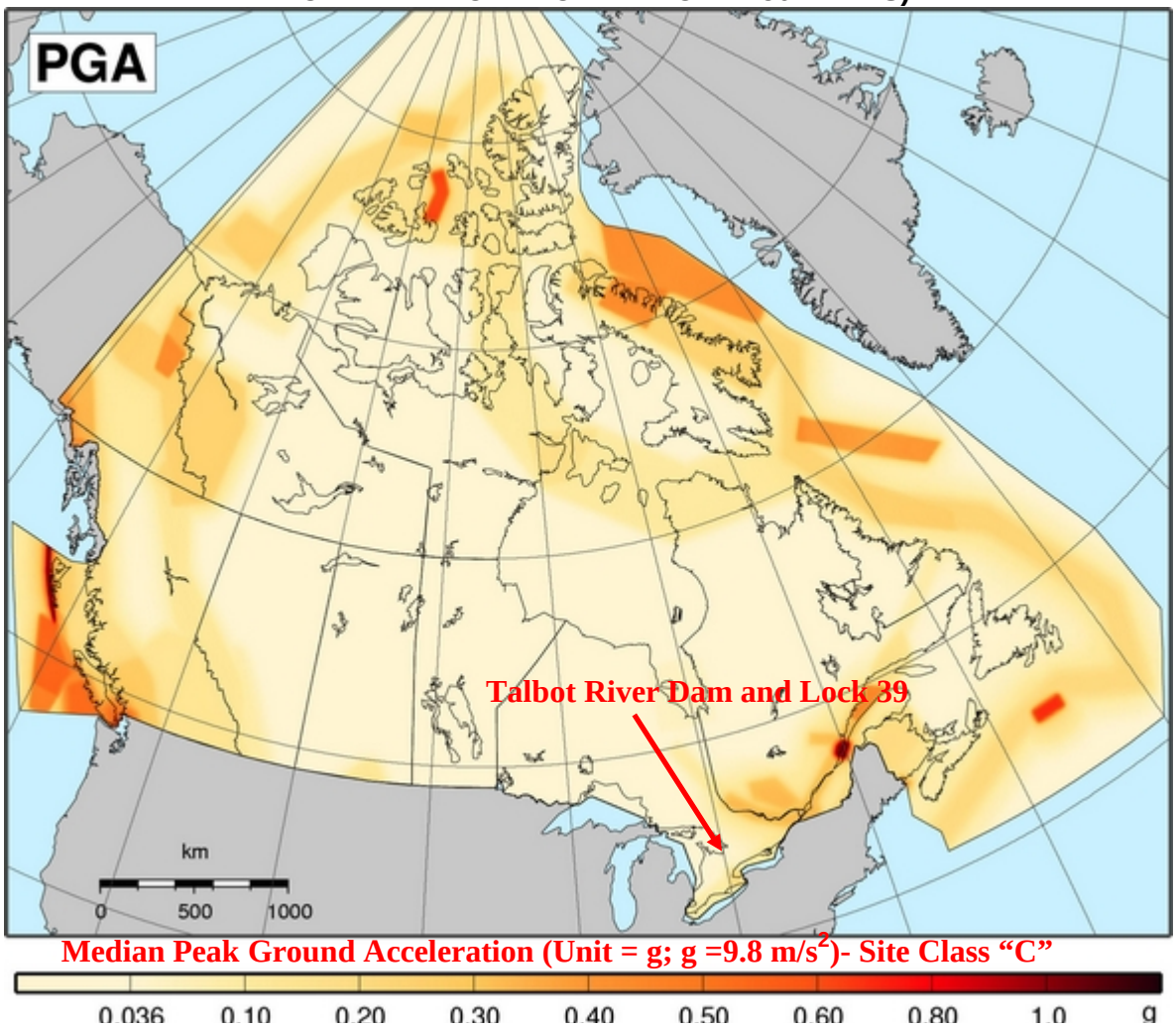


FIGURE 6.3.1-2:
NBCC 2010 PGA VALUES FOR TALBOT RIVER DAM AND LOCK 39 SITE FROM GEOLOGICAL SURVEY CANADA

2010 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: Shan Gnanasunthar, KGS Group February 05, 2014

Site Coordinates: 44.507 North 79.119 West

User File Reference: Talbot River Dam

National Building Code ground motions:
2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.166	0.117	0.070	0.023	0.048

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. ***These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.029	0.074	0.109
Sa(0.5)	0.018	0.051	0.076
Sa(1.0)	0.009	0.030	0.047
Sa(2.0)	0.003	0.010	0.015
PGA	0.008	0.021	0.031

References

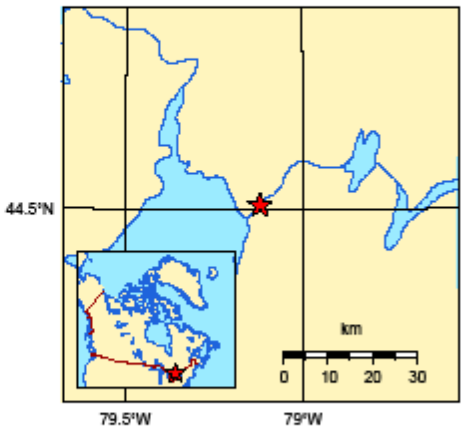
National Building Code of Canada 2010 NRCC no. 53301; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543 (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français



6.4 STRUCTURAL ANALYSIS

6.4.1 Stability Assessment/Analysis

The assessment of the concrete structures is based on:

- drawings and reports provided by PWGSC,
- available survey data
- field measurements

The structures were analyzed based on the 2007 CDA Dam Safety Guidelines criteria. KGS Group assessed the stability of the structures, and compared the results to the CDA acceptance criteria. The structural sections examined were as follows:

- Gravity Bulkheads
- Spillway
- Sluiceway
- Lock 39 – U-Frame Lock Chamber.

The above noted sections are identified in the “as-found” drawings provided in Appendix B.

6.4.1.1 Dam Structures

The stability of the dam structures was calculated using the gravity method. By this method, the dam is assumed to be a two dimensional rigid block. All loads are carried by gravity to the underlying bedrock, and the foundation pressure distribution is assumed linear. This is also known as rigid body analysis. The stability was assessed at the bedrock/concrete interface, which is typically the weakest plane of failure.

The load parameters and acceptance criteria for the stability assessment were based on the 2007 CDA Dam Safety Guidelines. KGS Group used stability parameters with no cohesion based on the 2007 CDA Dam Safety Guidelines and our previous experience and overall

industry practice. Usual, unusual and extreme loading combinations as specified in the 2007 CDA Dam Safety Guidelines were utilized in the calculations.

The major parameters used for the dam stability analyses were as follows:

General Parameters

Incremental Consequence Category	HIGH A
Water Unit Weight	9.81 kN/m^3
Friction Angle at Concrete to Rock Interface	36° lower bound to 45^0 upper bound
Cohesion at Concrete to Rock Interface	0.0 kPa
Submerged Unit Weight of silt (assumed)	7.7 kN/ m^3
Internal Friction Angle of silt (assumed)	25°
Concrete Unit Weight (assumed)	23.5 kN/m^3
Concrete Compressive Strength (assumed)	17 MPa
Unit Weight of Backfill	19.5 kN/m^3 for Dam and 18.0 kN/m^3 for Lock Monoliths
Internal Friction Angle of Backfill	30° for Dam and 28° for Lock Monoliths

As discussed in Section 4.4.3.1 a friction angle of 36° and zero cohesion (c) at the rock foundation and concrete interface was used for the base case stability analyses.

To meet the CDA stability requirements, two conditions must be met: one for sliding resistance and second for “location of resultant” (i.e., overturning). Friction angle and cohesion affects the sliding resistance but only tensile strength affects the overturning resistance. Typically, tensile strength is assumed to be equal to half the cohesion value.

It should be noted that the CDA requirements regarding the “location of resultant” is more stringent than the 2011 Ontario Ministry of Natural Resources (OMNR) Dam Safety Guidelines. The 2011 OMNR states:

"For the Usual Load cases, the Resultant should be in the middle third of the surface being analyzed. For new dams this is achieved by design. For existing dams this requirement may be waived if sliding factors of safety are met, and provided that the resultant is within the base of the dam and limits on allowable bearing stresses are not exceeded."

PCA may wish to take into consideration the 2011 OMNR less stringent stability requirements when prioritizing rehabilitation programs.

Where the structures did not meet the stability requirements with an assumed friction angle of 36° and zero cohesion, KGS Group performed sensitivity analyses. The reason for this work was to determine the required minimum friction angle for the dam to meet stability standards. A reasonable lower bound and upper bound for the friction angle is 36° and 45°. For the cases where some tensile strength is required to meet the overturning standards (i.e., location of resultant), KGS Group determined the minimum required tensile strength assuming a base shear friction angle of 36°. Typically, field data has determined only small values of cohesion (or σt) for similar sites to the Talbot Dam site. In general, the c is found to be less than 70 kPa (10 psi). KGS Group and the CDA Guidelines, however, do not generally include cohesion for the following reasons (see Section 4.4.3.1):

- The cohesion value is based on limited field samples which may not be representative for the entire foundation base.
- The cohesion value can change due to change in future site conditions such as seepage, leakage, and/or calcite formation at the rock/concrete interface.

The dam was built in year 1908 and decreased compressive strength with time due to freeze thaw would be expected for the dam. Since the concrete data is unavailable, the concrete compressive strength was assumed to be 17 MPa to account for the concrete age (Refer to Report DSO-05-05, Materials Properties Model of Aging Concrete, BUREAU OF RECLAMATION. Technical Service Centre, Denver, Colorado, Materials Engineering and Research Laboratory, 86-68180, Ref. 39).

6.4.1.2 Lock 39 Chamber

Lock 39 chamber is a U-frame structure consisting of a concrete base and concrete walls. The walls are subjected to the backfill pressures and the base is subjected to buoyant forces. Because the U-frame lock chamber is embedded between the backfill as shown on the drawings, there is no concern regarding the global sliding and overturning stability of the U-frame structure. However, since the U-frame lock chamber is subjected to buoyant forces which affect structural stability, the floatation stability of the lock was evaluated to exam if the structure has adequate flotation stability. Because CDA guidelines do not provide criteria for the flotation stability, KGS Group reviewed the flotation stability of the lock by using the calculation methods and performance factors as outlined in ETL 1110-2-307, "Floatation Stability Criteria for Concrete Hydraulic Structures", U.S. Army Corps of Engineers (USACE).

PWGSC did not have available any drawings nor structural data for the lock chamber, thus the integration of the lock chamber walls with the base could not be verified. Therefore, a review of the chamber wall sliding stability was deemed to be necessary. KGS Group performed the sliding stability analysis at the lock wall / base slab interface for applicable loading cases. In the analysis, a shear friction angle of 41° and c = 0 was assumed at the interface due to lack of data. The consideration of using zero cohesion is because the joint condition at the interface is unknown and a good bonded joint condition can not be confirmed. A shear friction angle of 41° was assumed as recommended by the CDA Guidelines for an unbonded concrete lift joint.

6.4.1.3 Loading

Earthquake

Since the Hazard Classification for Talbot River Dam is HIGH A, the Design Basis Earthquake (DBE) should have a probability of annual exceedence of 1 in 2500 for this dam as specified in Figure 6 of the Directive for Dam Safety Program of Parks Canada Dams and Water-Retaining Structures (2009). The horizontal Peak Ground Acceleration (PGA) is estimated to be 5.0% g for the site as discussed in Section 6.3.

Pseudo-Static Analysis (Seismic Coefficient) was performed by using a seismic coefficient equal to the PGA expressed as a fraction of gravity in accordance with CDA Guidelines. Earthquake-induced horizontal and vertical inertia forces were simultaneously taken into account for the stability analysis of the concrete structure. The vertical seismic coefficient is scaled from the horizontal seismic coefficient using a scaling factor in the range of 1/2 to 2/3. Two thirds of the horizontal seismic coefficient was assumed for the vertical seismic coefficient in the calculation. The earthquake-induced hydrodynamic pressure of the reservoir was also considered.

Ice

The approach to determine the thermal ice load must consider site-specific characteristics and operating information.

For the project location, a pure thermal ice load was calculated to be 77.0 kN/m based on the database of the Centre for Energy Advancement through Technological Innovation (Canada) (CEATI). The load presented was average results of four stations within 200 km from the Talbot River Dam for 1:100 year thermal ice loads. According to the Operator's observation, the estimated ice thickness of the ice formation is about 150 mm. If the 150 mm ice thickness could be confirmed, the magnitude of the ice load may be limited up to the buckling load of the ice sheet. The buckling load of a 150 mm thick ice sheet is about 65 kN/m (Refer to OPG Standard DS-PRO-08, Procedure for Determining Ice Loads in the Assessment of Existing concrete Dams). Therefore, the 77.0 kN/m CEATI ice load may be deemed to be conservative and reasonable for the stability assessment of the dam. This ice load was considered to act at 305 mm below the water level in the calculations. A 100% ice load applied on the stoplogs was used to check the concrete pier stability. It was assumed that the ice sheet might effectively bridge from pier to pier.

Dynamic ice load was not considered since a moving ice floe to impact the dam structure element was not found to have occurred in the past (confirmed by the Operators). It was also confirmed that no frazil ice or debris have jammed or blocked the sluiceways.

Water Pressure

Provided by PWGSC, maximum normal operating headwater levels for summer is at elevations 230.44 m and the tail water is dry. The head water level for the winter season was estimated to be 230.14 m and the related tail water was assumed dry at the toe of the dam. The estimated IDF level is at elevation 231.03 m for the headwater level and the associated tailwater level at the toe of dam is 230.44 m.

Full uplift, varying linearly from 100% headwater pressure at the upstream face to 100% tailwater pressure at the downstream face, was assumed. Once a cracked plane was indicated by the calculations, crack analysis was performed. The modified uplift was assumed to be full headwater pressure over the length of the crack, varying as a straight line from full headwater pressure at the end of the crack to full tailwater pressure at the toe. The stress distribution and shear-friction safety factor was calculated along the uncracked portion.

Silt Pressure

According to the data from the site measurements, a layer of reservoir silt, with varying thickness exists at the front of the dam. The pressure from silt was added to the stability analysis. No increment of silt pressure was assumed for the post-earthquake loading case due to the effect of liquefaction during earthquake may not be significant.

Backfill Pressure at D/S of the Dam

According to the available data including the new survey data, backfill is present at toe of the dam. The backfill is confined by the surrounding concrete walls and a paved road. The at-rest lateral pressure of the backfill was added to the stability forces under normal and earthquake loading conditions. However, under IDF condition, the contribution of the backfill would be removed since it was assumed that the backfill would be washed away during the flood.

Force due to passive rock wedge

The portion of dam base appears keyed into the bed rock as shown on the reference drawings (see Appendix B). However, the sliding capacity of the possible passive rock wedge

downstream of the key was not taking into the dam stability calculations. This is because the calculated passive pressure of the rock wedge is insignificant by using its submerged gravitational sliding friction resistance in absence of the rock cohesion. Note that the cohesion value of the bedrock can not be confirmed based on available data.

Lateral Earth Pressure

Lateral earth pressure on the lock chamber walls is a significant design element. Initially, "at-rest" earth pressure is used for the design of concrete gravity sections since the deflection / movement at the top of the concrete gravity sections should be insignificant and not be able to initiate the active earth-pressure wedge under lateral earth pressure. However, in the event that the existing retaining structure starts into an unstable mode and some rotation / translation takes place, the lateral pressure state will be reduced to the "active" pressure state.

The assessment using "at-rest" pressures resulted in stability factors that did not meet the CDA Guidelines. Because, the Sliding Safety Factor (SSF) is less than one and the location of the resultant is outside of the base for the "at-rest" pressure case, potentially, the walls have entered into a rotation/translation failure mode. This could result in a lateral pressure reduction towards the "active" pressure state. Thus the walls were also assessed for the "active" pressures. Dynamic earth pressure due to seismic events was estimated and added into the calculation by using the Mononobe-Okabe formula. The maximum ground water table (GWT) in the backfill is at El. 228.2 m as provided by the readings from the ground water instrumentation. The properties of the backfill materials are defined in the geotechnical report (see Appendix A). Note that the stability results are sensitive to the GWT, unit weight and the soil internal friction angles. The parameters used in the calculations are based on a limited number of borehole samples. Thus, the soil properties and GWT may vary along the entire length of the walls. In addition, to date, the GWT has only been monitored for a limited period of time.

6.4.1.4 Acceptance Criteria

For cohesion to be zero, the 2007 CDA Dam Safety Guideline specifies for dam structures the following performance factors in Table 6.4-1.

TABLE 6.4-1

Loading Case	Load Combination			
	Usual	Extreme (IDF)	Extreme (Earthquake)	Post-Earthquake
Sliding Stability Factor (SSF)	1.5	1.1(1)	---(2)	1.1
Location of the Resultant	Within Middle-third	Within base	Within Base	Within Base
Normal Compressive Stress at Toe	< 0.3 x f'c	< 0.5 x f'c	< 0.9 x f'c	< 0.5 x f'c

(1) IDF established with floods event of magnitude in excess of 1:1000 years
(2) To establish post-earthquake condition

Technical Letter ETL1110-2-307 of the U.S. Army Corps of Engineers specifies flotation stability criteria for U-frame locks as shown in Table 6.4-2.

TABLE 6.4-2

Loading Case	Normal Operating Condition	Extreme Maintenance Condition
	Normal Upper Pool & Lower Pool in Lock Chamber	Normal Upper Pool & Lock Chamber Dewatered
Floatation Safety Factor (FSF)	1.5	1.1

6.4.1.5 Results of Stability Analysis

KGS Group has computed stability factors for the five load-combination cases specified in the 2007 CDA Dam Safety Guidelines for the dam structures. Table 6.4-3 shows the five load cases that were considered for the stability analysis of all the dam sections. **Load Case One** is related to maximum normal operation water level in summer. **Load Case Two** represents winter operation water level plus ice condition. **Load Case Three** is for the flood condition. **Load Cases Four and Five** are the loading cases for earthquake and post earthquake condition, respectively.

Tables 6.4-4 through 6.4-6 show the summary of the results of the stability analysis at the base of the concrete structure for the Gravity Bulkhead, Spillway, and Sluices.

The load cases for the Lock have been modified from those used for the dam sections to represent the appropriate Usual, Unusual, and Extreme operating conditions of Lock. The results are presented in Tables 6.4-7A and 6.4-7B. Table 6.4-8 presents the flotation results for the Lock structure.

For the sections that did not meet the stability requirements based on a shear friction angle of 36° and no cohesion (i.e., no tensile strength) at the bedrock / interface, Table 6.4-9 summarizes the required minimum contact shear strength parameters for the sections to meet the stability standards.

Since calculated normal compressive stresses at the toe of the structures are small and meet the acceptance criteria and no issues have been identified, for simplicity these stresses are not listed in the summary table. Detailed calculations are available in Appendix J.

TABLE 6.4-3
LOADING DATA

Data Type	LOADING CASES				
	USUAL		UNUSUAL	EXTREME	
	Case 1	Case 2	Case 3	Case 4	Case 5
Head Water Level (H.W.L) (m)	230.44	230.14	231.03	230.44	230.44
Tail Water Level (T.W.L) (m)	dry	dry	230.44	dry	dry
Ice Load (kN/m)	---	77	---	---	---
Seismic Coefficient	---	---	---	5%	---
Drag Force	---	---	Max. 84.2 kN @ EL. 229.3 m	---	---
Uplift	Full	Full	Full	Full	Full
Gate Position	Closed	Closed	Open	Closed	Closed

Legend:

Case 1: Summer Normal Maximum Operating Water Level

Case 2: Winter Normal Maximum Operating Water Level plus Ice

Case 3: Inflow Design Flood (IDF)

Case 4: Earthquake Loads in Conjunction with Usual Loading Case 1

Case 5: Post-Earthquake to Consider Modified Uplift Pressures Applied to the Cracked Section

TABLE 6.4-4
RESULT OF STABILITY ANALYSIS – GRAVITY BULKHEAD

Loading Case		USUAL		UNUSUAL	EXTREME	
		Case 1 (Summer Normal Water Level)	**Case 2** (Winter Normal Water Level + Ice)	**Case 3** (IDF)	**Case 4** (Normal Water Level plus EQ)	**Case 5** (Post EQ Condition)
Sliding Stability Factor (SSF)	CDA Required	**1.5**	**1.5**	**1.1**	**---**	**1.1**
	Computed	1.41	0.81	4.18	---	1.21
Location of the Resultant	CDA Required	**Within Mid-Third**	**Within Mid-Third**	**Within Base**	**Within Base**	**Within Base**
	Computed	Within Base	Outside of Base	Within Mid-Third	Within Base	Within Base
Location of the Resultant from Toe 'a' (m)		0.67	---	1.38	0.45	0.2

Height of Section =	7.32 m
Effective Base Length of Section =	3.05 m
Compressive Strength of Concrete f'c =	17 MPa
Tensile Strength of Concrete / Rock =	0.0 MPa
Internal Friction Angle Concrete / Rock =	36°
Cohesion Concrete / Rock, c =	0.0 kPa
Uplift =	100%

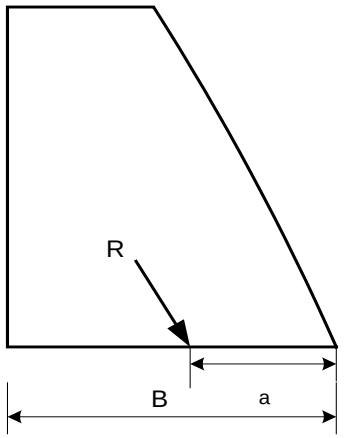


TABLE 6.4-5
RESULT OF STABILITY ANALYSIS – SPILLWAY

Loading Case		USUAL		UNUSUAL	EXTREME	
		Case 1 (Summer Normal Water Level)	**Case 2** (Winter Normal Water Level + Ice)	**Case 3** (IDF)	**Case 4** (Normal Water Level plus EQ)	**Case 5** (Post EQ Condition)
Sliding Stability Factor (SSF)	CDA Required	**1.5**	**1.5**	**1.1**	**---**	**1.1**
	Computed	1.06	0.67	2.35	---	0.93
Location of the Resultant	CDA Required	**Within Mid-Third**	**Within Mid-Third**	**Within Base**	**Within Base**	**Within Base**
	Computed	Within Base	Outside of Base	Within Base	Within Base	Outside of Base
Location of the Resultant from Toe 'a' (m)		0.3	---	1.14	0.23	---

Height of Section =	5.64 m
Effective Base Length of Section =	3.05 m
Compressive Strength of Concrete f'c =	17 MPa
Tensile Strength of Concrete / Rock =	0.0 MPa
Internal Friction Angle Concrete / Rock =	36°
Cohesion Concrete / Rock, c =	0.0 kPa
Uplift =	100%

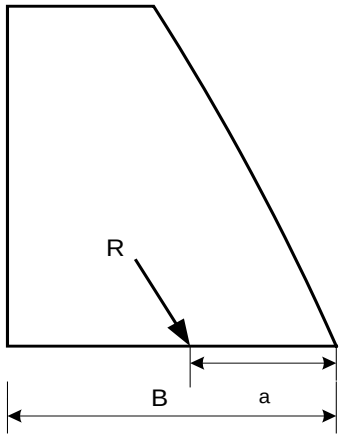


TABLE 6.4-6
RESULT OF STABILITY ANALYSIS – SLUICEWAY (INCLUDING PIERS AND ROLLWAY)

Loading Case		USUAL		UNUSUAL	EXTREME	
		Case 1 (Summer Normal Water Level)	**Case 2** (Winter Normal Water Level + Ice)	**Case 3** (IDF)	**Case 4** (Normal Water Level plus EQ)	**Case 5** (Post EQ Condition)
Sliding Stability Factor (SSF)	CDA Required	**1.5**	**1.5**	**1.1**	**---**	**1.1**
	Computed	1.70	0.92	4.33	---	1.61
Location of the Resultant	CDA Required	**Within Mid-Third**	**Within Mid-Third**	**Within Base**	**Within Base**	**Within Base**
	Computed	Within Base	Within Base	Within Mid-Third	Within Base	Within Base
Location of the Resultant from Toe 'a' (m)		2.31	0.71	3.08	1.94	2.28

Height of Section =	7.02 m
Effective Base Length of Section =	7.01 m
Compressive Strength of Concrete f'c =	17 MPa
Tensile Strength of Concrete / Rock =	0.0 MPa
Internal Friction Angle Concrete / Rock =	36°
Cohesion Concrete / Rock, c =	0.0 kPa
Uplift =	100%

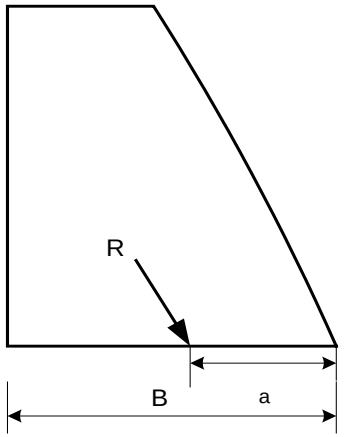


TABLE 6.4-7A

RESULT OF STABILITY ANALYSIS – LOCK # 39 CHAMBER MONOLITHS – BASED ON "AT-REST" EARTH PRESSURE

K_O=0.50

Loading Case		USUAL		UNUSUAL	EXTREME	
		Case 1 (Lock Operating at Upper Pool Level)	**Case 1.1** (Lock Operating at Lower Pool Level*)	**Case 1a** (Chamber Dewatered)	**Case 4** (Case 1 plus EQ)	**Case 4.1** (Case 1.1 plus EQ)
Sliding Stability Factor (SSF)	CDA Required	**1.5**	**1.5**	**1.3**	**1.1**	**1.1**
	Computed	2.2	0.97	0.8	1.5	0.9
Location of the Resultant	CDA Required	**Within Mid-Third**	**Within Mid-Third**	**Within Base**	**Within Base**	**Within Base**
	Computed	Within Base	Outside of Base	Outside of Base	Outside of Base	Outside of Base
Location of the Resultant from Toe 'a' (m)		0.47	---	---	---	---

*Lower Pool level = 226.04 (Same as the min. operating head at lock #40)
Height of Section = 7.01 m
Base Length of Section = 2.74 m
Compressive Strength of Concrete f'c = 17 MPa
Tensile Strength at Concrete lift Joints = 0.0 MPa
Internal Friction Angle at Concrete Joints = 41°
Cohesion at Concrete Lift Joints, c = 0.0 kPa
Uplift = 100%
Backfill Density = 18 kN/m3
Internal Friction Angle of Backfill = 28°
Water depth in Backfill above the Base = 4.07 m

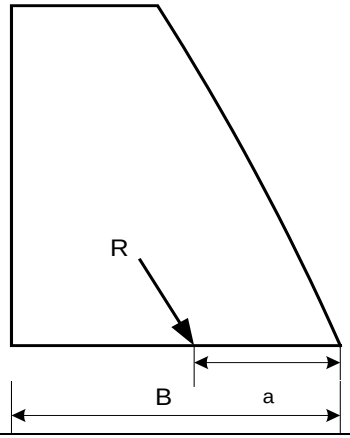


TABLE 6.4-7B

RESULT OF STABILITY ANALYSIS – LOCK # 39 CHAMBER MONOLITHS – BASED ON "ACTIVE" EARTH PRESSURE`

K_A=0.36

Loading Case		USUAL		UNUSUAL	EXTREME	
		Case 1 (Lock Operating at Upper Pool Level)	**Case 1.1** (Lock Operating at Lower Pool Level*)	**Case 1a** (Chamber Dewatered	**Case 4** (Case 1 plus EQ)	**Case 4.1** (Case 1.1 plus EQ)
Sliding Stability Factor (SSF)	CDA Required	**1.5**	**1.5**	**1.3**	**1.1**	**1.1**
	Computed	5.07	1.23	1.04	2.74	1.15
Location of the Resultant	CDA Required	**Within Mid-Third**	**Within Mid-Third**	**Within Base**	**Within Base**	**Within Base**
	Computed	Within Mid-Third	Outside of Base	Outside of Base	Within Base	Outside of Base
Location of the Resultant from Toe 'a' (m)		0.95	---	---	---	---

*Lower Pool level = 226.04 (Same as the min. operating head at lock #40)
Height of Section = 7.01 m
Base Length of Section = 2.74 m
Compressive Strength of Concrete f'c = 17 MPa
Tensile Strength at Concrete lift Joints = 0.0 MPa
Internal Friction Angle at Concrete Joints = 41°
Cohesion at Concrete Lift Joints, c = 0.0 kPa
Uplift = 100%
Backfill Density = 18 kN/m3
Internal Friction Angle of Backfill = 28°
Water depth in Backfill above the Base = 4.07 m

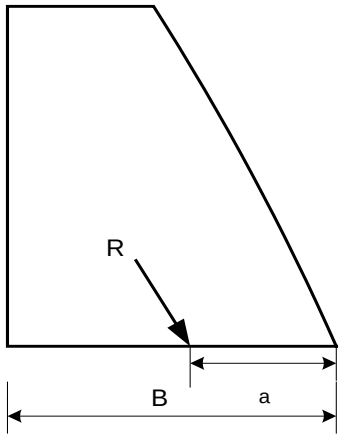


TABLE 6.4-8

RESULT OF FLOTATION ANALYSIS – U-FRAME LOCK CHAMBER MONOLITHS AT LOCK #39

Loading Case		**Normal Operating Condition** (Normal Upper Pool & Lower Pool in Lock Chamber)	**Extreme Maintenance Condition** (Normal Upper Pool & Lock Chamber Dewatered)
Floatation Safety Factor (FSF)	Required	**1.5**	**1.1**
	Computed	1.9	1.7

TABLE 6.4-9

MINIMUM REQUIRED CONTACT SHEAR STRENGTH PARAMETERS TO MEET STABILITY STANDARDS

Structure	Load Case	Minimum Required Contact Strength Parameters		
		Meets Req'd SSF & LOR (Friction Angle = 36°)	Meets Req'd SSF Only (Zero Cohesion)	
		Cohesion, kPa[/1/]	Friction Angle	Location of Resultant (LOR)
Gravity Bulkhead	Usual – Summer[/2/]	200	38°	Within Base
	Usual - Winter[/2/]	1090	53°	Outside of Base
Spillway	Usual - Summer[/2/]	320	46°	Within Base
	Usual - Winter[/2/]	1200	58°	Outside of Base
Sluiceway	Usual - Winter[/2/]	610	50°	Within Base
Lock Monoliths	Lock Operating at Lower Pool Level[/2/]	1130	42°	Outside of Base
	Lock Operating with Chamber Dewatered[/3/]	1160	42°	Outside of Base

[/1/] Cohesion is assumed to be equal to twice the tensile strength.

[/2/] Minimum required Sliding Safety Factor (SSF) ≥ 1.5 and Location of Resultant (LOR) must be within Mid-Third of Base.

[/3/] Minimum required sliding safety factor (SSF) ≥ 1.3 and Location of Resultant (LOR) must be within Base.

6.4.1.6 Discussion of Results

As previously noted, a reasonably achieved lower bound shear friction angle of 36° with zero cohesion (i.e., zero tensile strength) was used for the stability analyses. Sensitivity analyses were performed for the dam and lock sections that did not meet the stability standards using a 36° shear friction angle and zero cohesion to determine the minimum required shear strength parameters at the concrete / bedrock interface were determined. For all sections that did not meet the stability standards, the "location of the resultant" (i.e., overturning) requirement was not met and for some sections the sliding safety factor requirement was also not met. For the sections where the "location of resultant" requirement was not met, regardless of the shear friction angle, some tensile strength is required to meet the stability standards. The required tensile strengths presented in Table 6.4-9 are relatively high and are not likely to be confirmed as appropriate for the site conditions.

Ice Loading

Based on the CEATI ice load of 77.0 kN/m, the concrete dam did not meet the stability standards. Given that no obvious distress was observed at the structure during the inspection, it would appear that the structure is either still capable of carrying the design ice load (possibly with a minimal safety factor) or has not yet experienced an ice load of the assumed magnitude.

As a minimum, an ice monitoring program, and routine inspection programs are required to be implemented, and remedial measures should be carried out if any evidence of concrete cracking/distress or leakage is detected at the concrete/foundation interface. However, in summary, the dam structures do not have performance factors in accordance with the 2007 CDA Guidelines.

Gravity Bulkhead

The downstream backfill was assumed to contribute to the stability of the Gravity Bulkhead.

The results of the Gravity Bulkhead stability analysis show that the structure fails to meet CDA criteria for sliding stability and location of resultant (i.e., overturning) under the normal summer and winter operating loading cases.

Regardless of the shear friction angle, for the dam to meet the overturning requirements cohesion is required. The sensitivity analysis indicated that a minimum tensile strength σ_t equal to 545 kPa (and cohesion equal to 1090kPa) assuming Ø' = 36° is required. To only meet the sliding requirement, a minimum Ø' = 53° is required which is beyond the estimated upper bound for Ø'.

Given that the calculated sliding stability factor (SSF=0.8) for the ice loading case is far below the performance criteria, the concern of the dam safety under ice loading needs to be addressed. Remedial actions may be through ice control measures or post-tension anchoring, or at least, the concern can be mitigated (but not eliminated) through continued inspection and monitoring of the ice cover and its effects.

Spillway

The results of the Spillway stability analysis show that the structure fails to meet CDA criteria for sliding stability and location of resultant under the normal summer and winter operating loading cases and the post-EQ loading condition.

Regardless of the shear friction angle, for the dam to meet the overturning requirements cohesion is required. The sensitivity analysis indicated that a minimum tensile strength σ_t equal to 600 kPa (and cohesion equal to 1200 kPa) assuming Ø' = 36° is required. To only meet the sliding requirement, a minimum Ø' = 58° is required which is beyond the estimated upper bound for Ø'.

Given that the calculated sliding stability factors (SSF=1.06 and SSF=0.67) for summer and ice loading cases, respectively, are far below the performance criteria, the concern of the dam's safety needs to be addressed. Remedial actions may include installing post-tension anchors and/or adding additional mass to the structure.

Sluiceway

Because the drawings do not conclusively show the nature of the connections between the rollways and the piers, a full bay analysis was assumed to assess the sluice stability by combining the weights of concrete piers and the adjacent rollways. This assumption may be justified by the large ratio of depth to span of the rollways.

The results of the Sluiceway stability analysis show that the structure does not meet CDA criteria for sliding stability and the location of resultant under the winter operating loading case only.

Regardless of the shear friction angle, for the dam to meet the overturning requirement standards cohesion is required. The sensitivity analysis indicated that a minimum tensile strength σ_t equal to 305 kPa (i.e., cohesion equal to 610 kPa) assuming Ø' = 36° is required. To only meet the sliding requirement, a minimum Ø' = 50° is required which is beyond the estimated upper bound for Ø'.

Given that the calculated sliding stability factor (SSF=0.92) for the ice loading case is far below the performance criteria, the concern of the dam safety under ice loading needs to be addressed. Remedial actions may be through ice control measures or post-tension anchoring, or at least, the concern can be mitigated (but not eliminated) through continued inspection and monitoring of the ice cover and its effects.

U-Frame Lock Chamber Monoliths

The results of the Lock Chamber Monolith flotation stability analysis show that the structure meets USACE criteria.

The sliding and overturning stability was analyzed at the concrete joint between the chamber wall and the slab and the joint was assumed to be unbonded. The sliding stability analysis results for the walls of the U-Frame Lock Chamber Monoliths show that the walls do not meet CDA criteria for sliding stability and the location of resultant under normal operating condition at

the lower pool level and when the chamber is dewatered. The analysis found that the location of resultant is outside of the base.

In order for the location of resultant to meet stability standards, significant tensile strength is required at the concrete joint. Since no distress was identified at the lock chamber walls during the inspection, it may suggest that the assumption that the joint is unbonded is not valid. The following one or both actions are recommended:

- Carry out an investigation of the "as-built" condition of the Chamber Monoliths.
- Enhance the stability of the wall with tie-back anchors and/or by adding mass to the wall

6.4.2 Stoplog Design Review

Because the drawings do not show the stoplog size and the timber grade, the design review of the stop log was based on the site measurements and the assumption that the timber is a Select Grade Douglas Fir. This grade of timber was used for the stoplogs at Parks Canada's Elliot Falls Dam. The spans and locations of stoplogs can be found on Drawing No.: 12-0006-028 /1001 S03 provided in Appendix K. The shear and moment capacity for the 305 mm x 356 mm (12" x 14") wooden stoplogs was calculated according to "CSA Standard O86-01, Engineering Design in Wood".

KGS group calculated the shear and bending forces on the stoplogs for the following two load cases:

- Load case 1: Summer normal maximum operating water level at El. 230.34m.
- Load case 2: Winter normal operating water level with ice load.

The ice load was estimated to be 30 kN/m (2 kips/ft) for wooden stoplogs based on the 2011 OMNR Guidelines and OPG Standards.

In the calculation, the worst loading cases were considered for the bottom log under maximum summer hydrostatic pressure and for the upper stoplog subjected to the ice load. To calculate the shear and moment capacity of the stoplogs, one should consider the factors of load

duration, service condition and lumber size. Because the hydrostatic force of head pond is a permanent load versus the relative short term ice load, the different shear and moment capacity of the wooden stoplog was calculated for summer and winter loading cases accordingly in conformance to "CSA Standard O86-01, Engineering Design in Wood". The summary of the results are presented below in Table 6.4-10. Detailed calculations can be found in the Appendix K.

TABLE 6.4-10
STOPLOG ANALYSIS RESULTS

Components	Span	Load Case	Factored Shear / Moment Forces	Shear / Moment Capacity	Comments
Stoplogs - Sluiceway	7.92m	1 – Summer Case	Shear: 53.5 kN	Shear: 48.3 kN	x
			Moment: 105.8 kNm	Moment: 69.8 kNm	x
		2 – Winter Ice Case	Shear: 178.2 kN	Shear: 74.3kN	x
			Moment: 352.8 kNm	Moment: 107.4 kNm	x

"✓" and "x" denotes adequate and inadequate capacity, respectively.

The analysis results show that the bottom stoplog in the sluiceway has inadequate shear and bending capacity for the normal summer loading case. The stoplog was overstressed by 11 % for shear stress and 52% for bending stress.

Under ice loading condition, the stoplogs have inadequate shear and bending capacity as the stoplog was determined to be overstressed by 95% for the shear stress and 140% for the bending stress.

KGS Group was not able to inspect the condition of the bottom stoplog during the site visit since the bottom stoplog was covered by the rapidly flowing water. PCA should inspect the bottom stoplog to ensure that there are no signs of distress caused by shear and/or bending. A failure of the bottom log could only release limited amount of water and would not cause a severe flooding consequence but is a safety hazard for personnel in the vicinity of the structure. Regular inspection and monitoring of the stoplogs as an interim measure is deemed to be acceptable but the long term solution should be to replace the wooden stoplogs with steel logs.

At the time of the KGS Group site inspection, the upper six stoplogs of each bay were being stored on top of the deck. These stoplogs were inspected and no signs of distress were found that could be attributed to ice loads. This would indicate that the stoplogs are either capable of carrying the design ice load or that the stoplogs have not yet experienced the assumed ice loading. It should be note that the thermal ice loads can be greatly reduced when the ice sheet bridges across piers due to lateral deflection of the stoplogs that would occur before bending and shear failure in the ice loading event.

A failure of the bottom log could only release limited amount of water and would not cause a severe flooding consequence but is a safety hazard for personnel in the vicinity of the structure. Regular inspection and monitoring of the stoplogs as an interim measure is deemed to be acceptable but the long term solution should be to replace the wooden stoplogs with steel logs.

6.4.3 Lock Gate Design Review

The lock 39 gates are double-leaf miter gates framed horizontally. The horizontal straight timbers acting as beams are in effect a series of three-hinged arches which transmit the water pressures to the lock walls through the quoin hinges. They are subjected to combined bending and compressive stresses.

In general, the loads applicable to lock miter gate design are dead, hydrostatic, ice, temporal, boat impact and torsion. Dead load is the weight of the structure plus mud and ice; hydrostatic load is the water load on the gate produced by the pool differential; ice load is the expansive force which occurs in the ice sheet during a rapid warming trend; boat impact is the dynamic force applied to the gate by the barge impact; temporal load is the water surge forces from wave loads or overfilling of the lock; and torsion is the result of a twisting action from the operating strut force and the water resistance caused by the leaf moving.

As discussed with PWGSC, the lock gate design review was limited to a cursory structural assessment for the horizontal straight timbers and frames forming the valve openings for the most structurally critical gate, which is the downstream gate. The critical loading conditions consider for normal summer operating condition and winter ice condition are as follows:

- Load case 1: Summer normal maximum operating head water level at El. 230.44 m and related tailwater at El. 226.04 m.
- Load case 2: Winter normal operating water level at El. 230.14 m with ice load and related tailwater to be dry.

The ice load was assumed to be 30 kN/m (2 kips/ft) for timber logs based on the 2011 OMNR Guidelines and OPG Standards.

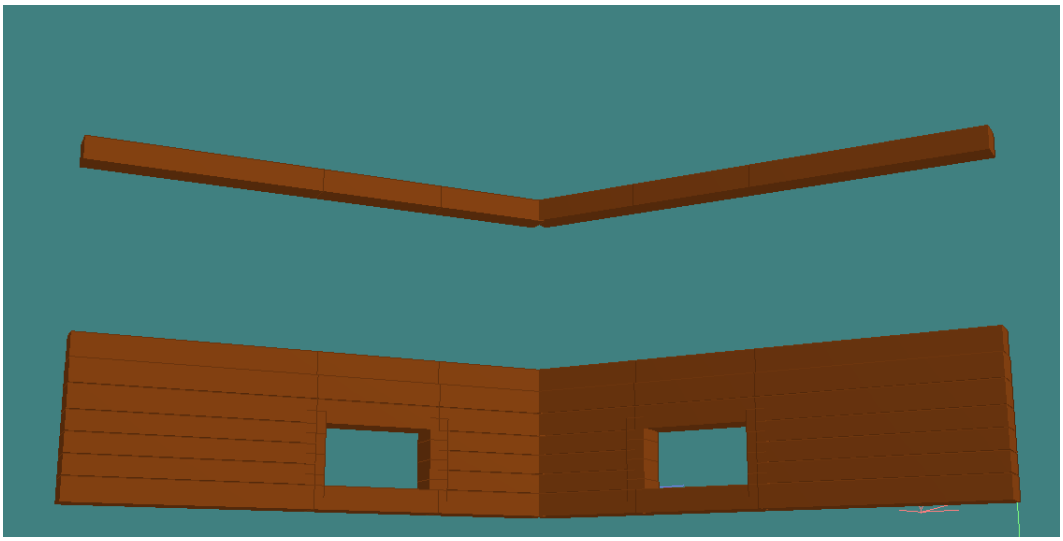
Dimensions and elevations used for the design review of the downstream lock gate is as follows:

- Sill elevation: 224.23 m
- Gate width (c/c of quoin hinges): 10.82 m
- Gate height: 6.72 m.

The thicknesses of the gate-leaves vary. The major portion of the gate-leaves is 406 mm (12”) think except a few of timbers at bottom of the gate have a thickness of 457 mm (18”). In the analyses, the structural assessment was performed for the critical horizontal straight timbers with cross-sectional sizes of 305 mm x 406 mm (12”x16”) and 305mm x 457mm (12”x18”), respectively. The details of the gate-leaves are as shown on the PCA Drawing No.: TSW 5405-G. No information was available for the timber grade, thus in discussion with PCA engineers, it was agreed to assume the timber to be Select Grade Douglas Fir.

A 3D model of the gate is developed in STAAD Pro as shown in Figure 6.4.3-1 .The horizontal timbers and valve opening frame are modeled as beam elements to withstand the applicable loadings.

FIGURE 6.4.3-1
3D MODEL OF THE CRITICAL TIMBERS OF THE LOCK GATE



The design review was carried out using a limit states design approach. In this approach, the structural timbers are designed to have sufficient strength, such that factored resistance ≥ effect of factored loads. The factored resistance and the factored load effects were determined in accordance with "CSA Standard O86-09, Engineering Design in Wood". The following critical loading cases were analyzed:

- hydrostatic pressure for the lower portion of the gate-leave; and
- ice loading for the upper timbers..

To calculate the shear and moment capacities of the timbers, the factors of load duration, service condition and timber size were taken into consideration. In the calculations for the shear and moment capacities of the timbers, the hydrostatic pressure of head pond was taken to be a permanent duration loading case and the ice to be a standard duration loading case. Thus, as noted in Table 6.4-11, the structural capacities for the hydrostatic loading case are different from those for the ice loading case. The summary of the results are presented below in Table 6.4-11. Detailed calculations can be found in the Appendix K.

TABLE 6.4-11
GATE ANALYSIS RESULTS

Components	Load Case	Factored Forces	Structural Capacity	Comments
Normal Timbers (305mm x 406mm) (h x b)	1 – Hydrostatic Loading Case	Shear: 61.0 kN	Shear: 68.8 kN	✓
		Moment: 93.0 kNm	Moment: 90.8 kNm	x
		Axial: 117.2 kN	Axial: 900.2 kNm	✓
		Resistance to Combined Bending and Axial Load: $M_f / M_r + P_f / P_r$ = 1.19 (≤ 1.0 required)		x
	2 –Ice Loading Case	Shear: 137.3 kN	Shear: 105.9 kN	x
		Moment: 209.0 kNm	Moment: 139.7 kNm	x
		Axial: 263.7 kN	Axial: 1311.0 kNm	✓
		Resistance to Combined Bending and Axial Load: $M_f / M_r + P_f / P_r$ = 1.70 (≤1.0 required)		x
Bottom Timbers around Valve Opening (305mm x 457mm) (h x b)	1 – Hydrostatic Loading Case	Shear: 129.1 kN	Shear: 73.4 kN	x
		Moment: 231.4 kNm	Moment: 109.0 kNm	x
		Axial: 309.9 kN	Axial: 1046.0 kNm	✓
		Resistance to Combined Bending and Axial Load: $M_f / M_r + P_f / P_r$ = 2.42 (≤1.0 required)		x

"✓" and "x" denotes adequate and inadequate capacity, respectively.

The results indicate that the 305 mm x 406 mm timbers of the gate have adequate shear, and compressive capacities against hydrostatic loading but the timbers were overstressed by 2% for the bending stress and 15% for the combined bending and compressive stress.

Under ice loading, the 305 mm x 406 mm timbers of the upper portion of the gate may have inadequate shear and bending capacity since the timbers were overstressed by 30% for shear stress and 50% for bending stress. The timbers behave as a three-hinged arch, and based on a 30 kN ice load, the timbers were calculated to deflect approximately 40 mm. This amount of deflection may be much larger than the thermal expansion of the ice at the gate producing the ice load. Thus, because of the ability of the gates to deflect, the actual ice load against gate may be less than 30 kN. Because there is no evidence of damage to the gate to-date, regular inspection and monitoring of the timbers during the winter season to identify any distress of the upper timbers is considered to be adequate.

The analysis results also show that the 305 mm x 457 mm horizontal timbers below and above the valves of the gate have inadequate shear and bending capacities for the hydrostatic loading. The timbers were overstressed by 76% for shear stress and 120% for bending stress. It appears that these timbers are under designed with respect to their structural capacities based on the current wood design code. It is recommended that PWGSC / PCA consider the following options:

- replace the timber gates with steel gates; or
- accept the non-compliance and monitor.

6.4.4 Structural – Conclusions and Recommendations

Some structures were found to be deficient in stability or structural capacity and require remedial action or as a minimum monitoring. Recommended remedial options are provided at the end of the report in Section 9.3.

6.5 EARTHFILL EMBANKMENTS SLOPE STABILITY ANALYSIS

The stability analysis completed under the DSR for the earthfill embankments has focused on the Right and Left Earthfill Embankments at Talbot River Dam and Northwest and Southeast Canal Earthfill Embankments, which are the more critical geotechnical structures. The stability conditions of the earthfill embankments on both sides of the Lock 39 walls are less critical geotechnically with the wide crest (12 to 15 m) and low height. Discussions on the stability conditions for Lock 39 are also provided in the following sections.

As part of this DSR, a computer-aided geotechnical stability analysis was completed on the embankments to estimate the factors of safety (FOS) against slope failure and to determine if the required design criteria under the various operating conditions have been satisfied.

The geotechnical stability analysis was completed on one section of each embankment using the limit equilibrium computer model SLOPE/W, which is part of the GeoStudio 2007 software program. All calculations were run using effective stress analyses with the Morgenstern-Price

Method of slices assuming a half-sine function for the inter-slice forces to determine the estimated factor of safety against failure to satisfy both force and moment equilibrium.

6.5.1 Design Cases Considered

The key design cases that were assessed to determine whether the stability requirements of the 2007 CDA Guidelines are met include the normal long-term conditions under steady-state seepage and more extreme conditions under seismic loads, short-term IDF reservoir surcharge and short-term instantaneous rapid drawdown of the upstream slope. For the normal condition, both the summer and winter cases were considered to determine the most critical conditions relative to stability. For the earthquake case, a pseudo-static analysis was performed using the established seismic parameters to estimate the stability under the design earthquake.

The transition from summer operating levels to winter operating levels potentially could happen over a short period of time (approximately in a day), therefore, a condition was assessed from normal operating level to the winter level. However, the PCA standard dam drawdown operation procedures revealed that transition from the normal operating level to the winter level happens gradually over a long period of time (i.e., several weeks).

For the IDF condition, the water level is higher than the concrete core walls of the Right and Left Earthfill Embankments at Talbot River Dam. As a result, the performance of these embankments would not be satisfactory for the IDF condition and significant damage/ wash out of the embankment fill would potentially occur. Further discussion on the IDF condition is provided in the following section.

The End-of-Construction case is normally applicable for the short term, and is not relevant for the current DSR stability evaluation as the embankments were constructed approximately 100 years ago and any construction-induced porewater pressures should have dissipated within the existing soils.

6.5.2 Assumptions for Analysis

6.5.2.1 Talbot River Dam

The cross sections with interpreted stratigraphy through the Left and Right Earthfill Embankments at Talbot River Dam and used for the slope stability evaluation are shown on Figures 6.5.2-1 and 6.5.2-2. The geometry was based on the topographic survey from November 2012 (Sections B and C shown on DWG. No: G02). These are near the maximum height of the embankments to the existing ground surface based on the topographic survey and background information. The stratigraphy of the embankments was based on the findings from the 2013 geotechnical site investigation and background information.

The piezometric level shown in the figures reflects the measured groundwater levels from VW and standpipe piezometers. At the Right Earthfill Embankment, the monitored water level at the crest on the downstream side has typically been approximately 2.3 m below the headpond level, showing considerable head loss across the concrete core wall which appears to be providing a reasonable barrier to limit seepage. The piezometric level was extended to the toe of the embankment as no wet conditions or seepage discharge was observed on the downstream slope during the site inspection and the soil investigation. A similar piezometric condition was assumed for the Left Earthfill Embankment given that both embankments have a similar concrete core wall as a seepage barrier and stratigraphic and observed seepage conditions are comparable.

For the hypothetical rapid drawdown analyses, the reservoir was considered to be lowered from the normal operating level (230.44 m) to winter level (228.54 m). During rapid drawdown operation, it was assumed that the embankment piezometric level dropped approximately 0.3 m due to assuming some drainage.

FIGURE 6.5.2-1
SIMPLIFIED STRATIGRAPHIC MODEL USED FOR RIGHT EARTHFILL EMBANKMENT SLOPE STABILITY EVALUATION

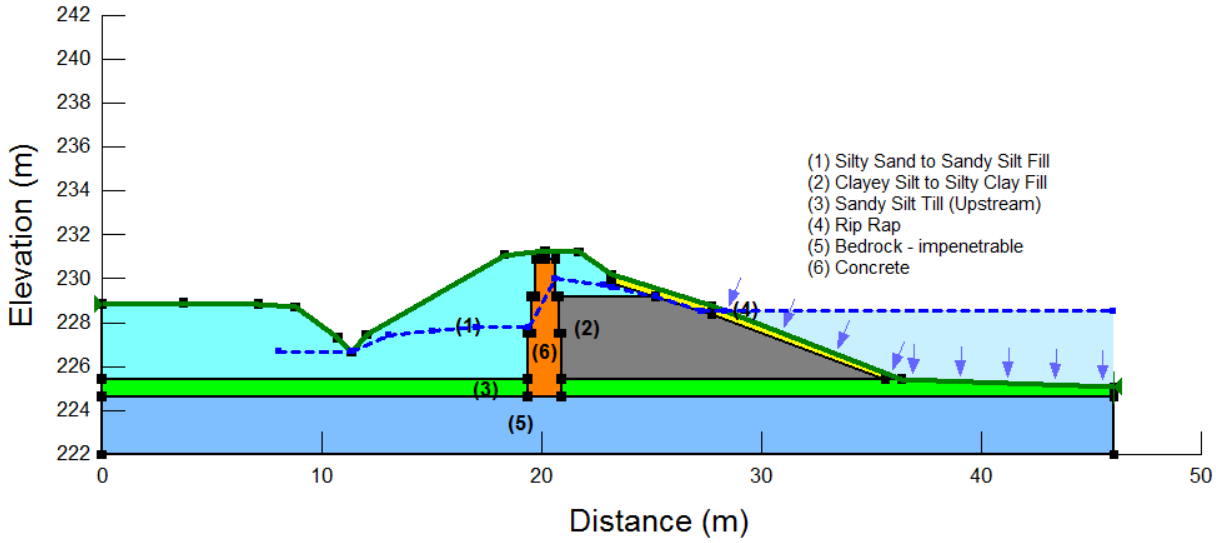
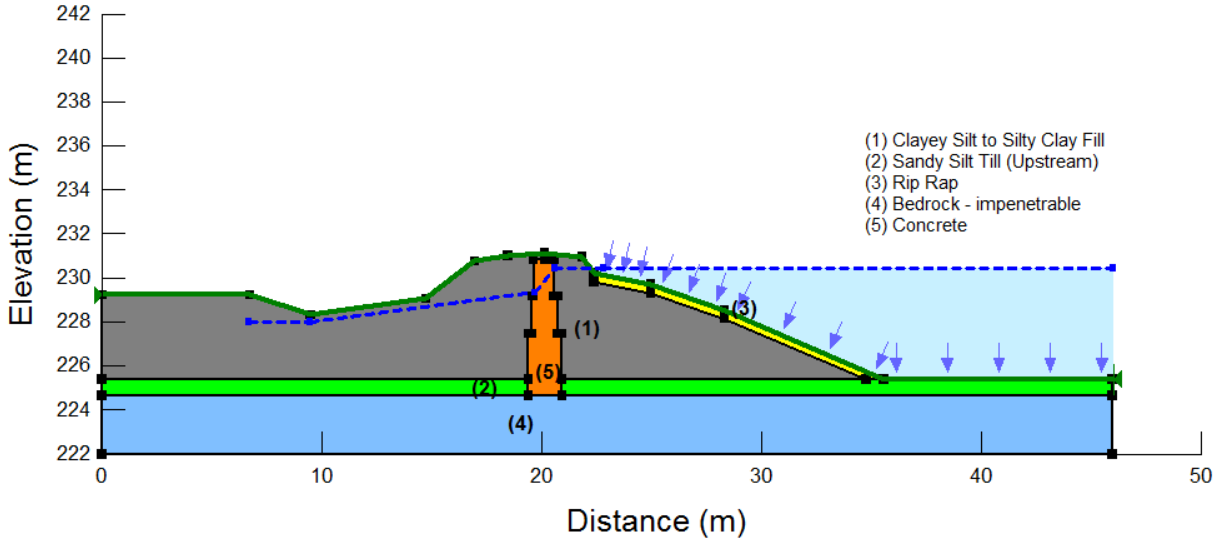


FIGURE 6.5.2-2
SIMPLIFIED STRATIGRAPHIC MODEL USED FOR LEFT EARTHFILL EMBANKMENT SLOPE STABILITY EVALUATION



6.5.2.2 Canal Embankments

The cross sections with stratigraphy through the Northwest and Southeast Canal Earthfill Embankments at Talbot Canal are shown on Figures 6.5.2-3 and 6.5.2-4.. The geometry was based on the topographic survey from November 2012 (Section STA.0+150 on DWG. No: G04

and Section STA.0+550 on DWG. No: G05). These are near the typical maximum height of the embankments to the existing ground surface based on the topography survey and background information. The simplified stratigraphy was based on the 2013 geotechnical site investigation and background information.

At the Northwest Canal Earthfill Embankment, the monitored water level at the crest of the downstream side has typically been approximately 1 m below the headpond level, showing some head loss across the clayey silt to silty clay embankment fill which appears to be providing a reasonable head loss and control the seepage. The piezometric level was extended near to the toe of the embankment as no wet conditions or seepage discharge was observed at the downstream slope and the toe areas during the inspection and the soil investigation. A similar piezometric condition is assumed for the Southeast Canal Earthfill Embankment given that both embankments consisted of very similar type embankment fill and have performed comparably.

FIGURE 6.5.2-3
SIMPLIFIED STRATIGRAPHIC MODEL USED FOR NORTHWEST CANAL EARTHFILL EMBANKMENT SLOPE STABILITY EVALUATION

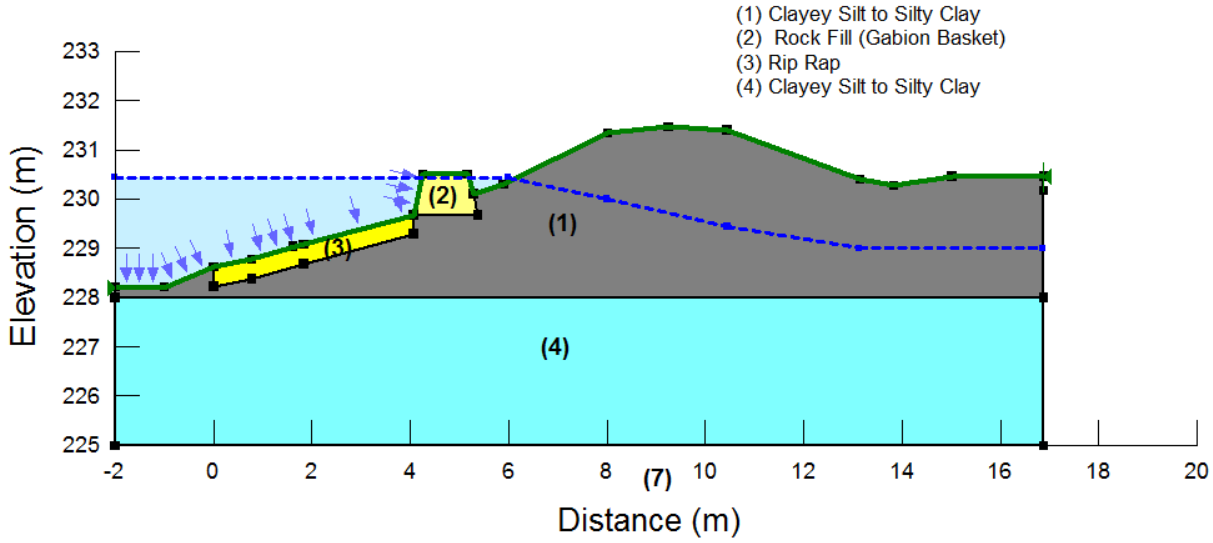
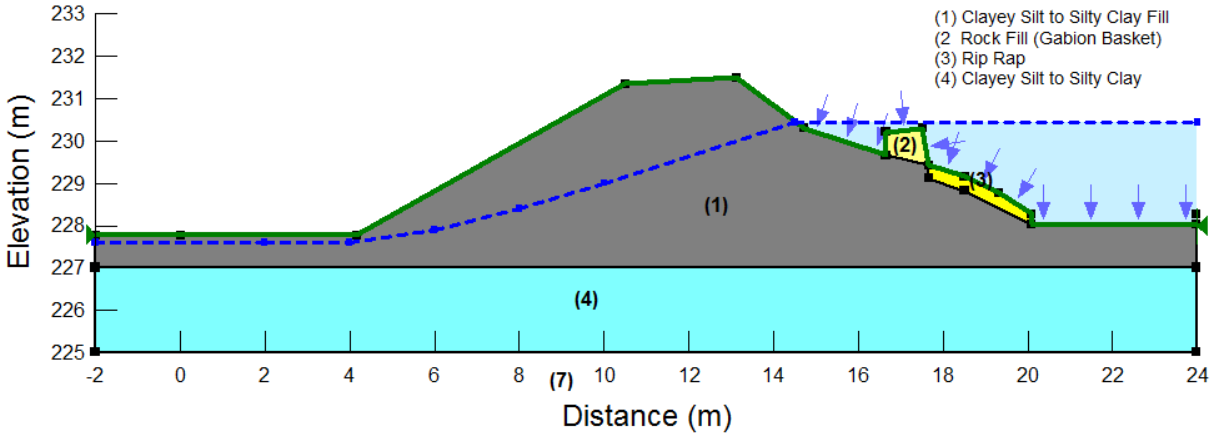


FIGURE 6.5.2-4
SIMPLIFIED STRATIGRAPHIC MODEL USED FOR SOUTHEAST CANAL EARTHFILL EMBANKMENT SLOPE STABILITY EVALUATION



6.5.2.3 Lock 39 Embankments

Based on the topographic survey and site inspection, the embankments for the Lock 39 walls are relatively low in height (less than 3 m), and the embankment crests extending back from the walls are quite wide (typically 12 to 15 m). In addition, the outer slopes on the embankments were approximately at 2.5H: 1V. The cross section of the Lock 39 embankments is shown on Section A in Drawing No G06. The stability conditions of the lock walls and connected base slab into the channel were evaluated in Section 6.4 and are not addressed herein the geotechnical evaluation.

Detailed geotechnical analyses on the outer slopes of the lock embankments were not considered necessary. The Lock 39 earthfill embankments have performed well and no visual in-stability has been observed on these outer slopes. Given the low height and wide crest width, potential slip surfaces that could affect the lock walls have fairly high estimated safety factors, which well exceed the minimum design criteria. In the event that some shallow sloughing or movement occurs on the downstream / outside on the downstream slope, it would not immediately impact the performance of the lock walls. Some maintenance would be required to ensure that the movement did not progress further, however there would be sufficient time to address any maintenance required. Note that ongoing inspection of these slopes is warranted to ensure they continue to perform acceptably.

6.5.3 Results of Stability Analyses

6.5.3.1 Talbot River Dam

The most critical potential slip surfaces for the Right and Left earthfill embankments that could affect their water retaining performance is a deeper-seated surface where overall slope movement could compromise support for the concrete core wall. Shallow potential slip surfaces of the outer slopes were also investigated, although they will have little potential impact on the water retaining ability of the embankment, particularly in the short term. If shallow failures did occur, they are considered more of a nuisance-type movement that could be addressed with maintenance before there could be any negative impacts to the performance of the dam. The intact vegetation at downstream side helps to maintain the surficial stability of the embankment and protect against shallow instability. The Right and Left Earthfill Embankments of the Talbot River Dam have both performed reasonably well, and no major deficiencies related to stability performance have been observed. The exception is the area with the exposed core wall on the Right Earthfill Embankment where erosion and / or shallow sloughing has been observed.

A summary of the results of the analyses using the representative upper and lower bound material properties presented in Section 4.4.3 is given in Table 6.5-1. The sensitivity analysis on the upper and lower bounds of shear strength were performed to provide a sense of the ranges in estimated FS that likely represent the actual conditions at site. Discussion on the stability conditions is provided below. Summary figures showing the key slope stability analyses results are presented in Appendix L.

TABLE 6.5-1
RESULTS OF SLOPE STABILITY ANALYSES – RIGHT AND LEFT EARTHFILL EMBANKMENTS

Case	Description	Reservoir Level (m)	Slope	Minimum Required FOS	Estimated Factor of Safety (Lower bound material properties)	Estimated Factor of Safety (Upper bound material properties)
Right Earthfill Embankment						
I	Normal operating Condition (Long-term Steady State Seepage)	230.44	U/S	1.5	1.43	1.61
			D/S	1.5	1.20	1.30
II	Extreme Short-term Earthquake Loading /1/	230.44	U/S	1.0	1.10	1.25
			D/S	1.0	1.09	1.18
III	Rapid Drawdown/2/	228.54	U/S	1.2	1.12	1.26
iv	IDF Condition/3/	231.03	U/S	1.2	1.48	1.67
			D/S	1.2	0.87	0.95
Left Earthfill Embankment						
I	Normal operating Condition (Long-term Steady State Seepage)	230.44	U/S	1.5	1.44	1.63
			D/S	1.5	1.62	1.83
II	Extreme Short-term Earthquake Loading /1/	230.44	U/S	1.0	1.10	1.25
			D/S	1.0	1.43	1.62
III	Rapid Drawdown /2/	228.54	U/S	1.2	1.08	1.23
iv	IDF Condition/3/	231.03	U/S	1.2	1.49	1.69
			D/S	1.2	0.80	0.91

Notes:
/1/ 0.05g PGA was used for the analyses (see Section 6.3). See Table 4.4-1 for the upper and lower bound geotechnical material properties.
/2/ The Rapid Drawdown case is only hypothetical and not applicable as long as PCA continues to do their drawdown from summer to winter water levels over a long period of time.
/3/ Concrete core wall will be overtopped resulting in potential damage/washout of the earthfill.

6.5.4 Discussion on Stability (Right and Left Earthfill Embankments)

At Right Earthfill Embankment, the stability analysis resulted in estimated safety factors greater than unity for all cases with consideration of the lower bound of the embankment material shear strengths with an exception of the short term IDF Condition. The stability criteria is only satisfied for the earthquake loading assuming the lower bound shear strength. However, the estimated safety factor at the upstream side (1.43) and at the downstream side (1.2) are greater than 1.0 but do not meet the recommended minimum 1.5 factor of safety. The stability criteria is generally satisfied with the exception of the normal conditions only at the downstream side assuming the upper bound materials shear strengths. The estimated safety factor for the normal conditions at the downstream side is still moderately greater than 1.0.

At the Left Earthfill Embankment, the stability analysis resulted in estimated safety factors greater than unity for all cases with an exception of the short term IDF condition assuming the lower bound of the embankment material shear strengths. The stability criteria is generally satisfied with the exception of the normal condition at upstream side and IDF condition assuming the lower bound material shear strengths. The estimated safety factor is also reasonably high (1.44). The embankment stability criteria is satisfied for all the cases except the IDF condition with upper bound shear strength materials properties.

It should also be noted that current analysis was performed for the deepest section of the embankments. Therefore, slightly higher factor of safeties may be expected on the shallow section of the embankment. The slope has performed well with no evidence of previous deep seated slope failures over the life of the structure (approximately 100 years) with the exception of the exposed concrete core wall due to shallow sloughing or erosion at the upper portion of the slope. The condition of the slope below water is unknown at the concrete exposed core wall area where there was evidence of previous movement at the upper portion of the slope (shallow sloughing), although there did not appear to be any recent significant slope movements. The repair of this portion of the embankment should be initiated on a medium priority. This repair may require a bathymetric survey and detailed stability analyses.

Considering that there has been no evidence of significant deep seated slope movements that would suggest concerns to the overall stability performance of the embankments, surveillance

and monitoring could be considered to confirm the performance of the overall slope through both visual inspection of the embankment and continued monitoring of the instrumentation to ensure that elevated porewater pressures are not observed. Remedial work to improve the estimated factor of safeties to satisfy the stability design criteria should be considered as a medium priority.

6.5.4.1 Canal Embankments

The most critical potential slip surfaces for the canal embankments are deep-seated where overall slope movement could affect their water retaining ability. Shallow potential slip surfaces of the outer earthfill slope were also investigated, although they will have less potential impact on the overall embankment performance, particularly in the short term. Note that there has been some ongoing shallow sloughing of the inner (wet side) slopes with the erosion and movement of the gabion baskets.

The downstream (dry side) slopes of the Northwest and Southeast Canal Earthfill Embankments have remained functional and effective in containing the canal water. There have been no significant instability or deficiencies related to their performance observed. However, loss of the upper slope, deterioration of the gabion baskets, and erosion of the inner (wet) slope has been ongoing.

A summary of the results of the analyses using the representative upper and lower bound material properties presented in Section 4.4.3 is presented in Table 6.5-2. The sensitivity analysis on the upper and lower bounds of shear strength were performed to provide a sense of the ranges in estimated slope stability factor of safeties that likely represent the actual conditions at site. The slope stability analyses results are presented in Appendix L.

TABLE 6.5-2
RESULTS OF SLOPE STABILITY ANALYSES – NORTHWEST AND SOUTHEAST CANAL EARTHFILL EMBANKMENTS

Case	Description	Reservoir Level (m)	Slope	Minimum Required FOS	Estimated Factor of Safety (Lower bound material properties)	Estimated Factor of Safety (Upper bound material properties)
Northwest Canal Earthfill Embankment						
I	Normal operating Condition (Long-term Steady State Seepage)	230.44	U/S	1.5	1.45	1.68
			D/S	1.5	2.17	2.45
II	Extreme Short-term Earthquake Loading[/1/]	230.44	U/S	1.0	1.16	1.31
			D/S	1.0	1.84	2.09
III	Rapid Drawdown[/2/]	228.54	U/S	1.2	1.12	1.27
IV	IDF Condition	231.03	U/S	1.2	1.56	1.76
			D/S	1.2	1.52	1.75
Southeast Canal Earthfill Embankment (STA. 0+450 to STA 0+0+950)						
I	Normal operating Condition (Long-term Steady State Seepage)	230.44	U/S	1.5	1.29	1.46
			D/S	1.5	1.12	1.27
II	Extreme Short-term Earthquake Loading[/1/]	230.44	U/S	1.0	1.03	1.17
			D/S	1.0	1.01	1.14
III	Rapid Drawdown[/2/]	228.54	U/S	1.2	0.91	1.02
IV	IDF Condition	231.03	U/S	1.2	1.38	1.56
			D/S	1.2	1.02	1.16

Notes:
[/1/] 0.05g PGA was used for the analyses (see Section 6.3). See Table 6.3.2 for the upper and lower bound geotechnical material properties.
[/2/] The Rapid Drawdown case is only hypothetical and not applicable as long as PCA continues to do their drawdown from summer to winter water levels over a long period of time.

6.5.5 Discussion on Stability (Canal Embankments)

At the Northwest Canal Earthfill Embankment, the stability analysis resulted in estimated safety factors greater than unity for all cases with consideration of the lower bound of the embankment material shear strengths. The stability criteria is generally satisfied with the exception of the rapid drawdown case at the upstream side for assumed lower bound shear strength. However, the estimated safety factor for normal condition is fairly high (1.45) and above unity (1.08) for short-term rapid drawdown but does not meet the recommended minimum (1.5) for normal condition and (1.2) for rapid drawdown condition. The stability criteria is generally satisfied for all the cases for assuming the upper bound materials shear strengths.

At the Southeast Canal Earthfill Embankment, the stability analysis resulted in estimated safety factors greater than unity for all cases with consideration of the lower bound of the embankment material shear strengths with an exception of the rapid drawdown case less than unity (FS=0.91). However, the rapid drawdown case may not be applicable as described in Section 6.5.1). The stability criteria is only satisfied for the earthquake loading for assuming the lower and upper bound material properties. The estimated safety factor for normal condition is fairly good at upstream (1.29) and low at downstream (1.12) for the normal condition for lower bound materials properties and does not meet the recommended minimum (1.5). The estimated safety factor for normal condition is fairly high at upstream (1.46) and fairly good at downstream (1.27) for the normal condition for upper bound materials properties and does not meet the recommended minimum (1.5).

It should also be noted that current analysis was performed for the deepest section of the embankments. Therefore, higher factor of safeties may be expected on the shallow sections of the embankment. The Southeast Canal Embankment is relatively low in height (less than 2 m) approximately between STA. 0+000 to STA. 0+450 and STA. 1+050 to STA. 1+350. The ground level in these areas beyond the toe of the downstream slope was approximately at the EL. 230 to 231 m, which is near or above the normal operating level of 230.44 m. Therefore there has very little to no differential head across the embankment. Given that the low height and very little differential head, potential slip surfaces that could affect the embankment performance have fairly high estimated safety factors for the downstream slope, which satisfies the minimum design criteria.

At Southeast Canal Earthfill Embankment, downstream slope at the concrete structure located near the middle of the canal has a maximum height of approximately 4.2 m. The stability conditions of this concrete section of the earthfill embankment of the concrete walls are less critical geotechnically with the wide crest (9 to 10 m). Potential slip surfaces that could affect the stability of the concrete wall have fairly high estimated safety factors, which well exceed the minimum design criteria. In the event that some shallow sloughing or movement occurs on the downstream slope, it would not immediately impact the performance of the canal/ concrete walls.

The upstream (wet side) canal slopes are generally in poor to fair condition due to loss of the upper slope, deterioration of the gabion baskets, and significant erosion of the inner (wet) slope. Ongoing erosion on upstream slope (wet side) is anticipated to be a relatively slow process as there are no significant waves except the waves associated with boat traffic. Repair of the canal slope should be considered as a medium priority. The overall risk of breach of the canal embankment is relatively low as considering relatively low height of the embankments. There would be a sufficient reaction time for repair although the embankment cannot be considered as stable for a long-term without repair. Inspection and monitoring of the canal embankments should be increased until the repairs are initiated. A few disturbances in the Southeast Canal Embankment were also observed between STA. 0+050 and STA. 0+250 (see Section 4.4.2, Photos 4.4.2-4 and 4.4.2-5). These disturbances appear to have been man-made. Their condition should be monitored until the repair is initiated.

Remedial work to improve the estimated factor of safeties to satisfy the stability design criteria should be considered as a medium priority where the embankment is relatively high and steeper slope approximately between STA. 0+450 to STA 0+0+950 at Southeast Canal Earthfill Embankment.

IDF Condition

The estimated water level during IDF is 231.03 m. During IDF, the water level will be very near to the crest of the embankments at some locations at the canal and the Talbot River Dam and do not have enough free board. The crest of the Talbot River Dam and Talbot Canal embankments were surveyed between 231.10 to 231.35 m. It should also be noted that the IDF

water level is higher than the top of the concrete core wall at Talbot River Dam as the top of the core wall elevation is at EL. 230.85 m. At Right and Left Earthfill Embankments at Talbot River Dam, the stability analysis resulted in estimated safety factors less than unity for the IDF condition at downstream side. The performance of the Talbot Dam earthfill embankments may not be satisfactory under IDF condition and possible significant damage/wash out of the embankment fill may occur. Remedial work to Increase the concrete core wall to the elevation of EL. 231.15 should be considered as a medium priority.

For the Northwest earthfill embankments at the canal, the stability analysis resulted in estimated safety factors greater than the CDA required 1.2 minimum FOS for the IDF condition. For the Southeast canal embankment, the estimated safety factor for the upstream slope exceeds the CDA minimum required FOS for the IDF condition and the downstream slope does not but the estimated FOS is greater than unity.

6.5.6 Earth Embankments Stability Analyses – Conclusions and Recommendations

- The estimated upper and lower bound shear strength parameters of the soils are based on the 2013 KGS Group geotechnical field investigation program (see Appendix A). Based on the estimated shear strength parameters all of the earthfill embankments meet the stability requirements for the “earthquake” loading condition. Some embankments do not meet the stability requirements for the “normal” loading condition but the FOSs are greater than 1.0.
- For the assumed rapid drawdown condition, all of the embankments did not meet the stability requirements, but the FOSs were greater than 1.0 except for the Southeast canal embankment based on the lower bound estimated material properties. Currently, based on the PCA operation procedures, the rapid drawdown condition is not applicable. PCA should ensure that the operation procedures continue to not rapidly draw down the water levels.
- The performance of the Talbot Dam earthfill embankments are not satisfactory under the IDF condition for two reasons:
    1. The IDF water level is higher than the top of the concrete corewall and therefore during IDF conditions significant damage/wash out of the embankment fill may occur.
    2. The estimated slope stability factor of safeties do not meet the CDA required 1.2 factor of safety and are less than unity.

- The Northwest canal embankment meets the CDA slope stability requirement for the IDF condition but the .downstream slope of the Southeast canal embankment does not, although it is greater than unity.
- As a minimum, the stability of the slope embankments should be monitored but PCA should consider addressing the stability deficiencies prior to the next DSR.
- The vegetation on the canal embankments should be removed (as a medium priority) to improve access for inspection and identification of any movement or need for remedial works.
- Detailed geotechnical analyses on the outer slopes of the lock embankments were not considered necessary. The Lock 39 earthfill embankments have performed well and no visual in-stability has been observed on these outer slopes. Given the low height and wide crest width, potential slip surfaces that could affect the lock walls have fairly high estimated safety factors, which well exceed the minimum recommended CDA stability criteria.

6.6 OTHER FAILURE MODES

In addition to the stability consideration of the structures and embankments, KGS Group has reviewed and assessed other potential failure modes and the adequacy of the design, construction and operation of the dam and lock structures to address these failure modes. The conditions and factors related to the consequences of a dam failure from each of the failure modes have also been reviewed. The potential failure modes are categorized based upon consideration of a number of factors including: the likelihood of occurrence under the given loads, the present degree of awareness of the condition, and potential impacts of a failure by that mode based upon load probability, presence of a feature or condition that makes the potential failure mode a physical possibility, the performance record related to that failure mode, and the magnitude of potential adverse consequences. For each of the potential failure modes identified, based on engineering judgment, the potential failure mode has been categorized according to the following system:

Category I – Highlighted Potential Failure Modes: Those potential failure modes of greatest significance considering need for awareness, potential for occurrence, magnitude of consequences, and likelihood of adverse response (physical possibility was evident, fundamental flaw or weakness was identified, and conditions and events leading to failure

seemed reasonable and credible) are highlighted. Development of the potential failure mode could be in progress, or the likelihood of initiation could be high such that enhanced monitoring or other risk mitigation measures should be considered.

Category II – Potential Failure Modes Considered But Not Highlighted: These are judged to be physically plausible and credible but are of lesser significance and likelihood than Category I. Continued awareness and some form of monitoring of these potential failure modes is usually advisable. Each failure mode is described and included with reasons for (adverse factors) and against (favourable factors) the potential occurrence of the failure mode. These potential failure modes are developed to the same level as Category I modes. The reasons that these potential failure modes are considered to be of lesser significance are noted and summarized in the discussion. This could include lack of direct indication of problem development, the loading required to initiate a problem is unlikely, and/or the magnitude of the consequences is small.

Category III – The Potential Failure Mode Condition Requires More Information or Analyses in Order to Classify it: These potential failure modes to some degree lacked information to allow a confident judgment on significance, and thus a dam safety investigative action or analysis can be recommended. Because this category of failure mode requires action before resolution, the failure mode may also be highlighted for enhanced monitoring.

Category IV – Potential Failure Mode Ruled Out: Potential failure modes may be ruled out because the physical possibility does not exist, information came to light that eliminated the concern that had generated the development of the potential failure mode, or the failure mode is clearly non-credible or reasonable to postulate because the likelihood of development leading to adverse consequences is negligible.

A brief summary description of the potential failure modes (PFM's) identified and the categorization of each are presented in the following Table 6.6-1:

TABLE 6-6-1
DESCRIPTION AND CATEGORIZATION OF POTENTIAL FAILURE MODES

Potential Failure Mode*	Description	Category
H1	During a flood event, log lifting equipment fails when logs are installed in the sluices and water level is high and cannot be discharged through the dam. Higher hydrostatic loads than expected under flood conditions results in a breach in the dam and/or the embankment between lock and dam is overtopped resulting in a breach in the embankment.	II
H2	Debris blocks sluiceways causing the outflows to be reduced and increasing water level resulting in the dam being subjected to a higher hydrostatic loading conditions than it can withstand.	II
S1	A breach in one of the canal earth embankments occurs due to piping and erosion either because of lack of sheet piling or sheet piling exists but the top end of the sheet piling is lower than the maximum operating water level of the canal or due to the piling corroding through with time.	II
S2	Normal loading conditions - increase in uplift or degradation of strength such that sliding initiates at the dam-foundation contact or a concrete cold joint resulting in a sudden breach down to contact or cold joint.	II
S3	A lock retaining wall fails due to higher than expected piezometric head level in the backfill and subsequently causing the exposed embankment to erode resulting in a breach.	III
S4	Normal loading conditions - deterioration of sluice piers causing sliding of a pier or shear failure of stoplog slots resulting in breach of pier and loss of stoplogs in two sluices or loss of stoplogs in one sluice.	IV
S5	The u/s and d/s gates malfunction or gates structurally fail and neither of the gates are able to close and stop the release of the water (i.e., breach through the lock).	IV

*Note: H denotes potential hydrologic failure mode, and S denotes potential “static” failure mode under normal operating conditions.

Only Categories II, III and IV failure modes for the Talbot River Dam and Lock 39 were identified.

6.6.1 Potential Failure Mode H1

Description: The stoplogs are installed and the headpond water level is high and the logs need to be removed to prevent the water level from rising above the maximum operating water level. Assuming the log lifting equipment malfunctions then the dam will not be able to discharge sufficient water. The dam will be subjected to higher than expected hydrostatic loads that could result in failure of a pier, spillway, or a bulkhead and/or an embankment (at dam or along canal) could be overtopped causing erosion and a breach in an embankment.

Breach Consequences: The cause of the breach could be due to a structural failure of the dam due to the excessive hydrostatic load (i.e., high water level) and/or erosion / washout of the embankments and/or the abutments of the dam.

There is risk of loss of life due to the breach of the dam (see Section 5.2.4).

Adverse Factors:

- The stability analyses for the structures (see Section 6.4), based on the assumed shear strength parameters for the concrete/bedrock interface found that gravity bulkhead and the spillway did not meet the sliding and "location of resultant" requirements for the usual summer loading condition.
- No full flow test (i.e., remove and reinstall all stoplogs from the deepest and widest sluice under high flow conditions) was carried out as part of this DSR to verify that the equipment adequately functions for high flow conditions. The full flow test was not carried out because of concerns of flooding downstream as a result of the test. Based on PCA's experience with the nearby Bolsover Dam, PCA has reported that potentially only a maximum of 8 stoplogs can be removed from each sluice with the existing flow control equipment,
- Under IDF conditions, with all stoplogs removed, the concrete core walls of the Right and Left earthfill embankments would be overtopped potentially resulting in damage/wash out of the earthfill and failure of one or both embankments.

Favourable Factors:

- Although it has not been confirmed that the flow control equipment is sufficient to remove all logs under high flow conditions the functional test did verify that the equipment is operating satisfactorily.

Rationale for Selection of Category II:

Although the gravity bulkhead and spillway did not meet the stability requirements for the usual summer loading condition, the sliding safety factors were greater than 1.0 and the "location of resultant " is within the base.

Because of the above noted deficiencies, it is possible that a breach in one of structures could occur. Based on the current operations practices, it is expected that the logs will be removed in time before water levels become prohibitive. However, if an abutment is overtopped, a breach in the abutment could occur depending upon how long it is subjected to the overtopping.

Potential Risk Reduction Measures, Monitoring, and/or Analyses:

- Increase the stability of the dam with post-tension anchors or by increasing the mass of the dam.
- Ensure that the log lifting equipment is well maintained and regularly operated and inspected.
- Carry out an annual functional flow test to demonstrate the safe and reliable operation of the flow control equipment.
- Carry out a full flow test, or if not practical, demonstrate / confirm (i.e., at an alternate dam) that the flow control equipment is capable of removing all stoplogs, safely and reliably, under high flow conditions, and if necessary upgrade the flow control equipment.

6.6.2 Potential Failure Mode H2

Description: Should debris accumulate at the sluices they could become blocked causing the water level to rise above the normal maximum operating water. The dam will be subjected to higher than expected hydrostatic loads that could result in failure of a pier, spillway, or a bulkhead and/or an embankment (at dam or along canal) could be overtopped causing erosion and a breach in an embankment.

Breach Consequences: As for potential failure modes H1 the cause of the breach could be due to a structural failure of the dam due to the excessive hydrostatic load (i.e., high water level) and/or erosion / washout of the embankment.

There is risk of loss of life due to the breach of the dam (see Section 5.2.4).

Adverse Factors:

- The stability analyses for the structures (see Section 6.4), based on the assumed shear strength parameters for the concrete/bedrock interface found that the gravity bulkhead and the spillway did not meet the sliding and "location of resultant" requirements for the usual summer loading condition.
- No debris monitoring and maintenance program is implemented at Talbot River Dam.

Favourable Factors:

- PCA personnel have reported that debris to-date has not been a significant issue.
- It is unlikely that the embankments will overtop because there is sufficient freeboard under maximum normal operating conditions. The water levels are well monitored and maintained thus if the operations are carried out as intended, the Operators should have sufficient advance warning to access the sluices and clear the debris before the water levels become prohibitive.

Rationale for Selection of Category II:

Based on the Operators' observations, debris is considered to not be an issue. However, because the sliding factor of safety for the gravity bulkhead and spillway under the usual summer loading condition is deficient, it is possible that a breach in one of these structures could occur with the higher than expected hydrostatic load.

Potential Risk Reduction Measures, Monitoring, and/or Analyses:

- Even though debris accumulating at the sluices has been reported to be a non-issue, consideration should be given to formalizing a debris inspection and maintenance program and training all Operators regarding the potential adverse effects of debris, and how to remove/handle the debris (especially during a flood event).
- Increase the stability of the dam with post-tension anchors or by increasing the mass of the dam.

6.6.3 Potential Failure Mode S1

Description: During normal operating conditions or during an IDF, a breach in one of the canal earth embankments due to piping and erosion either because of lack of sheet piling or sheet piling exists but the top end of the sheet piling is lower than the maximum operating water level of the canal.

Breach Consequences: There is risk of loss of life due to the breach of the embankment (see Section 5.2.4).

Adverse Factors:

- The earth embankments do not have any corewall. The original PWGSC Terms of Reference for this DSR noted that a steel sheet pile wall was installed along the crest of the embankments towards the upstream edge of both canal embankments, however, no background documentation or drawings were available to substantiate the existence of the steel sheet pile wall nor could any PCA personnel confirm that sheet piles were installed. KGS Group carried out test pitting at four select locations along the canal embankments to try and identify whether or not the sheet piles were installed. The water level was high during test pitting (approximately 0.8 m below the crest), and the test pitting depth and extent were limited to above the water level to ensure that there would be no seepage or erosion, particularly since the location and elevation of the sheet piles were not yet known. Sheet piles were not encountered within the investigated depth and extent of the test pits.
- Based on the KGS Group visual inspection, the canal embankments are generally in fair to poor condition geotechnically, in recognition of the significant erosion and notable loss of the upper portion of the upstream slope observed. Rotated, damaged and deformed gabion baskets were also observed in the both embankments.
- Seepage and erosion was observed in the summer of 2013 just upstream of Lock 40.

Favourable Factors:

- There were no signs of deep seated slope failures, significant slope movements, nor seepage that would suggest concerns related to the slope stability and safety of the

structures. Although not confirmed with the test pitting, it is still possible that sheet piling is installed.

Rationale for Selection of Category II:

Although there currently are no signs of seepage, slope failures or significant slope movements, it is possible that a breach due to piping and erosion could occur if there is no sheet piling or if the sheet piling was not installed deep and/or high enough.

Potential Risk Reduction Measures, Monitoring, and/or Analyses:

- Carry out further field investigations to determine if sheet piling is installed and if it has been installed correctly and to the appropriate depth and extends sufficiently high enough (i.e., above the IDF water level).
- If necessary, based on the field investigation results, install new sheet piling or modify or replace existing sheet piling.
- Implement a periodic monitoring program of the canal embankments. Key items to look for include sinkholes on the crest, failures / loss of the downstream slope, seepage conditions at the toe, etc.

6.6.4 Potential Failure Mode S2

Description: Under normal loading conditions an increase in uplift pressure and/or degradation at the interface of concrete cold joints and/or at the concrete/rock interface initiates sliding at the concrete/rock interface or cold joint, and no mechanism exists to arrest the movement until the dam breaches.

Breach Consequences: Sliding would result in a sudden rupture of the dam to the bedrock or the cold joint.

There is risk of loss of life due to the breach of the dam (see Section 5.2.4).

Adverse Factors*:*

- The stability analyses for the structures (see Section 6.4), based on the assumed shear strength parameters for the concrete/bedrock interface found that the gravity bulkhead and the spillway did not meet the stability requirements for the usual summer and winter loading conditions nor did the sluiceway for the usual winter loading condition.
- There is no drainage system. Drainage is typically the first line of defence against sliding instability as it reduces water pressures resulting in a decrease in driving forces and uplift on potential sliding planes.

Favourable Factors:

- Zero cohesion was assumed for the stability analyses. It is possible that some cohesion may exist and the shear friction angle may be found to be greater than assumed.
- Although the gravity bulkhead and spillway did not meet the stability requirements for the usual summer loading condition, the sliding safety factors were greater than 1.0.

Rationale for Selection of Category II:

Although to-date, there is no evidence of distress to the structures and they are functioning as intended. As a result of the sliding factor of safety for the gravity bulkhead and spillway under the usual summer loading condition is deficient, it is possible that a breach in one of these structures could occur.

Potential Risk Reduction Measures, Monitoring, and/or Analyses:

- Increase the stability of the dam with post-tension anchors or by increasing the mass of the dam.
- Monitor the performance of the structures for any evidence of seepage or movement and take remedial action if required.

6.6.5 Potential Failure Mode S3

Description: A lock retaining wall fails due to higher than expected piezometric head level in the backfill causing the embankment to erode resulting in a breach.

Breach Consequences: There is risk of loss of life due to the breach of the embankment (see Section 5.2.4).

Adverse Factors:

- The retaining walls do not meet the CDA Guidelines stability requirements for cases where the water level in the lock is low or when the chamber is dry.
- The piezometric water level in the backfill has a significant influence on the stability of the walls.
- A piezometer, as part of the KGS Group geotechnical investigation program, has been installed in the embankment to monitor the water level. The retaining walls do not meet the stability requirements based on the limited data retrieved to-date.

Favourable Factors:

- As noted above, a piezometer has been installed in the embankment to monitor the water level over the long term.
- During the site inspection, the walls were showing no signs of distress.
- No “as-built” drawings are available to confirm the construction details at the base joint between the base slab and walls. Potentially, steel rebar is installed that connects the wall to the slab and thus the assumption used in the stability analysis that the wall and slab are acting totally independently may not be appropriate.

Rationale for Selection of Category III:

KGS Group categorized this failure as a Category III primarily because there is not sufficient piezometric water level data and “as-built” construction information to rule out a failure of these walls due to the piezometric water levels in the backfill.

Potential Risk Reduction Measures, Monitoring, and/or Analyses:

- Carry out annual engineering and routine inspections of the walls.
- Continue to monitor the piezometric water levels.
- Confirm the “as-built” construction details for the walls.

- Lower the piezometric water level within backfill by improving the drainage system.
- Enhance the stability of the wall with tie-back anchors and/or by adding mass to the wall
- Modify the backfill material.

6.6.5.1 Potential Failure Mode S4

Description: Under normal operating conditions, assume gradual deterioration of the piers of the sluices resulting in sliding of a pier due to its sectional mass being reduced or shear failure of stoplog slots on the downstream side of the stoplog gains.

Breach Consequences: If a pier slides, this would result in a breach of the bay. If the stoplog slot fails, this would result in a loss of stoplogs in one bay. There is little warning time available to individuals downstream of the dam. Flooding could occur and there is potential for loss of life. Flooding and loss of life is possible in this scenario, only if the stoplogs are in place and the reservoir level is high.

Adverse Factors:

- For the winter loading condition, based on the assumed shear strength parameters for the concrete/bedrock interface, the sliding factor of safety for the sluiceway does not meet the stability requirements and it is less than 1.0.

Favourable Factors:

- No significant areas of concrete deterioration have been noted in any of the structures.
- Deterioration is slow and with appropriate monitoring should be able to be identified and repaired before any breach occurs.

Rationale for Selection of Category IV:

KGS Group categorized this failure as a Category IV primarily because no signs of distress were noted during the DSR inspection. The concrete deterioration is a slow progression and should be able to be identified and repaired if the structures are adequately monitored and repairs carried out before the sluice piers reach a critical state.

Potential Risk Reduction Measures, Monitoring, and/or Analyses:

To ensure that this potential failure mode remains at a Category IV it is essential that as a minimum an annual engineering inspection is carried out for the dam to confirm the structurally integrity of the concrete.

6.6.6 Potential Failure Mode S5

Description: The upstream and downstream lock gates malfunction or gates structurally fail and neither of the gates are able to close and stop the release of the water (i.e., breach through the lock).

Breach Consequences: In effect, this is a breach and therefore there is risk of loss of life (see Section 5.2.4).

Adverse Factors:

The structural analysis found that the downstream lock gate did not have sufficient structural capacity for its intended design loads.

Favourable Factors:

- Based on KGS Group's observations while witnessing the function of the gates, they appear to be operating satisfactorily and the Operators have reported no concerns regarding the operation of the gates.
- Based on KGS Group's inspection of the gates, they appear to be in good condition.

Rationale for Selection of Category II:

Although the analysis shows that a gate does not meet current design code standards, both the upstream and downstream gates are operating satisfactorily and are in good condition.

Potential Risk Reduction Measures, Monitoring, and/or Analyses:

- Replace the gates with steel gates that meet current design standards.

- Carry out annual engineering and routine inspections of the gates.
- Implement a maintenance program for the gates.

7.0 OPERATIONS AND OPERATOR SAFETY

The assessment of the operations at Talbot River Dam and Lock 39 is based on the following sources of information:

- site and equipment inspection;
- PCA Standing Orders;
- witnessing a functional flow control equipment test at the dam and witnessing the operations of the lock;
- an operations questionnaire (see Appendix F) provided to and completed by the Northern Sector Manager; and
- information provided verbally by PCA staff.

PCA Standing Orders provide general guidance to operations and maintenance PCA staff of the Trent-Severn Waterway (TSW). These Standing Orders include general safety and dam operation guidelines and are based on PCA's policies, Historic Canals Regulations, the Canadian Labour Code Part II, the Canadian Occupational Health and Safety Regulations (COHSR), established operating procedures and practices.

7.1 TALBOT DAM - INSTALLATION / REMOVAL OF STOPLOGS

To access the dam, one has to climb a steep slope that can be slippery and a falling hazard (Photo 7.1-1). Provisions should be made to provide a safer and easier access to the dam.

PHOTO 7.1-1
STEEP AND SLIPPERY SLOPE IS THE ACCESS TO THE DAM

Operation of the stoplogs at the dam is performed using two manual winches mounted on rails that span across the two sluices in the dam. Cranking the winches and manipulating the logs on the deck of the dam is heavy manual labour and could result in strain and pinch injuries if not executed with care. Work by the same crew on multiple dams in the Northern Sector in one day could tire the Operators excessively and contribute to increased work safety risk.

Operators work over an open sluice when removing and installing logs and this poses a hazard of falling into the sluice and possibly into the water below. Engineered fall restraint systems should be installed and used during the log operations.

Not only is it important that the above risks are mitigated to create a safer working environment, but also an injury of one individual, that requires medical attention, would remove from service the one and only two person crew that is dedicated to the Talbot River Dam and Lock 39 and Talbot Dam and Lock 38.

7.2 LOCK 39 OPERATIONS

KGS Group witnessed the operation of the Lock for the purpose of witnessing the condition of the mechanical equipment while in operation (see Section 4.3.4) and to assess the safety of the

Operators and public while undertaking their duties. Both upstream and downstream locking procedures are functionally similar, although sequences vary. The Operators have overall responsibility for the safety of patrons in the lock.

The PCA Standing Orders provide guidelines and procedures for the operation of the locks. There has been a change in the operation of the locks and the waterway since the Standing Orders were last published. The lock Operators are required to collect fewer tolls as patrons are encouraged to purchase a pass to use the waterway. This is a great enhancement to the safety of the lock Operators, as they have less need to lean over the edge of the lock in order to collect tolls from a wide range of vessels. This has reduced the likelihood of an Operator being injured in a fall or becoming trapped between a boat and the lock wall.

Guardrails are lacking along the lock and are inadequate on the walkways over the lock thus falling into the lock by the Operator or public is a significant safety hazard. Section 8.2.1 discusses in detail the guardrail requirements both for the Operators and the public.

No PCA employees were wearing water safety gear. Although life rings are easily accessible from anywhere around the lock, a person in the water must be conscious and capable of grasping a ring in order to use it. There is also the danger of an untrained or panicking person concussing a struggling person by throwing the ring at them instead of near them. The Operators carry a large amount of responsibility for the safety of others, and it is not clear whether the temporary employee has received enough training to safely abort the operation if the permanent PCA employee became injured.

PCA must assess the risks of a lock Operator being knocked unconscious during a locking operation, either due to accident or medical condition, and weigh these risks against the discomfort to their employees and the costs of obtaining self-inflating lifejackets for each worker. Note that PFDs will not prevent an unconscious person from drowning.

7.3 OPERATOR STAFFING AND WORKLOAD

PCA Northern Sector staff are responsible for 28 dams, 14 locks, and 4 bridges. The sites span from Bobcaygeon to Georgian Bay with headquarters at the Kirkfield office and shops at

Kirkfield and Washago. The Talbot River Dam and Lock 39 are staffed by a crew that varies throughout the year. The schedule is as follows:

- April 1st to May Long Weekend – No Operators are specifically assigned to Talbot River Dam and Lock 39 but there is a pool of five Operators that are dedicated to the Northern Sector for controlling the water levels and maintenance activities.
- May Long Weekend to Thanksgiving (navigation season) – Two Operators are dedicated to the operations at Talbot Dam and Lock 38 and Talbot River Dam and Lock 39. Students are also available during this period to assist with the lock operations, equipment lubrication and other duties such as cutting the lawn. The Operators and students work seven days per week and 11 hours per day from 8:30 am to 7:30 pm each day.
- Thanksgiving to November 7th – No Operators are specifically assigned to Talbot River Dam and Lock 39 but there is a pool of five Operators that are dedicated to the Northern Sector for controlling the water levels and maintenance activities. These Operators are located at the Kirkfield office, 17 km away from the dam.
- November 8th to March 31st – No Operators are specifically assigned to Talbot River Dam and Lock 39 but PCA maintenance staff are available if required.

Communications between operating staff and the Northern Sector office and the Peterborough headquarters is by cell phone. During night operations flashlights and / or generators are used.

There have been only minor employee or contractor safety incidents in the past 10 years at the Talbot River Dam and Lock 39.

7.3.1 Talbot River Dam

Stoplog operations at Talbot River Dam may be required as often as daily during freshet (i.e., in the spring) and approximately 2 to 3 times per week during the summer months. In the fall, stop log operations are required about once a week and typically during the winter no stop log operations are required.

Water level at the dam is sensed continuously and automatically logged every hour. A set of readings is emailed once per day to the water control engineer and Northern Sector manager.

The engineer and local operating staff can also call in to the level sensor at the dam at any time to obtain water level information. During freshet, Northern Sector PCA staff review the data daily.

Access to the dam is by public road, on water, on foot and in winter also by snowmobile.

In general, several problems may arise that could reduce the number of logs that can be removed from a sluice in the dam or that could extend the time required to remove the required number of logs. A U-bolt (i.e., “D”) could come out of a log, logs can roll 90 degrees in the sluice, logs can break or jam in the sluice and winch gears or latches can jump or break and allow the winch to free wheel. There are no known problems such as these that have occurred at the Talbot River Dam and reduced the number of stoplogs which can be removed during normal or flood conditions. However, PCA staff have no recollection or record of an occasion when all of the logs were removed under high water level conditions. The maximum number of logs that are believed to have ever been removed is seven out of ten stoplogs. Approximately 4 years ago seven stoplogs were removed from each sluice at Talbot River Dam and this caused flooding downstream. If a log jams, staff have always been able to resolve the problem based on experience and by using additional tools such as grapples. As a last resort staff could cut the log and let it go downstream to be collected later but this has not been necessary at the Talbot River Dam.

As infrequently as once a year some debris may interfere with hooking of the logs at this site but this has never created an issue and the amount of debris is never significant enough to block the sluices. There is no debris management program in place nor is one required.

Ice jams do not occur on this river system and are of no concern. From December to April ice formation does occur up to 150 mm thick and this is enough to affect the procedures effect for the logging operations. However, during the winter stoplog operations are not usually required and if they are necessary, Operators use tiger torches to melt the ice. Frazil ice has been observed at this site but is not an operational issue.

To jack down logs, the Operator is required to climb down into the open sluice and stand on the top log to install the jack and blocking wedges in the gains and then to operate the jack. Strain

and fall injuries could occur while climbing into the sluice and, once standing on the top log in the sluice, the Operator could slip and fall against the sluice walls, onto the sluice apron below or into the water below.

In the event of a flood emergency, the PCA water control engineer and the Northern Sector manager determine which of the 28 dam locations in the sector shall receive operating priority. Prioritization is based on percent full storage combined with travel distance. Response time, depending on location of the Operators and weather conditions, varies between 15 minutes to 1 ¼ hours with a maximum travel distance of approximately 70 km.

Typically, sufficient staff are available in the event of a flood emergency. However, combined with the requirements for the same staff to operate Lock 39 and Talbot Dam and Lock 38, it could be challenging for Operators to respond to operate the Talbot River Dam as well. If additional resources are required, PCA staff can request assistance from headquarters in Peterborough. Two Operators are on standby for the Kirkfield region. If an emergency happened at night or on weekends, the Northern Sector manager would try to find Operators. It could take 1 hour to contact someone and another hour for staff to obtain equipment and get to site.

In a major storm event it is conceivable that many or most of the Northern Sector 28 dams would require log removal or sluice gate operation in a short time. As this storm event could affect neighbouring districts, the availability of nearby additional support to the Northern Sector crew might be limited. Also, it is likely that access to some sites could be increasingly difficult due to flooding and water on roads. Working conditions in bad weather would reduce productivity of Northern Sector dam Operators. Combined, these risks, along with the recognized safety hazards of night work (which is typically not done), suggest that the Northern Sector crew might not be able to operate all the dams in an optimum time frame and that increased damage from the flows would be one consequence.

7.3.2 Lock 39

Two Operators operate the Lock 38 and Lock 39 as well. There are two gate leaves at the upstream end of the lock and two gate leaves at the downstream end of the lock. The gate

leaves are all manually opened and closed. A manually operated butterfly valve is located in each gate leaf for filling or draining the lock.

During the summer navigation season the lock is operated daily as often as every 20 minutes, although some days there is almost no vessel traffic.

Lock Operators perform periodic light maintenance on equipment such as oiling and greasing. In the fall Operators identify more significant maintenance requirements such as what needs to be repaired and then maintenance crews typically carry out the work in the fall. There is no documented formal maintenance program or documented maintenance records. Maintenance issues have arisen unexpectedly that have caused operating interruptions from 1 hour to 1 week in duration. The details of such interruptions are unknown but they could likely be reduced with a more formal maintenance program in place.

7.4 DAM MANAGEMENT CRITERIA

The PCA Directive includes requirements for management of the dams that correspond to their Hazard Classification. These requirements pertain to:

- The frequency of inspections and testing
- The need for Emergency Preparedness Plans (EPP) and Emergency Response Plans (ERP)

For the Talbot River Dam and for Lock 39, classified as HIGH A, the required frequency of inspections, as a minimum, indicated in the PCA Directive are:

- Dam Safety Reviews every 5 years
- Engineering inspections every year
- Routine inspections monthly.

Currently, PCA does not formally carry out engineering and routine inspections in accordance with the frequency specified in the PCA Directive. This DSR, as previously noted, is the first DSR that has been completed for this site.

No internal ERP nor EPP has been prepared for this site. In accordance with the PCA Directive and CDA Guidelines, an ERP and EPP are required for each site where a dam failure results in a life safety hazard. Thus, preparation of an ERP and EPP, to be used by PCA and other stakeholders that would be involved or affected by an emergency condition at the Talbot River Dam and Lock 39 is required given the sites HIGH A hazard classification.

Regardless of a site's hazard classification, in accordance with the PCA Directive and CDA Guidelines, operations, maintenance and surveillance (OMS) procedures are to be developed and documented in a manual. No OMS Manual exits for this site.

8.0 PUBLIC SAFETY

8.1 PUBLIC USAGE OF THE SITE

This site is accessed by the general public primarily for navigation purposes (i.e., use of the lock). The public is also noted by the staff to be in the vicinity of the site to tour the site, fish, swim, do watercraft activities such as waterskiing, wakeboarding, canoeing, kayaking, etc.

There have been no public safety incidents in the past 10 years at this site.

8.2 PUBLIC SAFETY ISSUES

The policy of allowing the public on dams brings with it the requirement to take reasonable measures to protect the public from harm. A detailed public safety review has been carried out by KGS Group and is reported in a separate document. In general, the following subsections summarize the public safety deficiencies.

8.2.1 Guardrail Requirements

If the dam and lock are only accessed by PCA staff then the Canada Labour Code (CLC), Canada Occupational Health and Safety Regulations (COHSR) guardrail requirements would apply as the PCA staff fall under federal jurisdiction. However, currently the public is permitted freely to access the dam and lock and therefore guardrails that meet the National Building Code of Canada (NBCC) or the CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC) are applicable versus guardrails that meet the COHSR requirements. If the dam or lock walkway is considered to be a pedestrian bridge then guardrails in accordance with the CAN/CSA S6-06 CHBDC should be installed.

The CDA Guidelines do not specify the type of guardrail system that is required for public safety. However, the Ontario Ministry of Natural Resources (OMNR) 2011 Best Management Practices for Public Safety Around Dams states that where a vertical fall at a dam creates a potential hazard, hand railings, guard rails, or fencing should be installed consistent with the

Ontario Building Code (OBC) where the public has access. The OBC guardrail requirements are similar to the NBCC requirements.

A description of the different codes/regulations follows:

8.2.1.1 Canada Occupational Health and Safety Regulations (COHSR)

Guardrails are required if a person is exposed to falling a distance greater than 2.4 meters or falling into water and shall consist of the following (COHSR, Sect 2.12):

- a horizontal top rail not less than 900 mm but not more than 1100 mm above the base of the guardrail;
- a horizontal intermediate rail spaced midway between the top rail and the base; and
- supporting posts spaced not more than 3 m apart at their centres.

Every guardrail shall be designed to withstand a static load of 890 N applied in any direction at any point on the rail.

Specific requirements are not provided in COHSR for guardrail systems that are made of wire rope but the Ontario Occupational Health and Safety Act and Regulations (OHSA) states that the following additional requirements shall apply to a guardrail system that is made of wire rope:

- The top rail and intermediate rail shall be made of wire rope that is at least 10 mm in diameter, and the rope shall be kept taut by a turnbuckle or other device.
- The outward deflection of the top rail and intermediate rail resulting from the loads above specified loads shall not extend beyond the edge of the surface to be guarded.
- The system shall have vertical separators at intervals of not more than 2.4 m and horizontal supports at intervals of not more than 9 m.

8.2.1.2 National Building Code of Canada (NBCC)

The NBCC guardrail requirements specify the following:

- Guardrails shall not be less than 1,070 mm high or not less than 900 mm if the walking surface served by the guardrail is not more than 1,800 mm above the finished ground level.
- Openings through the guardrail shall be of a size that will prevent the passage of a spherical object having a diameter of 100 mm unless it can be shown that the location and size of openings that exceed this limit do not represent a hazard.
- The guardrail shall have the structural capacity for the following loads:

    - Horizontal load of 0.75 kN/m applied inward or outward at any point at the top of the guard and concentrated load of 1.0 kN applied at any point.
    - Horizontal concentrated load of 0.5 kN applied inward or outward applied at any point on individual elements.
    - Evenly distributed vertical load of 1.5 kN/m applied at the top of the guardrail.

8.2.1.3 CAN/CSA S6-06 CHBDC Guardrail (Pedestrian Barriers) Requirements

- Guardrails shall be provided on both sides of pedestrian bridges.
- The guardrail shall have a minimum height of 1,050 mm.
- The openings in the barriers shall not exceed 150 mm in the least direction or shall be covered with chain link mesh and must meet specific design loading as specified in Clause 3.8.8 of CAN/CSA S6-06.
- The guardrail shall be capable of restricting a uniform load of 1.2 kN/m applied laterally and vertically simultaneously.

Further review is required to be carried out by PWGSC/PCA as to what is the most appropriate type of guardrail/barrier system that should be installed at the site. If public access is limited or restricted then the industrial 2-rail guardrail system is appropriate for this site. However, if open access to the public is continued to be allowed, then a guardrail in accordance with the NBCC or CAN/CSA S6-06 CHBDC is appropriate for this site.

8.2.2 Existing Guardrails

8.2.2.1 Talbot River Dam

On the upstream side of the gravity bulkheads the guardrail is galvanized industrial 2 rail system in accordance with COHSR (Photo 8.2.2-1). However, there is no railing at the landing of the steps of the south gravity bulkhead which presents a falling hazard (Photo 8.2.2-2). The falling height is less than 2.4 m thus the railing is not essential but it still would be beneficial to have a gate at the landing especially under wet and/or slippery conditions. Guardrail should also be considered to be installed on the downstream side of the steps for the south gravity bulkhead (Photo 8.2.2-3). As previously noted, currently the public are permitted to access this dam so the guardrails should be in accordance with the NBCC or CHBDC versus COHSR.

On the upstream side of the sluice deck, steel posts with wire mesh fencing is installed (Photo 8.2.2-4). This fencing is in good condition and meets the intent of preventing workers from falling off of the deck.

The downstream guardrail for both bulkheads and the sluice deck is a 3 strand wire rope system but the height is approximately 1.3 m which is higher than the acceptable maximum permissible height of 1.1 m (Photo 8.2.2-1). The wire rope is relatively slack and no structural analysis was carried out on system to determine its structural capacity as this was outside the scope of this inspection.

PHOTO 8.2.2-1
VIEW OF GUARDRAILS ON NORTH GRAVITY BULKHEAD

PHOTO 8.2.2-2
SOUTH GRAVITY BULKHEAD STAIR LANDING HAS NO GUARDRAIL OR GATE ON D/S SIDE

PHOTO 8.2.2-3
SOUTH GRAVITY BULKHEAD STAIRS HAVE NO GUARDRAIL ON U/S SIDE

PHOTO 8.2.2-4
WIRE MESH FENCING ON U/S OF SLUICE DECK

8.2.2.2 Lock 39

The walkways over the lock gates have industrial guardrail that meet the COHSR requirements but not the NBCC or CHBDC requirements. The guardrail is in good condition, however, there is a gap at the intersection of the gates that is large enough for a person to fall through,

particularly in winter ice and snow conditions (Photos 8.2.2-5 and 8.2.2-6). A safety barrier system is required at this location. There are no guardrails along the lock walls (Photo 8.2.2-7) which pose a significant falling hazard.

PHOTO 8.2.2-5
TOP OF LOCK GATE SHOWING GAPS BETWEEN GUARDRAILS

PHOTO 8.2.2-6
VIEW OF GAP BETWEEN GUARDRAILS ON LOCK GATE WALKWAY

PHOTO 8.2.2-7
NO GUARDRAILS ALONG LOCK WALLS

Currently the only location that has appropriate guardrails for the public are along the staircases to the lock (Photo 8.2.2-8).

PHOTO 8.2.2-8
GUARDRAILS ALONG STAIRCASES TO LOCK ARE SUITABLE FOR THE PUBLIC

8.2.3 Talbot River Dam Sluiceway Deck

As previously noted in Section 7.1, to access the dam, one has to climb a steep slope that can be slippery and a falling hazard (Photo 7.1-1).

There are various upstands such as stoplog supports and brackets for the frames of the gain covers that pose a potential tripping hazard for the Operators. (stoplog supports, rail stop–ends, and gain cover brackets). These have been painted to make them more visible in an attempt to reduce the hazard (Photo 8.2.3-1).

PHOTO 8.2.3-1
UPSTANDS ON SLUICE DECK ARE TRIPPING HAZARDS

As noted in Section 4.2.2, there are some deteriorated wooden planks noted on the deck, but PCA has informed KGS Group that these planks have subsequently been replaced.

8.2.4 Safety Booms

There are no public safety booms upstream or downstream of dam. On the upstream face of the dam there is a cable system to try and prevent a person or boat from being swept over the

sluices (Photos 8.2.4-1 and 8.2.4-2). This cable system is not sufficient for public safety. The cable system should be replaced yellow public safety booms installed upstream of the dam in accordance with the 2011 CDA Guidelines for Public Safety Around Dams. The safety booms should be oriented to be self rescuing. Consideration should also be given to also installing yellow public safety booms downstream of the dam.

PHOTO 8.2.4-1
DOUBLE STRAND CABLE IS INSTALLED ACROSS U/S OF DAM
(Winter Draw Down Water Level)

PHOTO 8.4.2-2
DOUBLE STRAND CABLE IS SIGNIFICANTLY HIGHER THAN WINTER DRAW DOWN WATER LEVEL

8.2.5 Fencing

Currently no fencing is installed at the dam or lock site. When the safety booms are installed, fencing should be installed from the upstream and downstream sides of the dam to the boom anchors to prevent the public from accessing the waterway near the dam.

8.2.6 Signage

Signage is installed on the upstream and downstream faces of the dam but the existing signage does not meet the CDA Guidelines in that an “emergency call” number is not provided on the sign nor is the name of the dam and owner identified on the signs (Photo 8.2.6-1).

As existing signs deteriorate and are required to be replaced, they should be replaced with ones that meet the CDA requirements.

PHOTO 8.2.6-1
SIGNAGE DOES NOT MEET THE CDA GUIDELINES

9.0 RECOMMENDATIONS

Recommendations based on this DSR are listed below by discipline. In addition, Table 9-1 summarizes the deficiencies/issues, recommendations, and the indicative cost to implement the recommendation.

9.1 GENERAL

As per the PCA Directive and the CDA Guidelines the following types of inspections should be carried out:

- *Routine Inspections* should be carried out monthly by trained PCA staff who know the Talbot Dam and Lock 38 site well enough to recognize changes in appearance or performance and signs of problems (e.g., sinkholes, seepages, piping, slop slippage, debris accumulation, ice buildup, etc.). Training can be provided in-house or by an engineering consultant.
- *Engineering Inspections* should be carried out annually as a minimum. The requirements for the annual engineering inspection are the following:
  - Thorough visual inspection of the earth embankments, geotechnical features, dam and lock and associated instruments (i.e., water gauges).
  - Detailed documentation of observations and assessment of conditions, features or deficiencies observed, highlighting changes from previous inspections, and providing recommendations for maintenance, repairs, investigation, or further monitoring.
  - The inspection must be performed by a qualified engineer experienced in dams and dam safety or by a certified engineering technologist (C.E.T.), or equivalent, under the direct supervision and certification of a qualified engineer, both experienced in dams and dam safety.
- *Special Inspections* shall be performed following an unusual event at the dam (e.g., unusually heavy rainfall, extreme flood, earthquake, unusual or improper operation, etc.) that may have a negative impact on the dam and lock. These inspections should be carried out during or as soon as possible afterwards. These inspections should be performed by qualified PCA staff and defects should be reported to and assessed by a qualified engineer.

9.2 HYDRAULIC / HYDROLOGY

- Develop reference levels and conditions to initiate emergency operation of the structures. These should be documented on operation plans and in the Emergency Response Plan.
- Verify and ensure that all stoplogs can be safely removed at FSL.

9.3 CIVIL STRUCTURES

9.3.1 Condition Assessment

9.3.1.1 Talbot River Dam

- *North Gravity Bulkhead Drainage Channel:* The condition of the concrete at the outlet end should be monitored and eventually repaired within the next 10 years.

- *Log Sluices*: Repair the spalled concrete on the south gain of the north sluice within the next 10 years.

9.3.1.2 Lock 39

- *Walls Within and Upstream of Lock*: Spalled / deteriorated areas should be repaired within the next 2 to 4 years.

- *Walls Downstream of Lock:* Monitor the spalled / deteriorated areas, especially the spalled and fractured area in the vertical face near a construction joint immediately downstream of the foot of the stairway, with an expectation to repair within the next 10 years.

9.3.1.3 Culvert

- *Inlet and Outlet and Adjacent Retaining Walls*: Repair the deteriorated concrete within the next 3 to 5 years.

9.3.2 Structural Stability Assessment

9.3.2.1 Talbot River Dam

- *Gravity Bulkheads*: To address the stability deficiency consider carrying out one or more of the following options:
  - Implement ice control measures.
  - Install post-tension anchors.
  - As a minimum, implement ice monitoring and routine inspection programs, and remedial measures should be carried out if any evidence of movement, concrete cracking/distress or leakage is detected at the concrete/foundation interface. As a minimum, the ice monitoring program should include a trained PCA Operator monitoring the ice formation locations and ice thicknesses and changes in appearance or performance of the dam every two weeks and keeping detailed documentation of this information over the duration of the winter. The ice monitoring inspection can replace the monthly routine inspections.
- *Spillway:* To address the stability deficiency consider carrying out one or more of the following options:
  - Install post-tension anchors and/or add additional mass to the structure.
  - To address the winter load case deficiency implement ice control measures, and confirm the shear strength parameters so that PWGSC/PCA can better assess the needs to or risks not to increase the stability of the dam .
  - In the short term, until the stability of the dam is enhanced and/or ice control measures are implemented, as a minimum, implement ice monitoring and routine inspection programs, and remedial measures should be carried out if any evidence of movement, concrete cracking/distress or leakage is detected at the concrete/foundation interface. Minimum ice monitoring requirements are the same as identified above for the Gravity Bulkhead.
- *Sluiceway*: To address the stability deficiency consider carrying out one or more of the same options identified above for the Gravity Bulkhead.

9.3.2.2 Lock 39

- *Chamber Monoliths:* Carry out one or more of the following options:
  - Undertake investigation to confirm the "as-built" cross section dimensions of the Chamber Monoliths.
  - Enhance the stability of the wall with tie-back anchors and/or by adding mass to the wall.
- A few disturbances in the southeast earthfill embankment were observed between the stations STA 0+050 and 0+250. These disturbances appear to have been man-made. Their conditions should be monitored.
- The concrete core wall of the upstream slope of the Right (North) Wing Earthfill Embankment, approximately 175 m from the Right (North) abutment, is exposed for an approximate 6 m length. At this location there is some localized erosion and shallow sloughing of the upper portion of the slope. Currently, this does not appear to be jeopardizing the stability of the embankment in the short term. However, increased visual monitoring of this section should be performed, particularly during storm or high wind events to monitor for changes in conditions. If additional distress or material loss is observed, then temporary protection measures such as placement of riprap material or fill buttressing may be required. This can be determined on an as-required basis. Final recommendations on treatment to this area will be provided as part of the DSR following completion of the analyses.

9.4 GEOTECHNICAL ASPECTS

9.4.1 Condition Assessment

- Ongoing inspection and monitoring is necessary to confirm good performance. This includes visual monitoring for any distress to the embankments as well as continued readings of the standpipe and vibrating wire piezometers to confirm groundwater levels within the embankment.
- In particular, a few areas that requires close attention during the inspections and monitoring include:

- A few disturbances in the Southeast Earthfill Embankment of the canal that were observed between the stations STA 0+050 and 0+250.
- The concrete core wall of the upstream slope of the Right (North) Earthfill Embankment, approximately 175 m from the Right (North) abutment, is exposed for an approximate 6 m length. This area should especially be monitored during storm or high wind events. If additional distress or material loss is observed, then temporary protection measures such as placement of riprap material or fill buttressing may be required. This can be determined on an as-required basis.
- The upstream (wet side) canal slopes are generally in poor to fair condition due to loss of the upper slope, deterioration of the gabion baskets, and significant erosion of the inner (wet) slope. Repair to the canal slope is expected to be required within the next 10 years. Inspection and monitoring of the canal embankments should be increased until the repairs are initiated.

- Remove vegetation from the canal and dam embankments and the dam abutments within the next 2 years.

9.4.2 Embankment Stability

- For the embankments that do not meet the CDA stability requirements (see Tables 6.5-1 and 6.5-2) carry out one of the following options:
  - modify the slopes to increase their stability ; or
  - implement a monitoring program; or
  - PWGSC / PCA assess the needs to or risks not to increase stability.
- To ensure that the slope stability of the embankments are not jeopardized due to rapid drawdown conditions, PCA must continue with their current procedures to slowly draw down the normal summer operating water level to the normal winter operating water level.

9.5 MECHANICAL EQUIPMENT

- Implement a documented maintenance program for both the dam and lock mechanical equipment.

9.5.1 Talbot River Dam

- *Full Flow Test:* Carry out a full flow test, or if not practical, demonstrate/confirm (i.e., at an alternate dam) that all logs can safely and reliable be removed from the sluices under full flow conditions.
- *Stoplogs:* As an interim measure, regularly inspect and monitor the condition of the stoplogs but plan to eventually replace the wooden stoplogs with steel logs.
- *Winches*: Carry out the following remedial work:
  - Periodically lubricate winch gear teeth and wire ropes.
  - Install guarding over exposed winch gears, which still allows easy greasing.
  - Check and tighten covers on all winch bushings.
- *Sluice Covers and Crossers:* Paint the sluice covers' support brackets and crossers with yellow paint.
- *Rails:* Realign and properly secure the rails to the dam deck.

9.5.2 Lock 39

- Consider replacing the timber gates with steel gates, or as a minimum, monitor the condition the lock gates with a plan to replace them at the first sign of any distress to one of them.
- Carry out NDE and close-up visual inspection of the gate leaf vertical tie rods.
- Ensure lock fill and discharge valve actuator handles are removed when Operators are not present.
- Consider alternative means to operate the gate actuators (e.g. electric motors) to eliminate the need for Operators to work near the edge of the lock deck.
- Carry out the following remedial work:
  - Eliminate excessive play between the gate actuator hand wheel hub and the actuator drive shaft.
  - Assess gate gudgeons, pintles and gate actuators to determine the root cause of the high level of force required to get the gates moving from the stopped position.
  - Repair two badly leaking lock filling valves in the upstream gate.
  - Install guarding over lock filling and draining valve actuator gears and pinions to reduce pinch hazards.

- Modify the gate actuators to secure the handles and to allow locking the actuators when Operators are not present.

9.6 OPERATIONS AND OPERATOR SAFETY

- An engineered fall restraint system should be installed and used during the log operations.
- Develop an engineered system (tools and procedures) for jacking down logs that minimizes or removes the need for Operators to enter into the sluice and stand on top of the logs.
- Assess the log removal requirements for the 28 dams in the Northern Sector during a major storm event to determine if existing response tools, protocols and staffing are adequate to minimize damage from flooding.
- Prepare a maintenance program for the dam, lock and associated equipment and commence documenting of maintenance activities. This will ultimately optimize maintenance expenditures, help minimize log removal time during major storm events and minimize interruptions in lock operations.
- Carry out an assessment of alternative means of operating sluices, including installing gates or log lifting machines at dam locations where the benefit of rapid response and risk reduction are greatest.
- Provide the required classroom and field training to operations, maintenance, and engineering staff covering:
  - Inspection of dams and locks for structural and public safety issues.
  - Identification of issues and problems that may impact the integrity of the structure or represent a safety hazard to the public.
  - Procedures and methods for carrying out required work in a safe manner consistent with the COHSR.
  - Fall arrest / restraint training.
  - Procedures and methods to undertake initial rescue of co-worker and self-rescue.
  - Preparation and operation of the dam under a major flood event or emergency situation.
  - Table top EPP and ERP exercises with MNR and local authorities.
  - Review of required record keeping.
- As per the requirements of the PCA Directive and CDA Guidelines, an Operation, Maintenance and Surveillance (OMS) Manual, Emergency Preparedness Plan (EPP) and Emergency Response Plan (ERP) should be prepared for Talbot River Dam and Lock 39

9.7 PUBLIC SAFETY

Refer to the Public Safety Assessment report that has been issued under a separate document for the detailed public safety recommendations. In general, the Public Safety recommendations are as follows:

Guardrails:

- New guardrails/barriers should be installed alongside the lock and approach channel. Because the lock is open to the public the new guardrail and the existing 2-rail guardrail system should be replaced with guardrail (i.e., pedestrian barrier) in accordance with CAN/CSA S6-06 or NBCC.
- The gap between the guardrails on the walkways over the lock gates needs to be addressed as soon as possible.
- Further review is required to be carried out by PWGSC/PCA as to what is the most appropriate type of guardrail/barrier system that should be installed at the dam site. If public access across the dam is limited or restricted then modifications to the existing guardrail is only required to be carried out as discussed in Section 8.2.2. On the downstream side of the south bulkhead stairs install a guardrail and at the downstream side of the stairs landing install a guardrail gate. If open access to the public is continued to be allowed, then the existing guardrail should be replaced with a pedestrian barrier in accordance with CAN/CSA S6-06 or NBCC.

Safety Booms: Replace the existing cables installed upstream of the dam with yellow public safety booms, oriented for self rescue, in accordance with the 2011 CDA Guidelines for Public Safety Around Dams. Consideration should be given to also installing yellow public safety booms downstream of the dam.

Fencing: When the safety booms are installed, fencing should be installed from the upstream and downstream sides of the dam to the boom anchors to prevent the public from accessing the waterway near the dam.

Signage: As existing dam safety signs deteriorate and are required to be replaced, they should be replaced with ones that meet the CDA requirements.

TABLE 9-1
SUMMARY OF DEFICIENCIES / ISSUES AND RECOMMENDATIONS

STRUCTURE / ITEM	DEFICIENCIES / ISSUES	RECOMMENDATIONS	PRIORITY[1]	COST[2],[3]
TALBOT RIVER DAM				
Gravity Bulkheads	• Spalled concrete upstream (North) • Do NOT meet CDA stability criteria for Usual Summer and Winter Loading Cases based on assumed shear strength parameters.	• Repair • Install post-tension anchors; or • Implement ice control measures and monitor; or • PWGSC/PCA assess the needs to or risks not to increase stability.	Medium	$70k $140k (anchors)
Spillway	• Does NOT meet CDA stability criteria for Usual Summer and Winter Loading Cases based on assumed shear strength parameters.	• Install post-tension anchors; or • Implement ice control measures; or • Confirm the shear strength parameters so that PWGSC/PCA can better assess the need to or risks not to increase stability.	Medium	$140k (anchors)
Sluiceway	• Does NOT meet CDA stability criteria for Usual Winter Loading Cases based on assumed shear strength parameters. • Spalled concrete at south gain of north sluice	• Install post-tension anchors; or • Implement ice control measures and monitor; or • PWGSC / PCA assess the needs to or risks not to increase stability. • Repair	Medium Medium	$100k (anchors) $60k

STRUCTURE / ITEM	DEFICIENCIES / ISSUES	RECOMMENDATIONS	PRIORITY[1]	COST[2],[3]
Drainage Channels	• Undermined channel and fractured concrete	• Repair	Medium	$20k
Winches	• Winch gears not fully guarded • Winch gear teeth and wire ropes not lubricated • Bushing covers are not securely bolted down • Capability to remove all stoplogs under high flow conditions was not confirmed	• Install guarding over exposed winch gears • Periodically lubricate the gear teeth and wire ropes • Check and tighten covers on all winch bushings • Confirm flow control equipment is capable of removing all logs from each sluice under high flow conditions	Short Medium Medium Short	$5k By PCA Staff By PCA Staff By PCA Staff
Sluice Covers and Crossers	• Missing protective coating in some areas	• Maintain protective coatings	Long	By PCA Staff
Winch Rails	• Not adequately secured to the dam deck and not well aligned	• Realign and properly secure rails to the dam deck	Medium	$10k
Access to Dam	• Potential slip and fall hazard to access the deck via a steep embankment	• Improve foot access by installing stairs to eliminate slip and fall hazards	Short	$30k
Fall Restraint System	• Operators work over open sluices, no fall restraint system exists on the deck for logging operations	• Install engineered restraint system and provide Operators with training to use the system	Medium	$30k
Routine Inspections	• Currently Routine Inspections are not being carried out.	• Carry out monthly routine inspections by trained PCA staff.	Short	By PCA Staff

STRUCTURE / ITEM	DEFICIENCIES / ISSUES	RECOMMENDATIONS	PRIORITY[1]	COST[2],[3]
Engineering Inspections	• Currently Engineering Inspections are not being carried out.	• Carry out annual Engineering Inspections in accordance with the PCA Directive.	Short	$8k
Operation, Maintenance and Surveillance (OMS) Manual; Emergency Preparedness Plan (EPP): and Emergency Response Plan (ERP)	• No OMS Manual • No EPP • No ERP	• Prepare OMS Manual • Prepare EPP • Prepare ERP	Short Short Short	$40k $30k $30k
Flood Forecasting	• Not clearly documented	• Document and consider adding real-time rainfall data and flood modelling.	Long	N/A
Guardrails	• The downstream side guardrails of the dam do not meet COHSR nor NBCC nor CHBDC. • Guardrails are lacking on d/s side of the south bulkhead stairs and at d/s side of stairs landing.	• Assess most appropriate upgrade of guardrails • On the d/s side of the south bulkhead stairs and stairs landing install a guardrail	Medium	$30k (replace all guardrails)
Safety Booms	• No safety booms are located upstream or downstream of the dam	• Install safety booms upstream and downstream of the dam	Medium	$100k

STRUCTURE / ITEM	DEFICIENCIES / ISSUES	RECOMMENDATIONS	PRIORITY[1]	COST[2],[3]
Signage	• Existing signs are not in accordance with CDA	• When safety booms are installed, install signage at anchors • Replace existing signage with signage that meets CDA requirements	Medium Medium	$10k $10k
Training	• No training programs exist for Operators.	• Implement training programs regarding dam safety and dam operations and maintenance	Medium	N/A
LOCK 39				
Lock Chamber Walls	• Do NOT meet stability requirements based on assumed dimensions.	• In short term – monitor • In long term – carry out one or more of the following options: - Confirm "as-built" cross section dimensions and reassess stability. - Enhance stability with tie-back anchors and/or by adding mass to wall.	Short Long	By PCA Staff $250k (tie-back anchors)

STRUCTURE / ITEM	DEFICIENCIES / ISSUES	RECOMMENDATIONS	PRIORITY[1]	COST[2],[3]
	• Concrete Deterioration - Concrete repairs are failing in the vicinity of the lock operating mechanisms - Erosion of the concrete at the foot of the walls and at high water level - Spalled concrete in a number of areas of the walls	• Repair	Medium	$100k
Approach Channel Walls	• Areas of spalled concrete, fractures, erosion along joints • Erosion at high water line	• Repair • Monitor and repair may be eventually required	Medium Long	$50k $50k
Walls D/S of Lock Chamber	• Erosion at high water line • Bulkheads are map cracked with white deposits (possible AAR). Some concrete spalling, particularly just below the concrete cap • The north wall has areas of spalled/fractured concrete	• Monitor and repair may be eventually required • Monitor and repair may be eventually required • Repair	Long Long Medium	- - $60k
Lock Gate Leaves	• Condition of vertical tie rods is unknown	• Perform NDE and detailed visual inspection of the tie rods	Medium	$10k

STRUCTURE / ITEM	DEFICIENCIES / ISSUES	RECOMMENDATIONS	PRIORITY[1]	COST[2],[3]
Lock Gate Actuators	• Operators must walk close to the edge of lock deck while operating gate actuators posing a potential fall hazard into the lock	• Consider alternative means to operator the actuators (e.g. electric motors) to eliminate need for Operators to walk close to edge of deck	Medium	$40k
	• Excessive play between gate actuator hand wheel hubs and actuator drive shafts is accelerating wear at that interface and presenting an ergonomic issue	• Eliminate excessive play	Medium	$10k
	• Actuators require a high level of force to get moving from stopped position	• Assess gate gudgeons, pintles and gate actuators to determine root cause of high force required	Long	TBD
	• Vandals can operate gates and remove actuator handles when PCA staff not present	• Modify actuators to secure handles and allow actuator locking when staff not present	Short	$5k
Lock Filling and Draining Valves	• Valve actuator gears not fully guarded	• Install guarding over exposed actuator gears	Short	$5k
	• Two lock filling valves in upstream lock gate leak badly	• Repair leaking valves	Medium	TBD
	• Vandals can operate valves and remove valve actuator handles when PCA staff not present	• Remove handles when staff not present	Long	By PCA Staff

STRUCTURE / ITEM	DEFICIENCIES / ISSUES	RECOMMENDATIONS	PRIORITY[1]	COST[2],[3]
Guardrails	• Gap between guardrails on walkway • Deficient	• In the short term install barrier to close gap. • Replace existing guardrails along the walkways over the locks and install new guardrails along lock walls in accordance with CAN/CSA S6-06 or NBCC.	Short Medium	$10k $80k
Equipment Maintenance	• No documented formal maintenance program or documented maintenance records exist resulting in unexpected operating interruptions from 1 hour to 1 week in duration	• Prepare a maintenance program and commence documenting of maintenance activities	Medium	TBD
CULVERT				
Outlet and Inlet and Wing Walls	• Deteriorated concrete at joints with wing wall and at exit and entrance to culvert	• Repair	Long	$70k
Culvert	• Corrugated steel arch section has deterioration and undermining of the concrete support wall	• Repair	Long	$50k
CANAL EARTHFILL EMBANKMENTS AND CONCRETE RETAINING WALLS				
Concrete Retaining Walls	• Both walls are extremely deteriorated with voids, erosion, spalling concrete and fractures	• Major repair or replacement	Medium	$350k

STRUCTURE / ITEM	DEFICIENCIES / ISSUES	RECOMMENDATIONS	PRIORITY[1]	COST[2],[3]
Gabion Baskets	• Gabion baskets are rotated, damaged and deformed	• Monitor in short term and repair within the next 10 years	Medium	$150k
U/S and D/S Southeast Embankment Slopes	• Do NOT meet the CDA slope stability requirements	• Modify the U/S and D/S slopes to increase the stability of the slopes ; or • Implement a monitoring program; or • PWGSC / PCA assess the needs to or risks not to increase stability.	Medium	$150k
Southeast Embankment between STA 0+050 and 0+250	• Disturbances which may be man-made.	• Monitor and eventually repair if required	Long	N/A
Embankments	• Vegetation on the embankments hinders the ability to inspect/monitor the condition of the embankments and can have negative impact on performance of embankments	• Clear the vegetation	Short	$75k
RIGHT (NORTH) EARTHFILL EMBANKMENT AND DAM ABUTMENT				
Concrete Core Wall	• IDF water level will overtop core wall • On u/s slope a 6 m long section of the core wall is exposed	• Increase core wall height. • Cover exposed core wall with earthfill	Medium	$100k

STRUCTURE / ITEM	DEFICIENCIES / ISSUES	RECOMMENDATIONS	PRIORITY[/1/]	COST[/2/,/3/]
U/S and D/S Slopes	• Do NOT meet the CDA slope stability requirements	• Modify the U/S and D/S slopes to increase stability; or • Implement a monitoring program; or • PWGSC / PCA assess the needs to or risks not to increase stability.	Medium	$140k
Embankment and Abutment	• Vegetation can have negative impact on performance of embankments and abutments	• Clear the vegetation	Short	$30k
LEFT (SOUTH) EARTHFILL EMBANKMENT				
Concrete Core Wall	• IDF water level will overtop core wall	• Increase core wall height.	Medium	$30k
U/S and D/S Slopes	• Do NOT meet the CDA slope stability requirements	• Modify the U/S and D/S slopes to increase stability; or • Implement a monitoring program; or • PWGSC / PCA assess the needs to or risks not to increase stability.	Medium	$30k
Embankment and Abutment	• Vegetation can have negative impact on performance of embankments and abutments	• Clear the vegetation	Short	$15k

[/1/]PRIORITY IDENTIFICATION:

Short: Work that needs to be done either to upgrade the structure or to meet current regulations and dam safety requirements. Generally, it is the result of an identified Deficiency and needs to be attended to within the next 2 years.

Medium: These deficiencies may include additional work that could improve the performance or issues that may become dam deficiencies. These items should be addressed before the next Dam Safety Review.

Long: These are issues which could enhance the performance of the dam or may also be opportunities for improvement. These issues are not currently considered to be urgent and can be scheduled at PWGSC/PCA's convenience.

/2/ Costs include engineering and construction costs but do not include PCA internal costs for training, engineering reviews, and project management. TBD = To be determined (further studies are required).

/3/ Assume all recommended repair work is carried out with one mobilization.

10.0 ASSET CONDITION RISK CLASSIFICATION

An Asset Condition Risk Classification has been completed for the purpose of assisting PCA decision making for the investment planning for future rehabilitation of PCA's dam infrastructures. The Asset Condition Risk Classification is provided under a separate document.

11.0 CONCLUSIONS

This section summarizes and concludes the findings of the DSR for the Talbot Dam and Lock 38 site. Currently, this site does not meet the regulatory requirements.

11.1 HYDRAULICS / HYDROLOGY

- The summer and spring PMF for the Talbot River Dam were estimated to be equal with a peak inflow to Talbot River Dam of 344 m^3/s.
- The estimated incremental loss of lives obtained from dam break simulations of the Talbot River Dam is in the range of 1 to 10, for a sunny day failure and for a breach during a flood.
- Based on the incremental consequences of a dam breach, the Talbot River Dam is classified as HIGH A, in accordance with the PCA Directive.
- The recommended IDF for the Talbot River Dam is the flood 37% of the way between the 1000-year flood and the PMF. A spring event of that magnitude would have a peak inflow to Talbot Dam of 219 m^3/s. The corresponding summer event would have the same peak flow.
- The Design Basis Earthquake (DBE) at Talbot Dam is the 2.500 year earthquake.
- The estimated discharge capacity of the dam, at the FSL (El. 230.44 m), is approximately 160 m^3/s. The estimated level at the dam required to pass the peak of the IDF is El. 231.03 m.
- Assuming all 11 stoplogs can be removed from both sluices to pass the IDF, the discharge capacity at the dam is adequate to pass the IDF with the minimum CDA required freeboard for the sluiceway and Lock 39 but the concrete core walls of the Right and Left earthfill embankments would be overtopped.
- PCA has noted that the capability of the existing flow control equipment to remove the stoplogs may be limited to a maximum of 8 stoplogs per sluice. In that case, 3 stoplogs would remain in each of the two sluiceway bays. The discharge capacity would be then limited to 120 m^3/s if the forebay level is at the FSL and 195 m^3/s if it is at the top of the earth embankment.
- The dam satisfies the requirement for minimum freeboard, indicated in the CDA Guidelines, during non-flood conditions.
- There are not established rules for emergency operation of the dam and lock. These are critical because the time available to operate the dam during a flood is limited and because

access for operation of the sluiceway is across the lock embankments and Left dam abutment.

11.2 CIVIL STRUCTURES

In general, the concrete structures are in good condition but warrant monitoring and some repair work is required to be carried out within the next 5 years as noted in Sections 4.2 and 9.3.

Gravity Bulkheads, Spillway, Sluiceways, and Lock 39 Chamber Walls: The results of the stability analyses show that the structures fail to meet CDA stability criteria. Table 11.2-1 provides a summary of the loading cases that the structures do not meet the stability critieria.
Stoplogs and Lock Gate: The results of the analyses show that the structures have inadequate structural capacities.

TABLE 11.2-1
STABILITY RESULTS SUMMARY (ASSUMED ZERO COHESION)

STRUCTURE	Ø'	LOADING CONDITION			
		NORMAL	WINTER	IDF	EQ
Gravity Bulkheads	36^0	×	×	✓	✓
	45^0				
Spillway	36^0	×	×	✓	×
	45^0				
Sluiceway	36^0	✓	×	✓	✓
	45^0				
Lock #38 Chamber	36^0	×	N/A	N/A	×
	45^0				
	45^0	✓			✓

Notes: × = *does not meet stability requirements*.
✓ = *meets stability requirements.*

11.3 GEOTECHNICAL ASPECTS

Condition Assessment

In general, the geotechnical performance of the Talbot River Dam and Lock 39 structures has been good and the geotechnical performance of the Talbot Canal Embankments has been fair with some concerns identified. The following issues were noted:

- ***Right Talbot River Dam Embankment***: The concrete core wall is exposed on the upstream side due to erosion and/ or local shallow sloughing of the upper portion of the slope. The exposed length of core wall was approximately 6 m long and located approximately 115 to 121 m from the North Abutment of the Spillway. The concrete core wall exposed area currently does not appear to be jeopardizing the stability of the embankment in the short term. If additional distress or material loss is observed, then temporary protection measures such as placement of riprap material or fill buttressing may be required.
- ***Northwest and Southeast Canal Earthfill Embankments:*** Loss of the upper slope, deterioration of the gabion baskets, and erosion of the inner (wet) slope has been ongoing. Ongoing erosion on the upstream slope (wet side) is anticipated to be a relatively slow process as there are no significant waves except the waves associated with boat traffic. The overall risk of breach of the canal embankment is relatively low considering the relatively low height of the embankments. However, the embankment cannot be considered to remain stable over the long-term without repair.
- **Vegetation:** The current tree growth and vegetation on the embankments and dam abutments is not an immediate threat to their performance but PCA should be proactive and remove the tree growth and vegetation on the canal and dam embankments and dam abutments.

Slope Stability Analyses

- ***Right Talbot River Dam Earthfill Embankment***: The slope stability analyses determined that the estimated safety factors for the more critical deep seated overall potential slip surfaces are greater than unity for all cases other than the IDF condition with consideration of the lower bound of the embankment material shear strengths. However, the Right earthfill embankment does not meet the CDA recommended minimum stability criteria and its slope stability factor of safety for its downstream slope is less than unity for the IDF condition. Remedial work to improve the estimated factor of safeties to satisfy the stability design criteria should be considered as medium priority.
- ***Left Talbot River Dam Earthfill Embankment***: The embankment generally meets the CDA recommended minimum stability criteria with an exception for the “normal” condition at the upstream side based on the assumed lower bound material shear strengths and for the IDF

condition at its downstream side. The estimated safety factor for its upstream side is still reasonably high for the "normal" condition (1.44).

- ***Northwest Canal Earthfill Embankment***: The stability analyses determined that the estimated safety factors were greater than unity for all cases based on using the lower bound material shear strengths in the analyses but did not meet the CDA recommended minimum stability criteria for the "normal" condition. The CDA stability criteria are satisfied based on using the upper bound materials shear strengths in the analyses.
- ***Southeast Canal Earthfill Embankment***: The Southeast canal earthfill embankment section between STA. 0+450 to STA. 0+950 does not meet the CDA recommended minimum stability criteria. However, the stability analyses determined that the estimated safety factors were greater than unity for all cases using the lower bound material shear strengths with an exception of the rapid drawdown case which is less than unity (FS=0.91). However, the rapid drawdown condition may not be applicable as the PCA drawdown operation happens gradually over a long period of the time (i.e., several weeks). Remedial work to improve the estimated factor of safeties of the above noted area of the embankment to satisfy the stability design criteria should be considered as medium priority.
- ***Lock 39 Embankments***: Detailed geotechnical analyses on the outer slopes of the lock embankments were not considered necessary. The Lock 39 earthfill embankments have performed well and no visual in-stability has been observed on these outer slopes. Given the low height and wide crest width, potential slip surfaces that could affect the lock walls have fairly high estimated safety factors, which well exceed the minimum recommended CDA stability criteria.

TABLE 11.3-1
SLOPE STABILITY RESULTS SUMMARY – EARTHFILL EMBANKMENTS

Case	Loading Condition	Slope	Estimated Factor of Safety (Lower bound material properties)	Estimated Factor of Safety (Upper bound material properties)
Right Earthfill Embankment				
I	Normal operating Condition (Long-term Steady State Seepage)	U/S	×	✓
		D/S	×	×
II	Extreme Short-term Earthquake	U/S	✓	✓

Case	Loading Condition	Slope	Estimated Factor of Safety (Lower bound material properties)	Estimated Factor of Safety (Upper bound material properties)
	Loading	D/S	✓	✓
III	Rapid Drawdown	U/S	×	✓
IV	IDF Condition/1/	U/S	✓	✓
		D/S	×	×
Left Earthfill Embankment				
I	Normal operating Condition (Long-term Steady State Seepage)	U/S	×	✓
		D/S	✓	✓
II	Extreme Short-term Earthquake Loading	U/S	✓	✓
		D/S	✓	✓
III	Rapid Drawdown	U/S	×	✓
IV	IDF Condition/1/	U/S	✓	✓
		D/S	×	×
Northwest Canal Earthfill Embankment				
I	Normal operating Condition (Long-term Steady State Seepage)	U/S	✓	×/✓
		D/S	✓	✓
II	Extreme Short-term Earthquake Loading1	U/S	✓	✓
		D/S	✓	✓
III	Rapid Drawdown	U/S	×	✓
IV	IDF Condition	U/S	✓	✓
		D/S	✓	✓
Southeast Canal Earthfill Embankment				
I	Normal operating Condition (Long-term Steady State Seepage)	U/S	×	✓
		D/S	×	×
II	Extreme Short-term Earthquake Loading	U/S	✓	✓
		D/S	✓	✓
III	Rapid Drawdown	U/S	×	×
IV	IDF Condition	U/S	✓	×/✓
		D/S	×	×

Notes /1/ Concrete core wall will be overtopped resulting in potential damage/washout of the earthfill.
× = *does not meet stability requirements*.
✓ = *meets stability requirements.*
×/✓ = marginally meets stability requirements.

11.4 MECHANICAL EQUIPMENT

11.4.1 Talbot River Dam

- The mechanical equipment at the dam is generally in fair condition with issues as summarized below:
  - Winch gear teeth have minor wear and are not lubricated.
  - Winch wire ropes are not lubricated.
  - Winch gears are not fully guarded.
  - The ends of one of the logs is slightly damaged.
  - A bushing cover on the hand crank shaft on the north winch is not securely bolted down.
  - Sluice crossers and sluice cover support brackets should be painted yellow to identify these tripping hazards.
  - The rails are not adequately secured to the deck and are not well aligned.
- Witnessing the functional test provided sufficient evidence to confirm that PCA staff are capable of operating the equipment to remove and install stoplogs. However, not all stoplogs were removed during the test, nor was it confirmed that the flow control equipment is adequate to remove all stoplogs under high flow conditions.

11.4.2 Lock 39

The mechanical equipment at the lock is generally in fair with a few issues as summarized below:

- Condition of gate leaf vertical tie rods is unknown since they could not be accessed for inspection.

- Excessive play between the gate actuator hand wheel hubs and the actuator drive shafts is accelerating wear at this interface and also presenting an ergonomic problem.
- Gate actuators require a high level of force to get them moving from the stopped position.
- Two lock filling valves in the upstream gate leak badly.
- Lock filling and draining valve actuator gears and pinions are not fully guarded.
- There are opportunities for vandals to actuate the lock gate leaves, operate the lock fill and drain valves, remove lock fill valve actuator handles and remove gate actuator hand wheels.

Witnessing the operation of the lock gates and lock fill valves provided sufficient evidence to confirm that PCA staff are capable of operating the equipment and that the equipment is adequate. The only concern noted by the Operators is that the actuators require a high level of force to get the gates moving from a stopped position. It was observed that Operators must repeatedly walk close to the edge of the lock deck while operating the gate actuators posing a potential fall hazard into the lock.

11.5 DAM OPERATIONS, MAINTENANCE, AND SURVEILLANCE

- There have been no employee or contractor safety incidents in the past 10 years at this site.
- For logging operations the Operators work over open gains without an engineered fall restraint system creating a falling hazard.
- Typically, sufficient staff are available to carry out the log and lock operations for the Talbot River Dam and Lock 39. However, during a flood event, combined with the requirements for the same staff to operate Lock 39 and Talbot Dam and Lock 38, it could be challenging for Operators to respond to operate the Talbot River Dam as well.

In a major storm event, the Northern Sector Operators may not be able to operate 28 dams in an optimum time frame to minimize damages from the flood event. As a storm event could affect neighbouring Districts, the availability of nearby additional support to the Northern Sector Operators might be limited.

No formal training is given to Operators; rather skills are developed on the job, with the junior Operators learning from the senior Operators. A training program with respect to dam safety, dam operations, and the PCA Standing Orders could be beneficial.

The Talbot River Dam and Lock 39 site does not have an OMS Manual, EPP, and ERP but are required as per the PCA Directive and CDA Guidelines.

Currently no inspection programs are implemented for the site. As per the PCA Directive, routine monthly inspections and annual engineering inspections are required to be carried out. Dam Safety Reviews should be carried out for this site every 5 years.

11.6 PUBLIC SAFETY

- There have been no public safety incidents in the past 10 years at this site
- A public safety assessment has been carried as part of this DSR and is reported in a separate document.
- In general, as discussed in Section 8.0, public safety deficiencies include:
  - Inadequate guardrails/barriers.
  - No public safety booms are installed upstream or downstream of the dam.
  - Signage is not in accordance with the CDA Guidelines.

12.0 STATEMENT OF LIMITATIONS AND CONDITIONS

12.1 THIRD PARTY USE OF REPORT

This report has been prepared for the Client to whom this report has been addressed and any use a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. KGS Group accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions undertaken based on this report.

12.2 CAPITAL COST ESTIMATE STATEMENT OF LIMITATIONS

The cost estimates included with this report have been prepared by KGS Group using its professional judgment and exercising due care consistent with the level of detail required for the stage of the project for which the estimate has been developed. These estimates represent KGS Group's opinion of the probable costs and are based on factors over which KGS Group has no control. These factors include, without limitation, site conditions, availability of qualified labour and materials, present workload of the Bidders at the time of tendering and overall market conditions. KGS Group does not assume any responsibility to the Client, in contract, tort or otherwise in connection with such estimates and shall not be liable to the Client if such estimates prove to be inaccurate or incorrect.

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14.0 GLOSSARY OF TERMS

Abutment	That part of the valley side or other supporting structure against which the dam is constructed.
Annual exceedance probability	The probability that an event of specified magnitude will be equal to or exceeded in any year.
Classification	A system of assigning dams to categories, usually on the basis of the hazards or the consequences of failure, so that appropriate dam safety standards can be applied.
Consequences of failure	Impacts on the downstream or upstream area of a dam as a result of failure of the dam or its appurtenances. In this Directive, the term “consequences” refers to the damage above and beyond the damage that would have occurred in the same event or conditions had the dam not failed. These may also be called incremental consequences of failure.
Dam	A barrier constructed for the retention of water, water containing any other substance, fluid waste, or tailings, provided the barrier is capable of impounding at least 30 000 m3 of liquid and is at least 2.5 m high. Height is measured vertically to the top of the barrier: (i) from the natural bed of the stream or watercourse at the downstream toe of the barrier, in the case of a barrier across a stream or watercourse; or (ii) from the lowest elevation at the outside limit of the barrier, in the case of a barrier that is not across a stream or watercourse. In this Directive, the term “dam” includes appurtenances and systems incidental to, necessary for, or connected with the barrier. The definition may be expanded to include dams less than 2.5 m high or with an impoundment capacity of less than 30 000 m3 if the consequences of dam operation or failure are likely to be unacceptable to the public, such as dams that create hydraulic conditions posing a danger to the public; dams with erodible foundations that, if breached, could lower the reservoir by more than 2.5 m; or dams retaining contaminated substances. If natural ground or bedrock exists or outcrops significantly at one or several locations along the crest, each section is considered a separate dam. Structures not separated by a natural barrier that impound water at different levels, such as multiple lifts locks, are classified as separate dams for dam safety purposes.
Dam safety review	A comprehensive, formal review carried out at scheduled intervals to determine whether an existing dam is safe, and if it is not safe, to determine what improvements are required.
Earthquake design ground motion	The level of earthquake ground motion at the location for which a dam structure is designed or evaluated.

Failure (of a dam)	An uncontrolled release of the contents of the reservoir.
Foundation	The rock and (or) soil mass that forms a base for the structure, including its abutments.
Freeboard	The vertical distance between the still water surface elevation in the reservoir and the lowest elevation at the top of the containment structure.
Hazard	A system state or set of conditions that together with other conditions in the system environment could lead to a partial or complete failure of the system. Hazards may be external (originating outside the system) or internal (errors and omissions or deterioration within the system).
Headwater	The water upstream from a structure or point on a stream.
Incremental consequences of failure	The incremental losses or damage that a dam failure might inflict on upstream areas, on downstream areas, or at the dam itself, over and above any losses or damage that would have occurred in the same event or conditions had the dam not failed.
Inflow design flood	The most severe inflow flood (peak, volume, shape, duration, timing) for which a dam and its associated facilities are designed.
OMS manual	An Operation, Maintenance and Surveillance (OMS) manual that documents the requirements and procedures for the safe operation, maintenance, and surveillance of a dam.
Probable maximum flood	An estimate of a hypothetical flood (peak flow, volume, and hydrograph shape) that is considered the most severe that is “reasonably possible” at a particular location and time of year. The estimate is based on a fairly comprehensive hydro meteorological analysis of critical runoff-producing precipitation (snowmelt if pertinent) and hydrologic factors favourable for maximum flood runoff.
Reservoir	The body of water, fluid waste, or tailings that is impounded by a dam, including its shores and banks and any facility or installation necessary for its operation.
Risk	A measure of the probability and severity of an adverse effect on health, property, or the environment. Risk can be estimated by the expectation of the consequences of an adverse event occurring (that is, the product of the probability of occurrence and the consequence).
Spillway	A weir, channel, sluice, conduit, tunnel, chute, gate, or other structure designed to permit discharges from the reservoir.

Surveillance	The close monitoring of dam behaviour, including systematic collection, analysis, and interpretation of data through visual inspections and instrumentation.
Tailwater	The water in the discharge channel immediately downstream of dam.
Tremie	Method that uses a pipe, through which concrete is placed below the water level.
Water-retaining structure	A barrier constructed for the retention of water, water containing any other substance, fluid waste, or tailings that does not fall under the definition of dam.

15.0 LIST OF ACRONYMS AND ABBREVIATIONS

AEP	Annual Exceedance Probability
CDA	Canadian Dam Association
CEATI	Centre for Energy Advancement through Technological Innovation.
COFU	Central Ontario Field Unit
CSA	Canadian Standards Association
DBE	Design Basis Earthquake
DEM	Digital Elevation Model
DSGI	Dam Safety General Inspection
DSR	Dam Safety Review
EFPC	Elliott Falls Power Corporation
EPP	Emergency Preparedness Plan
ERP	Emergency Response Plan
EGL	Energy Grade Level
FERC	U.S. Federal Energy Regulatory Commission
FSL	Full Supply level
GIR	Geographical Information Retrieval
GSC	Geological Survey of Canada
HPC	Hazard Potential Classification
ICC	Incremental Consequence Category
IDF	Inflow Design Flood
ILOL	Incremental Loss of Life
NBCC	National Building Code of Canada
NHSC	National Historic Site of Canada
OBC	Ontario Building Code
OGS	Ontario Geological Survey
OHSA	Occupational Health and Safety Act
OMNR	Ontario Ministry of Natural Resources
OMS	Operation Maintenance and Surveillance
PAR	Population at Risk
PCA	Parks Canada Agency
PGA	Peak Ground Acceleration
PMF	Probable Maximum Flood
PMP	Probable maximum Precipitation
PWGSC	Public Works and Government Services Canada
SSF	Sliding Stability Factor
TSW	Trent-Severn Waterway
USBR	United Sates Bureau of Reclamation
USACE	U.S. Army Corps of Engineers
WSC	Water Survey of Canada
WRIP	Water Resources Information Project

APPENDICES

APPENDIX A

GEOTECHNICAL INVESTIGATION DRILLING REPORT

Public Works and Government Services Canada

Travaux publics et Services gouvernementaux Canada

Talbot River Dam, Lock 39 and Talbot Canal 2013 Geotechnical Site Investigation Report

FINAL

KGS Group 12-0006-028
August 2014

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1.0 INTRODUCTION

1.1 BACKGROUND

KGS Group was retained by Public Works and Government Service Canada (PWGSC) to complete a geotechnical investigation for the Talbot River Dam, Lock 39 and Talbot Canal. The purpose of the investigation was to obtain geotechnical data to determine the material types and characteristics of the embankment fill and the foundations for use in the dam safety review (DSR) currently underway. Monitoring well and vibrating wire piezometers were also installed to measure the piezometric level within the embankments. The embankment material properties and the piezometric data will be incorporated into the evaluation of the geotechnical performance of the structure as part of the Dam Safety Review (DSR) being undertaken for PWGSC. Any investigations needed to complete final design of remedial measures that may be recommended from the DSR is separate and not included.

The site is located on the Talbot River near the town of Bolsover, Ontario (approximately 7 km west of Bolsover), and 1.2 km downstream from Lock 38 / Talbot Dam. The site is accessible from Canal Road from Hwy 50. The Talbot River Dam is a concrete gravity dam founded on bedrock and built in 1908 to maintain the navigation levels on the waterway and provide control of the watershed. There are two earthfill embankment structures associated with the Talbot River Dam. The Right Earthfill Embankment is 114 m long and consists of earthfill with a concrete core and an additional 14 m long end section with sheet pile core. The Left Earthfill Embankment is a part of the Northwest Earthfill Embankment of the Talbot Canal and consists of earthfill with a 12 m long concrete core wall from the concrete dam structure. Lock 39 is also a concrete gravity structure founded on timber piles and was constructed in 1908. Talbot Canal runs from Talbot River Dam to Lock 39, and includes two earth embankments (Northwest and Southeast Canal Earthfill Embankments) on either side of the nearly 1400 m long canal. The locations of the earthfill and concrete structures are shown in Figures 2.1a, 2.1b and Figure 2.1c.

Review of the existing documentation has shown that there was limited to no information on the earthfill materials used as backfill for the embankments and concrete walls. Initial / preliminary analyses of the earthfill embankments and concrete structures were completed as part of the DSR to evaluate the stability of the structures with variable assumptions on backfill materials

and piezometric levels. These analyses have shown that the stability conditions are very sensitive to the geotechnical assumptions, and this site investigation was performed to better identify the geotechnical conditions for incorporation into the DSR and better represent the actual stability conditions.

This geotechnical program was undertaken to provide key fill and foundation properties for incorporation into the DSR. However, the program was not considered extensive, and investigation activities that could be potentially required for design of any remedial work have not been included in this program.

A detailed scope of KGS Group's services is summarized below:

- Drilling of exploratory test holes, soil sampling and laboratory testing;
- Excavating test pits;
- Determination of embankment fill materials, foundation soils and their engineering characteristics;
- Assessment of the foundation conditions of the embankments;
- Installation of an automated piezometer system for monitoring the porewater pressure within the earthfill embankments.

2.0 2013 SITE INVESTIGATION PROGRAM

The 2013 site investigations program was undertaken to obtain the geotechnical data required to determine the embankment material types and engineering properties of the embankment fills and foundations, and to install monitoring wells and vibrating wire piezometers to measure the piezometric levels. The embankment material properties and the piezometric data were then incorporated into the evaluation of the geotechnical performance of the structure as part of the DSR.

The geotechnical exploratory program consisted of the following:

- Drilling of a total of fifteen (15) test holes along the crest of the embankments to the underlying bedrock and / or native soil to determine the composition and properties of the embankment fill materials and to assess the foundation conditions.
    - Six (6) test holes at Talbot River Dam: one (1) at Left (South) Earthfill Embankment and five (5) test holes at Right (North) Earthfill Embankment.
    - Two (2) test holes at the north side of the Lock 39.
    - Seven (7) test holes at the Talbot Canal Earthfill embankments: four (4) along the Northwest Canal Earthfill Embankment and three (3) test holes along the Southeast Canal Earthfill Embankment.
- Excavating four (4) test pits along the upstream side of the earthfill embankments of the Talbot Canal.
- Installation of three (3) monitoring wells (standpipes) and two (2) vibrating wire (VW) piezometers in the embankments;
    - One (1) VW piezometer and one (1) standpipe at Right (North) Earthfill Embankment of Talbot River Dam.
    - One (1) VW piezometer and one (1) standpipe at the north side of the Lock 39 structure.
    - One (1) standpipe at the Northwest Canal Earthfill Embankment at Talbot Canal.
- Completing laboratory index testing and advanced soil testing on representative embankment materials and the foundation soils.

Health and Safety and Environmental Protection Plans were prepared and submitted to PWGSC for review and approval prior to any work being conducted on site. Public Works and

Government Service Canada (PWGSC) and Parks Canada Agency (PCA) staff conducted an internal environmental assessment of the work plan, and confirmed the requirements for the field operation.

2.1 TEST HOLE DRILLING AND SAMPLING PROGRAM

The drilling services were provided by Ohlmann Geotechnical Services Inc. (OGS) of Almonte, Ontario. Laboratory testing of selected soil samples was completed at the Golder Associates Soils Laboratory in Mississauga, Ontario.

The drilling of the test holes and soil sampling was completed between June 5 and June 9, 2013 under continuous supervision and direction of KGS Group. Eight (8) test holes (TH13-105, TH13-201 and 202, and TH13-301 to 305) were readily accessible and were drilled using a bombardier mounted drill rig (CME75) equipped with continuous 200 mm diameter hollow stem augers and an automatic hammer, as shown in Photo 1. The other seven (7) test holes (TH13-101 to TH13-104, TH13-104a, and TH13-306 to 307) were not accessible by vehicle and were drilled using a standard portable-type drill rig equipped with casing-wash boring and a manual hammer, as shown in Photo 2.

All test holes were advanced to the underlying bedrock or native soils, as indicated in Table 2.1-1. The approximate locations of the test holes and test pits are shown in Figures 2.1a, 2.1b and 2.1c. Standard Penetration Tests (SPT's) were performed as part of the drilling program to estimate the in-situ density of the embankment fill and foundation soils, and disturbed soil samples were recovered from the sampler. All samples were visually classified in the field according to the Unified Soil Classification System (USCS) and representative samples were selected and bagged for laboratory testing. A Shelby tube sample of clayey soil was recovered within TH13-105 for a Consolidated Undrain Triaxial test. Detailed borehole log records that contain all field observations, soil descriptions, groundwater levels and laboratory test results are included in Appendix A.

TABLE 2.1-1
TALBOT RIVER DAM, LOCK 39 AND TALBOT CANAL - SUMMARY OF 2013 DRILLING PROGRAM

Structure	Test Hole #	Location	Total Depth Drilled (m)	Fill Thickness (m)*
Talbot River Dam	TH13-101 (Crest-D/S side)	Right Earthfill Embankment	6.55	5.5
	TH13-102 (Crest-U/S side)		6.02	5.8
	TH13-103 (Crest-D/S side)		6.02	5.8
	TH13-104 (Crest-U/S side)		2.13	2.1+
	TH13-104a (Crest-D/S side)		6.40	6.4
	TH13-105 (Crest-U/S side)	Left Earthfill Embankment	7.01	5.8
Lock 39	TH13-201 (Crest-U/S side)	North side	10.97	9.5
	TH13-202 (Crest-U/S side)		10.97	9.5
Talbot Canal	TH13-301 (Crest-U/S side)	Northwest Earthfill Embankment	6.55	±3.5
	TH13-302 (Crest-D/S side)		5.03	±3.2
	TH13-303 (Crest-U/S side)		5.03	±3.1
	TH13-304 (Crest-U/S side)		5.03	±3.3
	TH13-305	Southeast Earthfill Embankment	5.03	±3.1
	TH13-306 (Crest)		6.71	±3.0
	TH13-307 (Crest-U/S side)		4.88	±2.3

* Fill thickness was approximated based on visual inspection of the test holes and samples recovered, and was sometimes difficult to distinguish due to the similarity to the underlying native materials.

PHOTO 1
CME 75 DRILL RIG AT TH13-202

PHOTO 2
ELECTRICAL PORTABLE DRILLING EQUIPMENT AT TH13-101

FIGURE 2.1A
APPROXIMATE TEST HOLE LOCATIONS – TALBOT RIVER DAM

FIGURE 2.1B
APPROXIMATE TEST HOLE LOCATIONS – LOCK 39

FIGURE 2.1C
APPROXIMATE TEST HOLE AND TEST PIT LOCATIONS – TALBOT CANAL

2.2 TEST PITTING

The PWGSC Terms of Reference document for the DSR noted that a steel sheet pile wall was installed along the crest of both embankments at the upstream portion, however no background documentation or drawings were available to confirm the existence of the sheet piles and PCA personal could not confirm that sheet piles were installed in these embankments. Test pitting (TP-1 to TP-4) was performed using a rubber tire backhoe at four select locations along the canal embankments to try and identify whether the sheet piles were installed. The test pit locations are shown in the Figure 2.1c.

2.3 INSTRUMENTATION - PIEZOMETER INSTALLATION

As described in Section 2.0, three (3) standpipes (in TH13-101, -202 and -304) and two (2) VW piezometers (in TH13-101 and -202) were installed to monitor the groundwater conditions within the embankments and their foundations. Installation details, including the tip elevations of the piezometers, are shown on the summary test hole logs in Appendix A.

The standpipe requires manual reading to measure the groundwater level. The vibrating wire piezometers were connected to automated data loggers to continuously monitor piezometric levels and temperature within the embankment fills and the foundation soils. Detailed descriptions of the logger are provided in the following section. All piezometer readings collected to September 2013 are presented in Section 3.4.

PHOTO 3
INSTALLATION OF STANDPIPE PIEZOMETER AT TH13-202

PHOTO 4
DT 2011 VIBRATING WIRE PIEZOMETER INSTALLED AT TH13-202

PHOTO 5

INSTALLATION OF VIBRATING WIRE PIEZOMETER AT TH13-202

2.3.1 Single Channel Vibrating Wire Data Loggers

A Single Channel Data Logger Model DT2011 (Figure 2.3-1), manufactured by RST instruments, was installed on each of the piezometers in TH13-202 and TH13-105 in the earthfill embankments. The vibrating wire piezometer and battery powered data logger are economic and designed for reliable, unattended and continuous monitoring of a single vibrating wire sensor. The logger is encased in a water resistant IP 66 enclosure and connects directly to the vibrating wire piezometer via an IP68 rated sealed entry. The DT2011 loggers run on two (2) “AA” alkaline batteries and have an internal memory of 128KB that can store over 30,000 data sets of frequency and temperature. The operating temperature of the recorder is between –40°C to 60°C. Periodic battery changing and downloading of the data is typically performed two times per year.

FIGURE 2.3-1
DT2011 DATA LOGGER WITH VIBRATING WIRE PIEZOMETER

The DT2011 loggers are controlled and programmed with the Windows-based LogViewer® Software. Prior to installation, the DT2011 logger was programmed and verified in the field to ensure it was functioning properly. The vibrating wire piezometer was read with a standard readout box and readings were compared to DT2011 values for verification.

2.4 LABORATORY TESTING

Routine index and advanced laboratory testing were performed on select soil samples for use in material classification and characterization, as well as estimation of shear strength parameters. The routine index testing included moisture content testing, grain size analysis and plasticity index testing (Atterberg Limits testing). Advanced laboratory testing included Consolidated Undrained (CU) Triaxial testing for a Shelby tube sample of clayey silt fill and Consolidated Drained Direct Shear testing (DST) on reconstituted samples of sandy silt to silty sand fill obtained from the split spoon. The laboratory testing was completed at Golder Associates Ltd. Soil Laboratory in Mississauga, Ontario, with the results interpreted by KGS Group for use in the geotechnical evaluation of the DSR. The following soil testing standards of the American Society for Testing and Materials (ASTM) were used for the soil testing:

- ASTM D422 - Standard Test Method for Particle-Size Analysis of Soils;
- ASTM D1140 - Test Method for Amount of Material in Soils Finer than the No. 200 Sieve;
- ASTM D2216 - Standard Test Method for Laboratory Determination of Water (Moisture);

- ASTM D4318 - Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity;
- ASTM D3080 - Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions;
- ASTM D4767 - Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils.

3.0 SITE INVESTIGATION RESULTS

The site investigation program was undertaken to assess the engineering characteristics of the embankment fill materials and the foundation soils, and included drilling a total of fifteen (15) test holes and excavating four (4) shallow test pits. Soil stratigraphy and groundwater conditions encountered in each of the exploratory test holes are provided in Appendix A. The results of the laboratory tests completed on selected soil samples are presented in Table 3.5-1 and are also included in the summary logs. The soil classification and description provided herein are in accordance with the Unified Soil Classification System (USCS).

3.1 RIGHT AND LEFT EARTHFILL EMBANKMENTS - TALBOT RIVER DAM

3.1.1 Embankment Fill

The fill materials encountered within the Right and Left Earthfill Embankment consisted mainly of sandy silt to silty sand. Some clayey silt to silty clay fill materials were also encountered. The depth of the embankment fill estimated from test holes drilled along the crest varied from 5.5 to 6.4 m. The crest elevation of the embankments at the test hole locations varied between 231.15 and 231.33 m (approximate bottom of fill at El. 225.8 to 225.0 m±).

- ***Sandy Silt to Silty Sand Fill*** – Sandy silt to silty sand fill was encountered within the earthfill embankments at both the upstream and downstream sides of the Right Earthfill Embankment. The sandy silt to silty sand fill extended from the crest to depths varying between 5.5 m to 6.4 m at the downstream side and 1.8 m to over 2.1 m at the upstream side. The sandy silt to silty sand fill was generally brown, moist to wet, contained trace clay and gravel, and was in a loose to compact state with an average measured (uncorrected) SPT N-value of 5 blows per 300 mm (ranging from 1 to 24 blows per 300 mm). The moisture content ranged from 12% to 29% with an average of approximately 21% according to the laboratory testing. Grain size analyses indicated that the sand/silt fill typically consisted of approximately 50% sand (ranging from 37% to 65%) and 40% silt (ranging from 28% to 46%).
- ***Clayey Silt to Silty Clay Fill*** – Clayey silt to silty clay fill was encountered below the sandy silt to silty sand fill at the upstream side of the Right Earthfill Embankment and from the crest at the upstream side of the Left Earthfill Embankment. The clayey silt to

silty clay fill extended to depths varying between 5.5 to 5.8 m. In general, this fill was typically brown to grey, moist to very moist, very soft to soft in consistency, of intermediate plasticity, and contained trace amounts of fine-grained sand. The measured (uncorrected) SPT N-values were typically at or below 2 blows per 300 mm indicating soft to very soft in consistency. Moisture content ranged from 36% to 43% with an average value of approximately 39%. Atterberg Limits testing indicated that the clayey silt to silty clay was of intermediate plasticity (Liquid Limit from 28% to 48%, Plastic Limit from 14 to 20% and Plasticity Index from 14% to 28%). At 4 m depth in TH13-102, the in-situ moisture content was approximately at the liquid limit. Grain size analyses indicated that the clayey silt to silty clay fill consisted of approximately 42 to 56% silt and 38 to 55% clay.

3.1.2 Sandy Silt Till

The embankment fill was underlain by native soil of glacial origin consisting of sandy silt till which extended below depths varying between 5.5 to 5.8 m (or from El. 225.5 m to 225.8 m) below ground surface to bedrock. The sandy silt till was typically grey in colour, low plasticity to non plastic, moist to wet and contained a varying amount of sand and silt. The measured (uncorrected) SPT N-value of 38 blows per 300 mm to greater than 50 blows per 150 mm (exceeding the SPT refusal criteria), indicating the sandy silt till was generally in a dense to very dense state. The moisture content was about 6% from one laboratory test on one typical till sample. A grain size analysis test indicated that the till consisted of approximately 45% sand, 33% silt, 16% clay and 6% gravel.

3.1.3 Bedrock

Coring of the bedrock was performed at one test hole (TH13-101) located at the Right Earthfill Embankment, and extended approximately 250 mm into intact bedrock. Coring identified that the bedrock was grey limestone below the glacial till at El. 225 m (approximately 6.2 m below the ground surface). Power auger refusal was encountered at a depth of 7.01 m (EL. 224.1 m) at TH13-105 at the Left Earthfill Embankment on presumed sound bedrock. A suspected thin limestone layer was encountered in the SPT sampler at the bottom of the hole (depth of 6.4 m)

at the Right Earthfill Embankment (EL. 224.9 m) at TH13-104a. This also indicated the suspected limestone bedrock.

3.2 NORTH SIDE OF LOCK 39 EARTHFILL EMBANKMENT

3.2.1 Embankment Fill

The fill material encountered within the north side of Lock 39 mainly consisted of clayey silt to silty clay. The clayey silt to silty clay fill (embankment fill) extended from the ground surface to approximately 9.5 m deep to the underlying till (El. 221.9 m). The ground surface of the embankment at the test holes locations varied between 231.33 and 231.40 m. In general, the clayey silt to silty clay fill was typically brown to grey, moist to very moist, soft to very soft in consistency, of low plasticity, and contained trace amounts of fine-grained sand and trace to some topsoil at some depths. The measured (uncorrected) SPT N-values averaged at 3 blows per 300 mm and ranged from 2 to 6 blows/300mm indicating a soft to very soft in consistency. The moisture content ranged from 19% to 39% with an average value of approximately 27%. Atterberg Limits testing indicated that the clayey silt to silty clay was of low plasticity (Liquid Limit averaged at 24%, Plastic Limit averaged at 17% and Plasticity Index averaged at 7%). Grain size analyses indicated that the fill consisted of approximately 74% silt (ranging from 72% to 76%), approximately 16% clay (ranging from 15 to 18%) and approximately 10% sand (ranging from 7 to 13%).

3.2.2 Sandy Silt Till

The embankment fill was underlain by native sandy silt till which was observed at a depth of approximately 9.5 m below grade (El. 221.9 m). The sandy silt till layer extended to a depth of over 11 m below the ground surface (El. 220.4 m) where both test holes were ended. The sandy silt till was typically grey in colour, low plasticity to non plastic, moist to wet and contained a varying amount of sand and silt and wet fine grained sand layers. Sand material backed up into the hollow stem augers at an approximate depth of 11 m at TH13-201 where the drilling had to be discontinued. The measured (uncorrected) SPT N-value of 15 and 24 blows per 300 mm indicated that the sandy silt till was generally in a compact state. The moisture content ranged from 10% to 17% and averaged at approximately 14%. A grain size analysis test indicated that

the till consisted of approximately 68% silt, 13% silt and 19% clay. Bedrock was not encountered at a maximum depth of approximately 11.0 m below grade (El. 220.4 m) at the end of both test holes, TH13-201 and TH13-202.

3.3 NORTHWEST AND SOUTHEAST CANAL EARTHFILL EMBANKMENTS- TALBOT CANAL

3.3.1 Embankment Fill

The fill materials encountered within the canal embankments mainly consisted of clayey silt to silty clay fill. The thickness of the embankment fill material estimated from the test holes varied from 2.3 to 3.5 m. The crest elevation of the canal embankments at the test hole locations varied between 231.20 and 231.53 m (approximate bottom of fill at El. 228.9 to 227.9 m).

A 200 mm thick black topsoil layer or granular fill was encountered at the ground surface of the test holes (i.e. crest of embankments). The upper topsoil or granular layers were underlain by a clayey silt and silty clay fill, which was typically brown, moist to very moist and of intermediate plasticity. The clayey silt to silty clay fill contained trace amounts to some fine-grained sand and trace to some topsoil at occasional depths. A 200 to 300 mm thick black topsoil layer was encountered at the depths of 2.2 m and 3.2 m at TH13-302 and -307 respectively, and were likely the original topsoil at the base of the embankment during the construction. The uncorrected SPT N-values were typically 1 to 4 blows per 300 mm indicating a soft to very soft in consistency, but reached up to 14 blows/300 mm at the shallow depths at TH13-302, -305 and -307. The moisture content was tested and ranged from 14% to 34% with an average value of approximately 25%. Atterberg Limits testing indicated that the clayey silt was of intermediate plasticity (Liquid Limit averaged at 41%, Plastic Limit averaged at 18% and Plasticity Index averaged at 23%). A grain size analysis (TH13-306 S4) indicated that the clayey silt fill typically consisted of approximately 81% silt plus clay (approximately 10% clay) and 19% sand.

3.3.2 Native Clayey Silt to Silty Clay

Clayey silt to silty clay (native soil) was encountered below the embankment fill, and extended from the bottom of the embankment to a depth varying approximately from 4.9 m to 6.6 m below

grade at the end of the test holes (with the exception of TH13-306 where 0.2 m of silt till was encountered at the bottom of the hole, see Section 3.2.5 below). In general, the clayey silt to silty clay soil was typically brown and grey, moist to very moist and of intermediate plasticity. The clayey silt fill contained trace to some amounts of fine-grained sand. The uncorrected SPT N-values were typically 1 to 4 blows per 300 mm indicating a soft to very soft consistency, but reached up to 13 blows/300 mm at the local areas at TH13-306 and -307. Grain size indicated that the soil typically consisted of approximately 82% to 99% silt plus clay (clay content of approximately 33%), and 1% to 18% sand.

3.3.3 Sandy Silt Till

The deepest test hole TH13- 306 indicated that the clayey silt to silty clay layer was underlain by glacial sandy silt till at a depth of 6.5 m below ground surface (EL. 224.7 m). The sandy silt till was grey in colour, moist, low plasticity and very dense. Bedrock was not encountered at a maximum depth of approximately 6.7 m below grade (El. 224.6 m) at the bottom of the test holes.

3.3.4 Test Pitting

Shallow test pitting [approximately 2.5 m long by 1.0 m wide by 1.2 m deep] was performed using a rubber tire backhoe at four select locations along the upstream side of the canal embankments to try and identify whether the sheet piles were installed. Note that similar test pitting was completed at the canal embankment just upstream of Lock 40 where seepage and erosion was observed in the summer of 2013. At this location near Lock 40, the sheet piles were found near the upstream edge of the crest. The water level was high during test pitting (approximately 0.8 m below the crest), and the test pit depth and extent were limited to ensure there would be no seepage or erosion, particularly since the location and elevation of the sheet piles were not yet known. Sheet piles were not encountered within the investigated depth and extent of the test pits.

Embankment fill encountered during the test pitting within the investigated depth (±1.2 m) mainly consisted of clayey silt to silty clay fill. The clayey silt fill was typically brown, moist to very moist and of low to intermediate plasticity and contained some sand and trace to some

topsoils. The test pits were backfilled with the excavated embankment fill and compacted to a density similar to the surrounding soil using vibrating plate tamper equipment.

3.4 GROUNDWATER CONDITIONS

As described in Section 2.0, three (3) standpipes and two (2) VW piezometers were installed to monitor groundwater conditions during this site investigation. All the three standpipes were installed within the foundation soils of the embankments. Standpipe readings have been monitored two (2) times since installation to date, one in June 2013 and one in September 2013. The monitoring zone and the results of the groundwater level monitoring are summarized in Table 3.4-1. The standpipe readings indicated that the groundwater levels within the foundation soils were within 0.6 m between the readings (June and September 2013), although possible fluctuations are not known.

Two (2) VW piezometers were installed during this investigation as detailed below.

- One installed in TH13-101 (Talbot River Dam) within the sandy silt till with tip elevation at approximately El. 225.3 m.
- One installed in TH13-202 (Lock 39) within the embankment fill with tip elevation at approximately El. 222.9 m.

From June 6, 2013 to September 12, 2013 the vibrating wire piezometers were set up to collect data every 5 minutes in order to monitor groundwater levels within the embankment fills or foundation soils and to observe groundwater fluctuations during operation of the lock. The monitoring data from the installed VW piezometers are summarized in Figures 3.4a and 3.4b together with the upstream reservoir levels and Talbot Canal levels from PCA. The typical lock operation consists of lowering and refilling of approximately 6.0 m over an approximate 20 to 30 minute period. The review of piezometer data revealed that the groundwater levels during the lock operations have shown only approximately 25 to 50 mm groundwater fluctuation, indicating that there is a very slight to no impact on the piezometric levels within the embankment during the lock operation. As shown in Figures 3.4a and 3.4b, the piezometric levels measured in the Right Earthfill Embankment of Talbot River Dam varied between El. 227.9 m to El. 228.2 m during the monitoring period and between El. 227.8 m to El. 228.4 m at the north embankment of Lock 39.

TABLE 3.4-1
GROUNDWATER MONITORING DATA – STANDPIPE READINGS

Operating Level	**Talbot River Dam @ El. 230.44 m**	**Lock 39 @ El. 230.44 m**	**Talbot Canal @ El. 230.44 m**
Ground Elevation	**El. 231.26 m**	**El. 231.40 m**	**El. 231.60 m**
Piezometer	**Standpipe**	**Standpipe**	**Standpipe**
Test Hole Number	**TH13-101**	**TH13-202**	**TH13-304**
Tip Elevation	**El. 224.96 to El. 225.42**	**El. 220.52 to El. 220.98 m**	**El. 226.56 to El. 227.02 m**
Monitoring Zone	**Till**	**Till**	**Clayey Silt/Silty Clay**
Date	**Total Measured Pressure Head (m)**	**Total Measured Pressure Head (m)**	**Total Measured Pressure Head (m)**
15-Jun-13	227.36	227.53	229.47
12-Sep-13	227.96	227.12	-
13-Sep-13	-	-	229.34

FIGURE 3.4A
GROUNDWATER LEVEL MEASUREMENTS - NORTH EMBANKMENT OF TALBOT RIVER DAM

FIGURE 3.4B
GROUNDWATER LEVEL MEASUREMENTS - NORTH EMBANKMENT OF LOCK 39

3.5 LABORATORY TESTING RESULTS

3.5.1 Routine Index Testing

The results of routine index testing on the soils, including moisture content, grain size analysis (sieve and hydrometer testing) and Atterberg Limits testing, are summarized in Table 3.5-1 according to test hole number and sample depth, as well as on the detailed borehole logs presented in Appendix A. The results were also discussed in Sections 3.1 to 3.3.

TABLE 3.5-1
TALBOT RIVER DAM, LOCK 39 AND TALBOT CANAL: INDEX CHARACTERISTICS OF SOIL SAMPLES

Test Hole #	Sample Depth (m)	Moisture Content (%)	Plasticity (%)			Grain Size Distribution (%)						
			LL	PL	PI	Gravel	Sand				Silt (<0.075 to 0.002 mm)	Clay (<0.002 mm)
							Coarse (<4.75 to 2.0 mm)	Medium (<2.0 to 0.425 mm)	Fine (<0.425 to 0.075 mm)	Sand Total		
TH13-101	0.90	14	-	-	-	-	-	-	-	-	-	-
	2.44	-	-	-	-	0.7	0.4	3.5	60.6	64.5	28.0	6.8
	2.75	12	-	-	-	-	-	-	-	-	-	-
	4.27	-	-	-	-	0.0	0.0	0.0	50.4	50.4	44.2	5.4
	4.58	27	-	-	-	-	-	-	-	-	-	-
	5.20	29	-	-	-	-	-	-	-	-	-	-
TH13-102	0.61	-	-	-	-	0.0	0.0	0.3	47.8	48.1	44.1	7.8
	0.91	22	-	-	-	-	-	-	-	-	-	-
	3.66	-	-	-	-	0.2	0.0	0.1	6.6	6.7	55.7	37.4
	3.70	35.7	35	17	18	-	-	-	-	-	-	-
	5.90	33	-	-	-	-	-	-	-	-	-	-
TH13-103	5.49	-	-	-	-	5.4	3.0	6.9	27.1	37.0	45.5	12.1
TH13-104	-	-	-	-	-	-	-	-	-	-	-	-
TH13-104a	0.90	13	-	-	-	-	-	-	-	-	-	-
	2.13	22	-	-	-	-	-	-	-	-	-	-
	3.36	22	-	-	-	-	-	-	-	-	-	-
	4.58	22	-	-	-	-	-	-	-	-	-	-
	5.80	23	-	-	-	-	-	-	-	-	-	-
TH13-105	3.05	-	28	14	14	-	-	-	-	-	-	-
	3.80	42.8	48	20	28	-	-	-	-	-	-	-
	3.81	-	-	-	-	0.0	0.0	0.5	3.1	3.6	41.8	54.6
	6.01	-	-	-	-	5.8	5.4	11.9	28.2	45.5	33.0	15.7
	6.32	6	-	-	-	-	-	-	-	-	-	-
TH13-201	0.99	23	-	-	-	-	-	-	-	-	-	-
	3.28	27	-	-	-	-	-	-	-	-	-	-
	6.10	-	-	-	-	0.0	0.0	0.3	5.9	6.2	75.4	18.4
	6.31	28	25	17	8	-	-	-	-	-	-	-
	7.85	39	-	-	-	-	-	-	-	-	-	-
	9.38	10	-	-	-	-	-	-	-	-	-	-
TH13-202	3.29	19	23	17	6	-	-	-	-	-	-	-
	4.57	-	-	-	-	0.2	0.0	0.3	12.8	13.1	71.5	15.2
	4.80	27	-	-	-	-	-	-	-	-	-	-
	6.33	29	-	-	-	-	-	-	-	-	-	-
	7.85	24	-	-	-	-	-	-	-	-	-	-
	10.67	-	-	-	-	0.2	0.6	1.8	10.4	12.8	68.0	19.0
	10.90	17	-	-	-	-	-	-	-	-	-	-

TABLE 3.5-1 - CONTINUED
TALBOT RIVER DAM AND LOCK 39: INDEX CHARACTERISTICS OF SOIL SAMPLES

Test Hole #	Sample Depth (m)	Moisture Content (%)	Organic Content (%)	Plasticity (%)			Grain Size Distribution (%)						
				LL	PL	PI	Gravel	Sand				Silt (<0.075 to 0.002 mm)	Clay (<0.002 mm)
TH13-301	1.85	27	-	-	-	-	-	-	-	-	-	-	-
	3.35	14	-	-	-	-	-	-	-	-	-	-	-
	4.90	50.2	-	44	19	25	0.0	0.0	0.4	1.1	1.5	43.7	54.8
TH13-302	1.82	22	-	-	-	-	-	-	-	-	-	-	-
	4.88	28	-	-	-	-	-	-	-	-	-	-	-
TH13-303	1.82	27	-	-	-	-	-	-	-	-	-	-	-
	3.36	51	-	-	-	-	-	-	-	-	-	-	-
	4.90	34	-	-	-	-	-	-	-	-	-	-	-
TH13-304	1.82	34	-	-	-	-	-	-	-	-	-	-	-
	3.40	34.2	-	31	17	14	0.0	0.0	0.2	0.8	1.0	65.4	33.6
	4.88	38	-	-	-	-	-	-	-	-	-	-	-
TH13-305	1.82	28	-	-	-	-	-	-	-	-	-	-	-
	3.40	43.1	-	48	19	30	-	-	-	-	-	-	-
	4.88	31	-	-	-	-	-	-	-	-	-	-	-
TH13-306	0.90	20	-	-	-	-	-	-	-	-	-	-	-
	2.10	-	-	-	-	-	0.0	0.0	1.2	17.4	18.6	71.1	10.3
	3.37	49	-	-	-	-	-	-	-	-	-	-	-
	4.58	28	-	-	-	-	-	-	-	-	-	-	-
	5.80	43	-	-	-	-	-	-	-	-	-	-	-
TH13-307	0.90	25	-	-	-	-	-	-	-	-	-	-	-
	2.14	29	-	-	-	-	-	-	-	-	-	-	-
	3.40	29	-	-	-	-	0.0	0.0	0.1	18.1	18.2	71.9	9.9
	4.58	46	-	-	-	-	-	-	-	-	-	-	-

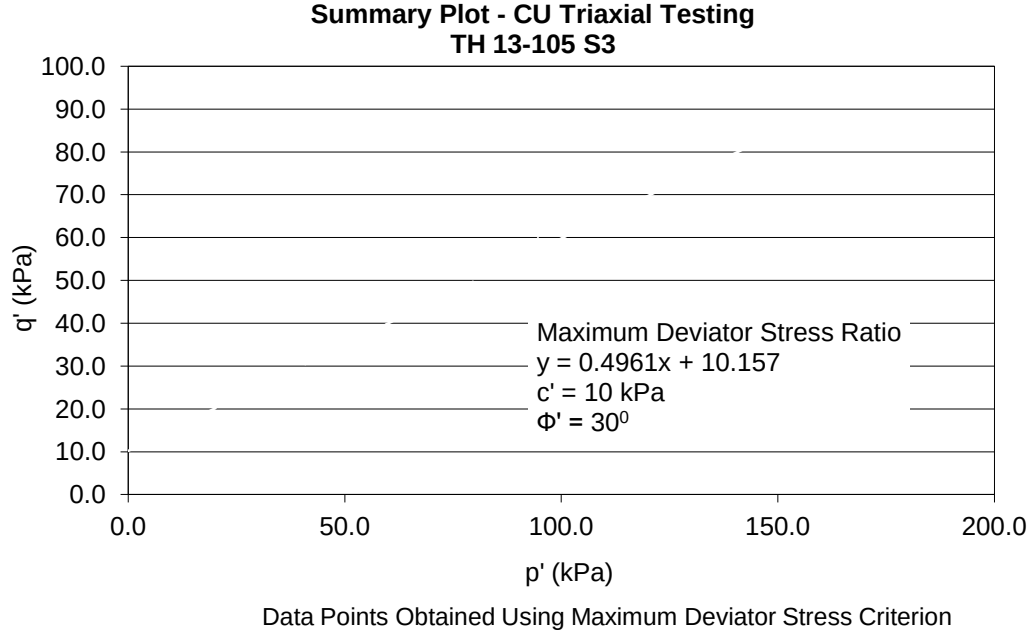
3.5.2 Consolidated Undrain Triaxial Tests

The Consolidated Undrained (CU) Triaxial tests were undertaken to estimate the effective shear strength parameters of the cohesive soil. Three (3) staged Consolidated Undrained Triaxial tests with pore pressure measurements were performed on the relatively undisturbed Shelby tube sample of clayey silt to silty clay fill from TH13-105 S3 (depth from 3.05 m to 3.81 m below grade). The test was carried out at the confining pressures of 70 kPa, 120 kPa and 170 kPa. The plot of p-q using maximum deviator stress criterion is shown in Figure 3.5-1 from the test. The estimated clayey silt to silty clay fill effective shear strength interpreted from CU Triaxial tests are listed below and representative of the intact strengths and not previously sheared or residual strengths:

Effective friction angle of clayey silt to silty clay fill Φ'= 30 degree

Effective Cohesion of clayey silt to silty clay fill c'= 10 kPa.

FIGURE 3.5-1
PLOT OF P-Q USING MAXIMUM DEVIATOR STRESS CRITERION



3.5.3 Direct Shear Tests

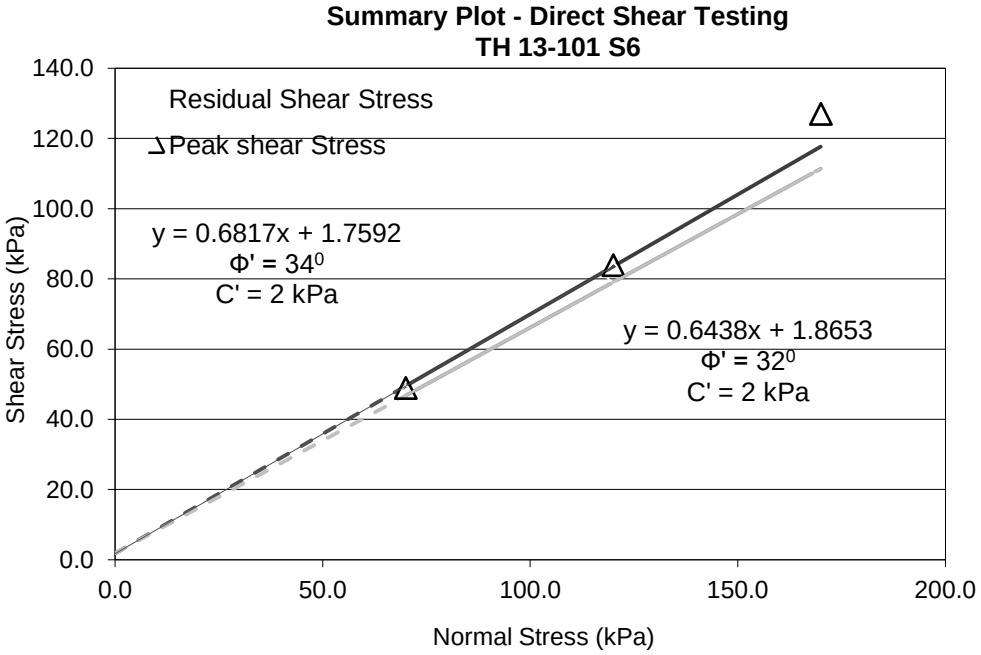
Direct shear tests were undertaken to estimate the effective shear strength parameters of relatively cohesionless soil. Three (3) Consolidated Drained Direct Shear Tests were performed (DST) on the disturbed split spoon sample of sandy silt to silty sand fill at the normal stresses of 70 kPa, 120 kPa and 170 kPa, representative of the in-situ normal stress state at the embankments. The sample was obtained from TH13-101 S6 at depths from 3.05 m to 3.66 m below grade. The direct shear test sample was prepared at the wet density of 19.5 kN/m^3 with 15% moisture content. Note that the sample was reconstituted to a density considered reasonably representative of the existing state to provide an approximation of the in-situ conditions. However, some variability can be expected and should be considered within the DSR analysis.

The plot of shear stress vs normal stress is shown in Figure 3.5-2 from the test. The estimated sandy silt to silty sand fill effective shear strength interpreted from Direct Shear tests are listed below:

Effective residual friction angle of sandy silt to silty sand Φ'= 32 degree
Effective residual cohesion of sandy silt to silty sand c'= 2 kPa.

Effective Peak friction angle of sandy silt to silty sand Φ'= 34 degree
Effective Peak cohesion of sandy silt to silty sand c'= 2 kPa.

FIGURE 3.5-2
PLOT OF SHEAR STRESS VS NORMAL STRESS



4.0 GEOTECHNICAL ENGINEERING ASSESSMENTS

4.1 ENGINEERING PARAMETERS FOR THE EMBANKMENT MATERIALS

Representative engineering properties of the embankment fills and foundation soils for Talbot River Dam, north side of Lock 39 embankment and Talbot Canal embankments are presented on Table 4.1-1. The soil strength parameters of the embankment fill and foundation soils were estimated from in situ Standard Penetration Tests (SPT) and laboratory index testing using empirical correlations and the interpretation of the advanced laboratory testing results, as well as previous experience with similar soil materials. Ranges of shear strengths have been provided to represent the variations observed from the investigations. Some cohesion was measured in the silty clay / clayey silt from the CU triaxial test (10 kPa). Cohesion within the non-plastic materials was assumed to be zero.

TABLE 4.1-1
EFFECTIVE SHEAR STRENGTH PARAMETERS FOR EARTHFILL EMBANKMENTS

Material	Representative Lower and Upper Bounds, Φ'	Consistency based on SPT	Cohesion, c' (kPa)	Estimated Unit Weight, γ_{sat} (kN/m^3)
North and South Embankments of Talbot River Dam				
Sandy Silt to Silty Sand Fill	30° to 32°	Loose to Compact	0	19.5
Clayey Silt to Silty Clay Fill	27° to 30°	Soft to Firm	0	18.0
Sandy Silt Till	38° to 40°	Dense to Very Dense	0	21.0
North of Lock 39 Embankments				
Clayey Silt to Silty Clay Fill	27° to 30°	Soft to Very Soft	0	18.0
Sandy Silt Till	38° to 40°	Compact	0	21.0
Northwest and Southeast Canal Embankments of Talbot Canal				
Clayey Silt to Silty Clay Fill	27° to 30°	Soft to Very Soft	0	18.0
Clayey Silt to Silty Clay	27° to 30°	Soft to Very Soft	0	18.0
Sandy Silt Till	38° to 40°	Very Dense	0	21.0

5.0 CLOSURE

1. This geotechnical program was undertaken to provide in-situ properties of the fill and foundation materials necessary to carry out the DSR. Any additional or more detailed investigation activities that could be potentially required for design of any remedial work have not been included in this investigation program.

2. A total of fifteen (15) test holes were drilled during the exploratory program to investigate the embankment fill materials and foundation soils of the earthfill embankments. Three (3) monitoring wells (standpipe piezometers) and two (2) vibrating wire piezometers were installed in the test holes to monitor the groundwater level within the earthfill embankments or their foundation soils.

3. The fill materials encountered within all the embankments at the site mainly consisted of sandy silt and/or clayey silt to silty clay. The sandy silt fill was loose to compact, had low to no plasticity, was moist to wet and contained varying amounts of topsoil and clay. The clayey silt to silty clay fill was firm to very soft, low to intermediate plasticity and contained trace to some amounts of fine-grained sand and contained varying amounts of topsoil.

4. The native soil encountered at the site consisted of clayey silt to silty clay and/or sandy silt till. The clayey silt to silty clay was soft to very soft, intermediate plasticity and contained trace to some amounts of fine-grained sand. The sandy silt till was compact to very dense.

5. The piezometric data revealed that there was only a very slight to no impact on groundwater level within the left embankment during the lock operation.

6. The soils strength parameters of the embankment fill were estimated based on the field testing results, the laboratory testing results and previous experience with similar materials.

6.0 STATEMENT OF LIMITATIONS AND CONDITIONS

6.1 THIRD PARTY USE OF REPORT

This report has been prepared for the Client to whom this report has been addressed and any use a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. KGS Group accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions undertaken based on this report.

6.2 GEOTECHNICAL INVESTIGATION STATEMENT OF LIMITATIONS

The geotechnical investigation findings and recommendations of this report were prepared in accordance with generally accepted professional engineering principles and practice. The findings and recommendations are based on the results of field and laboratory investigations, combined with an interpolation of soil and groundwater conditions found at and within the depth of the test holes drilled by KGS at this site

APPENDICES

APPENDIX A1

TEST HOLE LOGS

KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO. **TH 13-101**

CLIENT PWGSC

PROJECT TALBOT RIVER DAM AND LOCK 39

SITE Talbot River Dam- Right Wing Earthfill Embankment

LOCATION Crest of Dam, 10 m N of North Abutment of concrete dam (D/S side)

DRILLING METHOD Casing and Washboring, Portable Rig (Manual Hammer)

JOB NO. **12-0006-028**
GROUND ELEV. **231.26 m**
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED **07/06/2013**
UTM (m) N
E

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY %	SPT (N) blows/0.15 m / DYNAMIC CONE (N) blows/ft
231	0	0	**EMBANKMENT FILL (0.0 - 5.49 m)** **Sandy Silt to Silty Sand Fill (0.0 - 3.66 m)** - Brown, moist, compact, low plasticity to non plastic, trace clay.				S1	42	3, 6, 6, 7
	1						S2	8	12, 15, 6
230		5	- Varying sand and silt content. - Loose below 1.22 m.				S3	100	7, 2, 2, 1, 1, 2, 1
	2		- Compact below 1.83 m.				S4	71	4, 6, 5, 2, 2, 1, 1
229			- Grading to more sandy, loose, and very moist below 2.44 m. - Grain Size Distribution: Gravel (0.7%), Sand (64.5%), Silt (28.0%), and Clay (6.8%) at 2.44 m.				S5	75	1, 1, 1, 1, 2, 2, 14
	3	10					S6	67	4, 11, 4, 3, 2, 2, 2
228			**Silty Sand Fill (3.66 - 5.49 m)** - Brown, wet, compact to loose, low plasticity to non plastic, with fine-grained sand. - 0.15 m thick black topsoil layer at 3.81 m.				S7	62	3, 1, 2, 2, 1
	4		- Grading to more silty, brown/grey, and wet below 4.27 m. - Grain Size Distribution: Gravel (0.0%), Sand (50.4%), Silt (44.2%), and Clay (5.4%) at 4.27 m.				S8	75	2, 2, 3, 1, 2
227		15					S9	62	1, 2, 1
	5		- Wet, loose below 4.88 m.						
226									
225.8			**SANDY SILT TILL** - Grey, very moist, very dense, low plasticity to non plastic.	VW	5.9		S10	17	50, 50/(50 mm penetration)
	6	20			5.9				
225.0			**LIMESTONE BEDROCK** Light grey, fine grained, competant.						
225									
224.7			**END OF HOLE ON BEDROCK AT 6.55 m.**		6.6				
	7		Notes: 1. Test hole completed on June 8, 2013. 2. Installed a flushmount piezometer to a depth of 6.25 m. 3. Installed a vibrating wire (SN 12-9209) to a depth of 5.95 m. 4. Test hole backfilled with grout.						
224		25							
	8								
223									
	9	30							
222									

Cu POCKET PEN (kPa) ★
Cu TORVANE (kPa) ◆
20 40 60 80
PL MC LL
%
20 40 60 80

SAMPLE TYPE ☒ Split Spoon

CONTRACTOR	INSPECTOR	APPROVED	DATE
OGS Inc.	C.H. / S.G.	MJ	9/4/15

GEOTECHNICAL-SOIL LOG P:\PROJECTS\2012\12-0006-028\DESIGN\GEO\LOGS\TALBOT GINT _SN\TALBOT RIVER DAM LOGS.GPJ

KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO. **TH 13-102**

CLIENT PWGSC

PROJECT TALBOT RIVER DAM AND LOCK 39

SITE Talbot River Dam- Right Wing Earthfill Embankment

LOCATION Crest of Dam, 21 m N of North Abutment of concrete dam (U/S side)

DRILLING METHOD Continuous Split Spoon, Portable Rig (Manual Hammer)

JOB NO. **12-0006-028**
GROUND ELEV. **231.28 m**
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED **08/06/2013**
UTM (m) N
E

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.15 m ▲ / DYNAMIC CONE (N) blows/ft △
231				**EMBANKMENT FILL (0.0 - 5.79 m)**				
				Sandy Silt Fill (0.0 - 1.83 m) - Brown, moist, soft, low plasticity to non plastic, trace clay.		S1	62	2 1 1 3
	1			- Very moist to wet below 0.61 m.				
				- Grain Size Distribution: Gravel (0.0%), Sand (48.1%), Silt (44.1%), and Clay (7.8%) at 0.61 m.		S2	71	1 2 3 3
230		5		- Wet, very soft below 1.22 m.		S3	58	0 1 2 2
	2			**Clayey Silt to Silty Clay Fill (1.83 - 5.79 m)** - Grey/brown, very moist, very soft, intermediate plasticity.		S4	75	0 0 0 1
229				- Topsoil mixed between 1.83 and 2.44 m.		S5	54	0 1 1 1
	3	10		- Very soft below 3.1 m.		S6	50	0 0 0 2
228				- Grain Size Distribution: Gravel (0.2%), Sand (6.7%), Silt (55.7%), and Clay (37.4%) at 3.66 m.		S7	54	0 1 1 2
	4					S8	22	1 1 1 1
227		15		- Sampler sank to 5.18 m on last blow from 4.87 m.				
	5			- Very soft, very moist to wet below 5.4 m.		S9	58	0 0 0 1
226								
225.5								
225.3	6	20		**SANDY SILT TILL** - Grey, very moist, very dense, low plasticity to non plastic.		S10	56	5 ▲50 /(75 mm penetration)
225				**END OF HOLE IN TILL AT 6.02 m.**				
				Notes: 1. Water level at 1.22 m below grade upon completion of drilling. 2. Test hole completed on June 8, 2013. 3. Test hole backfilled with bentonite chips.				
	7							
224		25						
	8							
223								
	9	30						
222								

Cu POCKET PEN (kPa) ★
Cu TORVANE (kPa) ◆
20 40 60 80
PL MC LL
%
20 40 60 80

SAMPLE TYPE ☒ Split Spoon

CONTRACTOR	INSPECTOR	APPROVED	DATE
OGS Inc.	C.H. / S.G.	MJ	9/4/15

GEOTECHNICAL-SOIL LOG P:\PROJECTS\2012\12-0006-028\DESIGN\GEO\LOGS\TALBOT GINT _SN\TALBOT RIVER DAM LOGS.GPJ

KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO. **TH 13-103**

CLIENT PWGSC

PROJECT TALBOT RIVER DAM AND LOCK 39

SITE Talbot River Dam- Right Wing Earthfill Embankment

LOCATION Crest of Dam, 59 m N of North Abutment of concrete dam (D/S side)

DRILLING METHOD Continuous Split Spoon, Portable Rig (Manual Hammer)

JOB NO. **12-0006-028**
GROUND ELEV. **231.33 m**
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED **08/06/2013**
UTM (m) N
E

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.15 m ▲ / DYNAMIC CONE (N) blows/ft △
231				**EMBANKMENT FILL (0.0 - 5.79 m)**				
				Sandy Silt Fill (0.0 - 1.83 m) - Brown, moist, compact, low plasticity to non plastic, trace clay.		S1	42	2, 3, 4, 8
	1					S2	50	7, 6, 3
230		5		- Loose below 1.22 m.		S3	42	4, 2, 1, 2
	2			**Sandy Silt to Silty Sand Fill (1.83 - 5.79 m)** - Brown, very moist to wet, loose, low plasticity to non plastic.		S4	50	2, 1, 1, 1, 0, 1
229				- Varying sand and silt content.				
				- Loose and very moist to wet below 2.8 m.		S5	58	8 (Rock encountered)
	3	10				S6		1, 1, 2, 1, 2, 1
228				- Brown/grey, wet, and loose below 3.66 m.		S7		1, 1, 2
	4					S8		2, 2, 3, 2, 1, 1
227		15		- Varying sand and silt content.				
	5					S9		1, 4, 7, 3, 2
226				- Compact below 5.18 m.				
225.5				- Grain Size Distribution: Gravel (5.4%), Sand (37.0%), Silt (45.5%), and Clay (12.1%) at 5.49 m.		S10		
225.3	6	20		**SANDY SILT TILL** - Grey, moist, very dense, low plasticity to non plastic.				18, 50/(75 mm penetration)
225				**END OF HOLE IN TILL AT 6.02 m.**				
				Notes: 1. Test hole completed on June 8, 2013. 2. Water level at 4.2 m below grade upon completion of drilling. 3. Test hole backfilled with bentonite chips.				
224	7	25						
	8							
223								
	9	30						
222								

Cu POCKET PEN (kPa) ★
Cu TORVANE (kPa) ◆
20 40 60 80
PL MC LL
%
20 40 60 80

SAMPLE TYPE ☒ Split Spoon

CONTRACTOR	INSPECTOR	APPROVED	DATE
OGS Inc.	C.H. / S.G.	MJ	9/4/15

GEOTECHNICAL-SOIL LOG P:\PROJECTS\2012\12-0006-028\DESIGN\GEO\LOGS\TALBOT GINT _SN\TALBOT RIVER DAM LOGS.GPJ

KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO. **TH 13-104**

CLIENT PWGSC

PROJECT TALBOT RIVER DAM AND LOCK 39

SITE Talbot River Dam- Right Wing Earthfill Embankment

LOCATION Crest of Dam, 102 m N of North Abutment of concrete dam (U/S side)

DRILLING METHOD Continuous Split Spoon, Portable Rig (Manual Hammer)

JOB NO. **12-0006-028**
GROUND ELEV. **231.29 m**
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED **08/06/2013**
UTM (m) N
E

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.15 m ▲ / DYNAMIC CONE (N) blows/ft △	Cu POCKET PEN (kPa) ★ / Cu TORVANE (kPa) ◆
231	0	0		**EMBANKMENT FILL (0.0 - 2.13 m)** **Sandy Silt Fill (0.0 - 2.13 m)** - Brown, moist, loose, low plasticity to non plastic trace clay.		S1	42	1, 6, 1, 2	
	1					S2	62	3, 1, 1	
230		5		- Very moist to wet below 1.22 m.		S3	75	4, 2, 1, 1, 1, 1	
229.2	2					S4	67	50 (Concrete Corewall)	
229				**END OF HOLE ON CONCRETE CORE WALL AT 2.13 m.**					
	3	10		Notes: 1. Concrete corewall encountered at 1.22 m. Test hole moved to downstream side of embankment (TH 13-104a). 2. No free water in the test hole upon completion of drilling. 3. Test hole backfilled with bentonite chips.					
228									
	4	15							
227									
	5								
226									
	6	20							
225									
	7								
224		25							
	8								
223									
	9	30							
222									

SAMPLE TYPE ☒ Split Spoon

CONTRACTOR	INSPECTOR	APPROVED	DATE
OGS Inc.	C.H. / S.G.	MJ	9/4/15

GEOTECHNICAL-SOIL LOG P:\PROJECTS\2012\12-0006-028\DESIGN\GEO\LOGS\TALBOT GINT _SN\TALBOT RIVER DAM LOGS.GPJ

DAM SAFETY GENERAL INSPECTION

Site Name: Talbot River Dam and Lock 39 **River System:** Talbot River
Dam Component: **HWL:** m **TWL:**
Description: The dam was built in 1908 and comprises two 7.6m stoplog weirs, two spillways (6.1 m and 5.8 m wide) and 6.7 m long gravity bulkheads at each end. The dam has a reservoir height of 6.3 m. The lock comprises concrete approach and downstream walls flanking wooden lock control gates. Between the gates, the lock has vertical concrete walls.
Purpose: A regulating dam and a canal control lock.

Length: 50m **Height:** 8.0m **Width:** **ICC Rating:**

Recommended Actions:

Item	Summary of Inspection Observations and Identified Deficiencies:	Recommended Actions:
12-01	Some deck planks are rotten	Replace in 2013
12-02	Handrail baseplate bolts missing	Investigate to determine if railing support is adequate. Add bolts if not
12-03	Gravity sections have minor seepage at lower joints, some joints appear open and there is localized minor spalling and erosion along joints.	Monitor
12-04	Map cracking in all concrete. Some open cracks on downstream piers and erosion above sluiceway	Monitor
12-05	Falling risk towards water at bottom of steps, south gravity	Install rail within next year
12-06	Steep slopes adjacent to dam make winter access unsafe	Install proper steps on the embankment in 2013
12-07	Undermined and fractured concrete on downstream retaining walls and drain channels	Repair in the next 1 to 3 years
12-08	General freeze/thaw deterioration along lock, upstream, internal and downstream walls	Repair in the next 2 to 4 years
12-09	Falling hazard between lock gates in winter closed position and slipping hazard under gate handrails	Add rails/chains/kick plates to provide safe condition
Yr-#		

Date of Inspection: November 27, 2012 **Date of Last Inspection: Not known**

Weather: Cold, intermittent snow

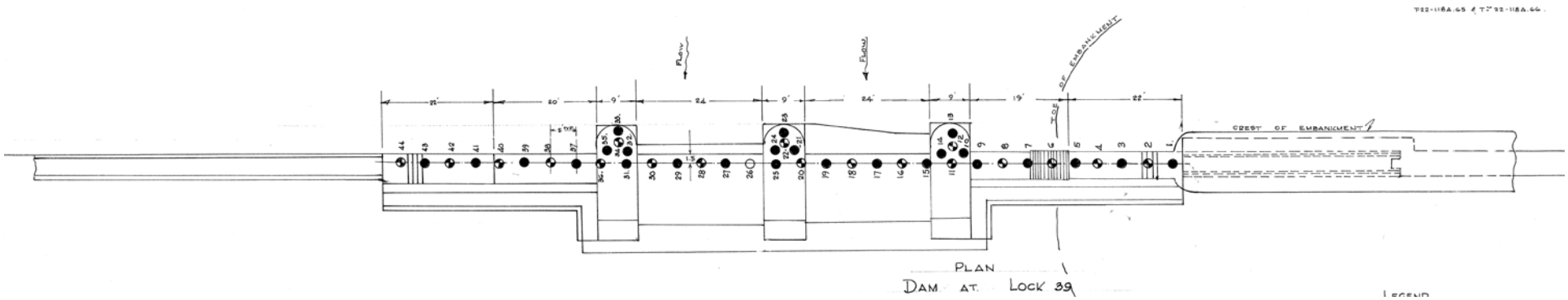
DAM SAFETY GENERAL INSPECTION

Persons Present During Inspection: Paul Read, Holly Hampton, Yongbo Fu

This is to certify that the above dam has been inspected and the following are the results of this inspection.

Name and Signature of Inspection Leader

SITE or STRUCTURE



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KGS GROUP

DAM SAFETY GENERAL INSPECTION

STOPLOG SLUICE

DECK [Photo 1 to 9)

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	The deck was partially snow covered during inspection. This is a 2" thick wooden plank deck supported on steel I beams. The steel beams are painted and in good condition. A number of deteriorated planks were observed. It is understood that PCA plans to replace these in the spring 2013. There were a number up upstands including stoplog and rail supports, rail stop ends etc that present a tripping hazard. All of these have been painted yellow to make them more visible
Condition of Joints	N/A
Movement	None evident
Hand Rails	Upstream comprises galvanized tube handrail with chain link infill. Downstream is comprised of painted steel angle posts supporting 3 cable strands and a wooden beam that performs the function of a kick-plate and post support. A number of post base-plates have only 1 or 2 bolts which may make them unstable . Cables are corroded but still functional. Handrails are 40" high which is lower than the safety standard height.
Log Lifting Equipment	Two cast-iron winches. In good condition. Reported in detail in mechanical DSGI.
Chainage Markers	N/A

PIERS [Photo 10 to 17]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	Surfaces are in generally good condition but some areas of erosion noted particularly downstream of stoplogs. Zebra mussels on the upstream noses. Downstream of the stoplogs, the piers have map cracking. This suggests possible alkali-aggregate reaction in the concrete but no major distress is evident. Generally, the cracks are tight but some cracks towards the downstream face are open (approx up to 5mm wide). The upper parts of the piers appear to be new concrete caps. White deposits (calcite or silica gel) were evident below some joints and along some cracks.
Condition of Joints	Good (lift joints). Some very minor erosion along joints
Movement	None evident
Waterline Deterioration	Minor upstream. Erosion along internal downstream faces just above rollway.

DAM SAFETY GENERAL INSPECTION

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Beam Seats	Good condition
Stop Log/Gate Gains & Covers	Stoplogs appear to be in good condition. Gains are also in good condition where visible to inspect. Steel lined gains look to be in good condition where visible.

APRON **[Photo 18]**

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	Not visible
Condition of Joints	n/a
Movement	n/a
Undercutting	n/a

NORTH GRAVITY BULKHEAD SECTION

CREST / DECK **[Photo 19 to 21]**

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	The top of the deck of the gravity section was not visible due to snow conditions. There are two parts to the gravity section. The approach from the dam is mass concrete and there is then an overflow section abutting the sluiceway pier. On the solid portion, the deck is top of concrete. On the overflow span, there is a timber plank deck supported on painted steel I beams. The beams are in good condition. Cracks are evident in the access steps. These cracks run parallel to the line of the dam.
Condition of Joints	N/A
Movement	None observed

DAM SAFETY GENERAL INSPECTION

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Hand Rails	Steel tube post and rail upstream. Two rails. Downstream is 3-strands of cable mounted on angle iron posts. All in good, painted condition. Kick plate on downstream side of deck.
Construction Joint Drain Covers	N/A
Chainage Markers	N/A

UPSTREAM FACE [Photo 22 to 23]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	Fair to good. Map cracking with white deposits is evident but little associated concrete deterioration. Cracks near the top of the section are up to 4 mm wide. There appears to be a new concrete cap added to the top of the section.
Condition of Joints	Horizontal construction joints are open and weathered with some freeze/thaw deterioration evident. There are a number of spalled areas along the joints with one large spall evident near the north end of the overflow section.
Movement	None evident
Waterline Deterioration	No

DOWNSTREAM FACE [Photo 24 to 25]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	Fair to good. Some repairs evident that remain in good condition. Horizontal cracking with white deposits along the cracks.
Condition of Joints	Generally good, some areas of spalled concrete along the horizontal lift joints.
Movement	None evident
Leakage	Lower 600 mm is wet with some water in the channel below. This seems to be coming through the lowest lift joint.
Vegetation	Some evident in spalled areas.

DAM SAFETY GENERAL INSPECTION

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE

SOUTH GRAVITY BULKHEAD SECTION

CREST / DECK [Photo 26 to 27]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	Not visible from above due to snow. Deck construction is the same as the North gravity Section. Where visible it appears to be in good condition. Cracks are evident in the access steps. These cracks run parallel to the line of the dam. The deck over the overflow section is wooden planks on painted steel beams. The condition of the planks could not be ascertained but these are programmed to be replaced in summer 2012. The steel beams are in good condition as are the beam seats.
Condition of Joints	N/A
Movement	None evident
Hand Rails	Steel tube post and rail upstream. Two rails. Downstream in 3-strands of cable mounted on angle iron posts. All in good, painted condition. Kickplate on downstream side of deck. At the bottom of the steps at the South end, the railing is missing on the upstream side. This presents a possible falling hazard towards the water.
Construction Joint Drain Covers	N/A
Chainage Markers	N/A

UPSTREAM FACE [Photo 28 to 30]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	Generally good to fair condition but map cracking evident throughout with minimal associated deterioration. Cracks near the top of the section are open up to 5m wide.
Condition of Joints	Lift joints only. Generally tight with minor weathering but there is one horizontal joint at overflow sill level that is open and width varies up to approximately 25mm wide.
Movement	Generally no but there is a small (5mm) offset at the open joint.
Waterline Deterioration	None evident

DAM SAFETY GENERAL INSPECTION

DOWNSTREAM FACE [Photo 31 to 33]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	Concrete remains visually sound although there is map cracking with associated white deposits
Condition of Joints	Good (lift joints only)
Movement	None evident
Leakage	There is evidence of seepage through a horizontal joint near approximately 0.5m below the overflow sill (build-up of extensive white deposits below the wall) Most lift joints have minor build-up of white deposits along the joint.
Vegetation	Minimal

TAILRACE CHANNEL [Photo 34 to 36]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Sidewall/Slope Condition	The banks downstream have concrete retaining walls. These are generally in good condition but there is erosion and spalling in some locations, particularly in the vicinity of a corrugated drain pipe on the south side and around the location of the gravity wall drainage channel outlets.
Drainage Channels	The channels are concrete lined. The both channels are extensively spalled and undermined in the vicinity of the outlet into the downstream channel
Movement	None evident
Leakage	N/A
Vegetation	Minimal

OTHER ASPECTS [Photo 37]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Structure Access	The dam structure is adjacent to a main road and is easily accessible. It was noted that to get onto the deck it is necessary to climb the embankment on either side of the dam. In winter conditions this can be hazardous due to the steepness of the embankment and the absence of proper steps in the embankment. This is a safety concern and should be addressed.

DAM SAFETY GENERAL INSPECTION

NORTH LOCK WALL

TOP OF WALL [Photo 38 to 39]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	Not visible due to snow
Condition of Joints	Not visible
Movement	None evident
Hand Rails	There are no handrails along the canal walls

LOCK FACE [Photo38 to 42]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	Upstream of the lock, The surface is in fair to good condition but there are a number of locations where the concrete has spalled (8 counted), leaving holes in the approach wall. Size of holes typically 300 – 600 mm wide and 300 – 600 mm long and up to 600 mm deep. Inside the lock, the walls are generally in fair to good condition. There are a number of areas, however where the concrete is deteriorated. This tends to be at the base of the wall and looks like freeze/thaw deterioration. This is likely due to moisture transfer from the ground behind the wall saturating the wall through the winter period with the lock drained. Freeze/thaw cycles then work on the concrete. Near the gates, there is waterline erosion of the concrete which could be caused by boats in the lock. There is a failed concrete repair at the downstream end by the gates associated with the gate operating lever. Downstream of the lock, the concrete is sound with some waterline spalling of concrete. The wall up to normal water level, however is rough and this appears to be due to pop-out of aggregates near the surface of the concrete. There is erosion along the length of the wall at the higher water mark level. This appears to be up to 75mm deep and 150 mm wide.
Condition of Joints	Vertical joints tend to be in good condition. With some associated minor spalling. A horizontal joint inside the lock and approximately 1.6m above the lock floor is deeply eroded with associated spalling.
Movement	None evident
Waterline Deterioration	None evident at summer water level but deterioration and minor surface spalling evident near the base of the wall

DAM SAFETY GENERAL INSPECTION

DOWNSTREAM BULKHEAD [Photo 43 to 44]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	The bulkhead is generally sound although there is some concrete spalling just below the cap at the top of the wall. Elsewhere, map cracking of the concrete which is mostly sub-horizontal in orientation is present with white deposits along the cracks, suggesting possible AAR.
Condition of Joints	There are extensive white deposits along horizontal construction joints on the south abutment and also, but to a lesser extent on the north abutment.
Movement	None evident
Leakage	Nothing major
Stairs	The stair concrete is generally in good condition. With some cracking parallel to the line of the canal evident in the risers.
Handrails	Galvanized steel handrail appears to be relatively new and in good condition.

LOCK GATES

GATES [Photo 45 to 47]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Wooden Gate Condition	The wooden beams making up the lock gates are in good condition with no deterioration evident.
Handrails	The handrails on the gates are yellow painted steel tubing with 2 horizontal rails on the outer side (upstream and downstream) but only one rail on the inner (lock side.. It was noticed that the gap between handrails on each gate when closed is wide enough to fall through. At one end there was a chain strung across the gap at the level of the lower rail but otherwise the gap was open. All the above presents a falling hazard and bars or chains should be installed to make the gates safe to cross when closed. Consideration should be given to providing additional rails and kick plates to prevent slipping under the handrail.
Controls	See mechanical DSGI

DAM SAFETY GENERAL INSPECTION

SOUTH LOCK WALL

TOP OF WALL [Photo 38 to 39]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	The top of the wall was not visible due to snow conditions
Condition of Joints	N/A
Movement	None evident
Hand Rails	None Present

LOCK FACE [Photo 48 to 51]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	Upstream of the lock, there is erosion at the high water line that is approximately 50 mm deep and 300 mm wide. Just below current water level there is an open horizontal joint with associated areas of freeze/thaw spalling. There are two large areas of spalled concrete where the deterioration appears to extend deep into the concrete. Apart from these issues the wall is in good condition Inside the lock, the walls are generally in good condition. However, at the downstream end there is a failing concrete repair in the vicinity of the gate operating beam and spalled areas of concrete are present. There are also areas of spalled concrete along the foot of the lock wall, particularly at the downstream end. There is one large spalled area about mid-length of the wall, near the foot of the wall Downstream of the lock, the concrete wall is in good condition.
Condition of Joints	Vertical joints tend to be in good condition.
Movement	None evident
Waterline Deterioration	There is some erosion along the high waterline near to the lock gates. This is likely due to boat impact over time.

DOWNSTREAM BULKHEAD [Photo 52]

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
Surface Condition	The South side bulkhead and stairs are in similar condition to the north side with some small

DAM SAFETY GENERAL INSPECTION

CONDITION	OBSERVATION: SKETCH, MEASURE, PHOTOGRAPH, LOCATE
	areas of spalled concrete just below the cap and map cracking with associated white deposits evident. There could be AAR present in the concrete
Condition of Joints	There are extensive white deposits along horizontal construction joints on the north abutment but the joints are tight with just a few areas of spalled concrete.
Movement	None evident
Leakage	None evident
Stairs	The stair concrete is generally in good condition. With some cracking parallel to the line of the canal evident in the risers.
Handrails	Galvanized steel handrail appears to be relatively new and in good condition

DAM SAFETY GENERAL INSPECTION

PHOTOS/SKETCHES/FIGURES

DAM SAFETY GENERAL INSPECTION

Photo 1 – View along Sluice Deck Looking North

Photo 2 – Steel Deck Support Beams on Sluiceway

DAM SAFETY GENERAL INSPECTION

Photo 3 – Deck Looking South

Photo 4 – Rotten Plank on Sluice Deck

Photo 5 – Tripping Hazards

Photo 6 – Lifting Beam on Wooden Support Beam (Tripping hazard) and Upstream Handrail

DAM SAFETY GENERAL INSPECTION

Photo 7 – Downstream Kick-Plate and Post support

Photo 8 – Missing Bolts in Post Baseplates

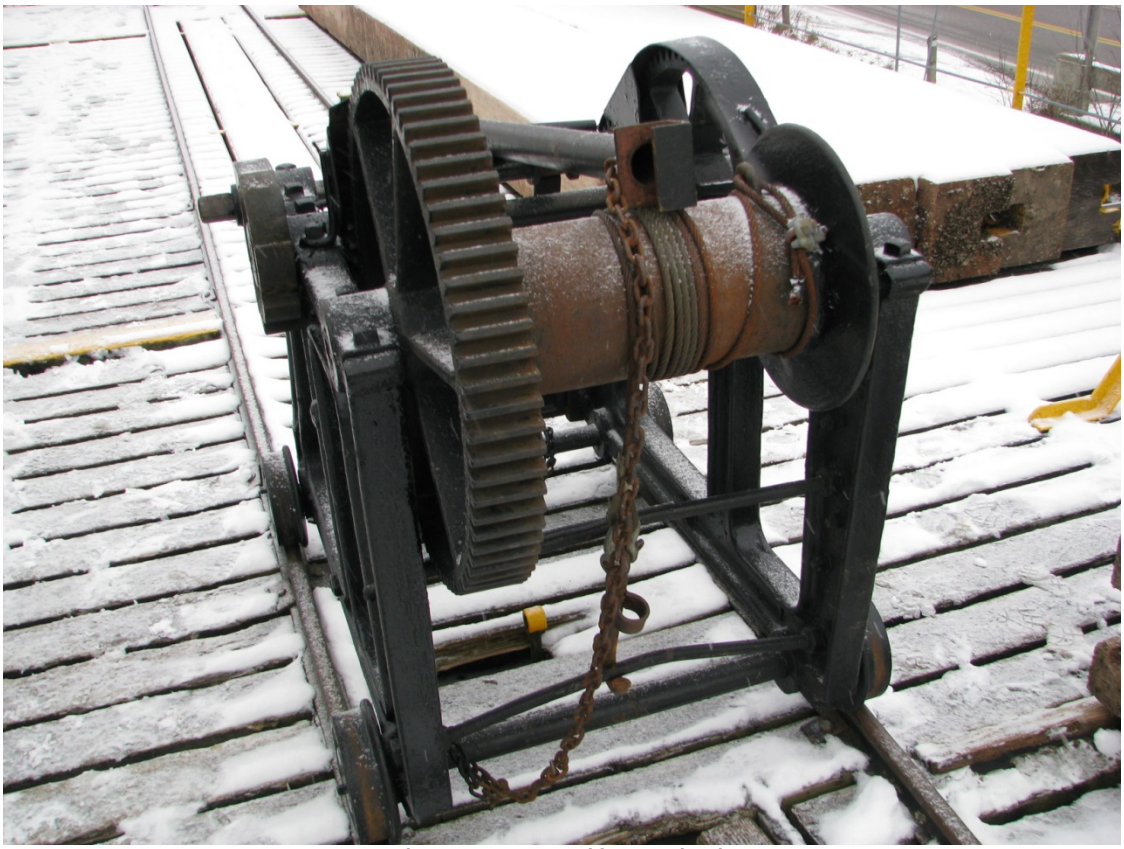

Photo 9 – Log Lifter Winch

Photo 10 – Upstream View of Piers

DAM SAFETY GENERAL INSPECTION

Photo 11 - Downstream View of Sluiceway

Photo 12 – South Pier

DAM SAFETY GENERAL INSPECTION

Photo 13 – Centre Pier

Photo 14 – North Pier

Photo 15 – South Pier Map Cracking, White Deposits and Open Cracks

Photo 16 – Underside of Deck and Beam Seats

DAM SAFETY GENERAL INSPECTION

Photo 17 – Minor Erosion on Pier Face (Typical)

Photo 18 – Flowing Water Covering Apron

DAM SAFETY GENERAL INSPECTION

Photo 19 – North End of Dam

Photo 20 – North Gravity - Underside of Bulkhead Deck

DAM SAFETY GENERAL INSPECTION

Photo 21 – North Gravity Deck Showing Handrails

Photo 22 – North Gravity - Upstream Face of Dam

DAM SAFETY GENERAL INSPECTION

Photo 23 – North Gravity - Spalled Concrete Section – rubble and Vegetation Behind

Photo 24 – North Gravity -Minor Seepage at Base of Gravity Section

Photo 25 – North Gravity, Vegetation and Minor Spalling along Lift Joints

Photo 26 – South Gravity Section Deck

Photo 27 – South Gravity - Cracks in Steps

Photo 28 – South Gravity – Upstream Face

DAM SAFETY GENERAL INSPECTION

Photo 29 – South End of South Gravity – Note Missing Rail at Bottom of Steps

Photo 30 – South Bulkhead – Open Joint

DAM SAFETY GENERAL INSPECTION

Photo 31 – South Bulkhead

Photo 32 – South Bulkhead – Overflow Section and Beam Seating

DAM SAFETY GENERAL INSPECTION

Photo 33 – South Bulkhead
White Deposits Indicating Minor Seepage at Lift Joint and Map Cracking

Photo 34 – South Downstream Channel

Photo 35 – North Downstream Channel and Drainage Channel

Photo 36 – Drainage Collector Channel

DAM SAFETY GENERAL INSPECTION

Photo 37 – Steep, Slippery Embankment on North End

Photo 38 – Lock 39 Looking Upstream

DAM SAFETY GENERAL INSPECTION

Photo 39 – Lock 39 Looking Downstream

Photo 40 – Lock 39 - Upstream North Wall

Photo 41 – Freeze/Thaw Deterioration at Base of North Wall And Minor Spalling at Vertical Joint

Photo 42 – Failing Concrete Repair at Downstream End of Lock, Erosion along Lower Construction Joint

Photo 43 – Downstream Gates and Bulkheads

Photo 44 – North Stairway

DAM SAFETY GENERAL INSPECTION

Photo 45 – Top of Lock Gates – Gap Between handrails

Photo 46 – Downstream Lock Gates

Photo 47 – Upstream Gates

Photo 48 – Upstream Approach Wall, South Side

Photo 49 – South Lock Wall Looking Upstream

Photo 50 – South Lock Wall Downstream End

Photo 51 – South Wall Downstream of Lock

Photo 52 – South Abutment, Staircase and Wall

GEOTECHNICAL DAM SAFETY GENERAL INSPECTION SHEET

TALBOT RIVER DAM

DAM SAFETY GENERAL INSPECTION

HEADPOND

North Shoreline and Slope	[Photos G1and G2]
CONDITION	**OBSERVATION**
Slope Condition	Good- No signs of any active or historic slope failures. Erosion protected by riprap along shoreline and vegetation above. Absence of riprap observed in some areas. Some minor erosion observed above riprap along the shoreline upstream from the dam, but it was not a threat to the dam performance. Overall the slope was in good condition with no stability issues observed.
Movement	None observed.
Seepage	None observed.
Vegetation	Generally occasional trees, grass, weeds and some brush vegetation on the slope and along the shoreline above the riprap between the shoreline and Canal Road which follows the north shoreline more than 600 m upstream from the dam.

South Shoreline and Slope	[Photos G3]
CONDITION	**OBSERVATION**
Slope Condition	Good- No signs of any active or historic slope failures. Erosion protected by riprap along shoreline and vegetation above. Overall the slope was in good condition with no stability issues observed.
Movement	None observed.
Seepage	None observed.
Vegetation	Generally natural forest, trees, grass, weeds and some brush vegetation at the slope and along the shoreline.

TAILRACE

North Shoreline and Slope	[Photos G4 and G5]
CONDITION	**OBSERVATION**
Slope Condition	Good- No signs of any active or historic slope failures. Erosion protected by riprap along shoreline and vegetation above beyond the concrete walls and bridge downstream from the spillway. Overall the slope was in good condition with no stability issues observed.
Movement	None observed.
Seepage	None observed.
Vegetation	Generally low brush and grass with some trees.

South Shoreline and Slope	[Photos G6]
CONDITION	**OBSERVATION**
Slope Condition	Good- No signs of any active or historic slope failures. Erosion protected by riprap with concrete slab above, along shoreline near dam structure. Minor undermining observed below the slab, but it is not a threat to the dam performance (Photo G6). Overall the slope was in good condition with no stability issues observed.
Movement	None observed.
Seepage	None observed.
Vegetation	Generally low brush and grass with some trees.

DAM SAFETY GENERAL INSPECTION

ABUTMENTS – At Concrete Structure of Talbot River Dam

Right (North) Abutment (Adjacent to north side of Spillway)	[Photos G7 and G8]
CONDITION	**OBSERVATION**
Material	Embankment fill, material type not known. Spillway abutment tied into the concrete core wall of the earthfill embankment.
Surface Condition	Fair to good. Crest-grass and vegetation cover. Upstream- Riprap, primarily finer size riprap visible at surface (75 to 250 mm with gravel). No evidence of significant erosion within the riprap but some erosion observed above the riprap. Downstream-grass, vegetation cover. Stacked stone wall along the face of the downstream slope at the abutment. Note that the stone wall appeared intact, although the shape of the stones indicated possible movement / settlement in the past, particularly in the lower half of the wall near the concrete structure (see Photo G8). The movement did not appear recent.
Contact	Not visible. No evidence of leakage/ seepage on downstream side on abutment.
Horizontal Alignment	Not applicable.
Movement	None observed except some erosion above riprap at the upstream side and possible previous movement of the stone wall on the downstream slope.
Settlement	None observed except possibly the stone wall on the downstream slope.
Depression/Sinkhole	None observed.
Ruts and/or Puddles	None observed.
Surface Protection	Vegetation cover and riprap – appeared intact.
Erosion/ Weathering	Some erosion observed above the riprap on the upstream side.
Vegetation	Generally grass and vegetation. Some tree growth located on the upstream slope, and the downstream slope within the stacked stone wall. Vegetation assessment for recommended control measures (as necessary) under review.

Left (South) Abutment (Adjacent to south side of Spillway)	[Photos G9 and G10]
CONDITION	**OBSERVATION**
Material	Embankment fill, material type not known. Spillway abutment tied into the concrete core wall of the earthfill embankment.
Surface Condition	Fair to good. Crest-grass and vegetation cover. Upstream- Riprap, some erosion observed above the riprap. Downstream-grass, vegetation cover. Stacked stone wall lined up with the face of the downstream at the abutment.
Contact	Not visible. No evidence of leakage/ seepage on downstream side on abutment.
Horizontal Alignment	Not applicable.
Movement	None observed except some erosion above riprap at the upstream side and local movement of stones on downstream wall face associated with tree growth.
Settlement	None observed.
Depression/Sinkhole	None observed.
Ruts and/or Puddles	None observed.
Surface Protection	Vegetation cover and riprap – appeared intact.
Erosion/ Weathering	Some erosion observed above the riprap on the upstream side.
Vegetation	Generally grass and vegetation. Some tree growth located on the upstream slope, and the downstream slope within the stacked stone wall. Vegetation assessment for recommended control measures (as necessary) under review.

Right (North) Earth Embankment- 160 m long

Crest	[Photos G11 to G12]
CONDITION	**OBSERVATION**
General	Embankment fill – type unknown. Concrete core wall exposed on the upstream side due to erosion and or local shallow sloughing of the upper portion of the slope. Exposed length is approximately 6 m long, located approximately 115 to 121 m from the North Abutment of the Spillway.
Surface Cracking	Snow covered during inspection so the crest was partially obscured. Some uneven areas observed but no evidence of surface cracking.
Horizontal Alignment	Some minor erosion at the upstream edge of the crest throughout the dam, causing misalignment of the crest edge. More significant erosion observed at core wall exposed area

DAM SAFETY GENERAL INSPECTION

Crest	[Photos G11 to G12]
CONDITION	**OBSERVATION**
	as noted above. Downstream side- no visual misalignment observed.
Settlement	Some uneven areas although no significant settlement observed.
Depression/Sinkhole	Some minor depression areas as noted above. No evidence of sinkholes.
Ruts and/or Puddles	None observed.
Surface Protection	Grass vegetation cover – appeared intact.
Vegetation	Generally grass and vegetation. Some tree growth at the crest of the upstream and downstream sides. Vegetation assessment for recommended control measures (as necessary) under review.
Animal Burrow	None observed.

Upstream Slope	[Photos G13 to G15]
CONDITION	**OBSERVATION**
Slope Stability	Fair. No signs of any active or historic slope failures or movements with the exception of the area of exposed concrete core wall noted above. Some erosion observed above the riprap along the majority of the slope away from the area of the exposed core wall. Note that the upper slope above the riprap was steeper due to loss of material from erosion –
Slope Protection	Riprap provides erosion protection. Primarily finer to larger size riprap (typically 75 mm to 450 mm) visible at surface. Some silt infill within riprap also observed.
Depression/Sinkhole	None observed.
Erosion	As mentioned above.
Abutment Contacts	North abutment of the dam blended well into the existing grade of the north shoreline.
Animal Burrow	None observed.
Vegetation	Generally grass and vegetation. Some tree growth at the upstream slope. Vegetation assessment for recommended control measures (as necessary) under review.

Downstream Slope and Toe Area	[Photos G16 to G18]
CONDITION	**OBSERVATION**
Slope Stability	Good- No signs of any active or historic slope failures. Erosion protected by grass and vegetation cover. Overall the slope was in good condition with no stability issues observed.
Unusual Movement	None observed.
Slope Protection	Generally grass and vegetation cover – appeared intact.
Seepage	Evidence of possible minor water flow and stagnated water observed at the ditch along the toe of the dam (Photo G17). The source of water is unknown. The water flow may be associated with snow melting at the time of inspection. There was no visual evidence of seepage through the dam or piping / internal erosion, although dense vegetation and snow partially obscured inspection.
Wet Area(s)	As mentioned above.
Depression/Sinkhole	None observed.
Condition of toe	Intact / no unusual movement.
Erosion	Minor erosion and small gullying observed approx. 10 m north of the concrete structure (approx. 0.45 m wide by 0.2 m deep - see Photo G18).Some erosion also noted in the bottom of the drainage ditch along the toe of the slope, particularly closer to the concrete structure where the ditch was deeper.
Abutment Contacts	The north abutment blended well into the existing grade and road surface along the headpond shoreline.
Animal Burrow	None observed.
Vegetation	Generally grass and vegetation. Some tree growth at the downstream slope. Vegetation assessment for recommended control measures (as necessary) under review.

DAM SAFETY GENERAL INSPECTION

Instrumentation	
CONDITION	**OBSERVATION**
Piezometer	None.
Survey Monuments	None.
Inclinometer	None.
Relief Well	None.
Weirs	None.

Left (South) Earthfill Embankment

Crest	[Photos G19]
CONDITION	**OBSERVATION**
General	Embankment fill – type unknown. The crest surface was uneven in some areas.
Surface Cracking	Snow covered so surface was partially obscured. Some uneven areas observed but no evidence of significant surface cracking.
Horizontal Alignment	Some erosion above riprap along the upstream edge of the crest throughout the dam caused misalignment of this edge. Downstream side- no visual misalignment observed.
Settlement	Some uneven areas, although no significant settlement observed.
Depression/Sinkhole	Some depression areas as mentioned above, although no evidence of significant sinkholes.
Ruts and/or Puddles	None observed.
Surface Protection	Grass vegetation cover – appeared intact.
Vegetation	Generally grass and vegetation. Some tree growth at the crest of the upstream and downstream sides. Vegetation assessment for recommended control measures (as necessary) under review.
Animal Burrow	None observed.

Upstream Slope	[Photos G20]
CONDITION	**OBSERVATION**
Slope Stability	Fair. No signs of any active or historic slope failures or movements. Some loss of upper portion of the slope due to erosion above the riprap. Overall the slope was in good condition with no stability issues observed.
Slope Protection	Riprap provides erosion protection. Primarily finer to larger size riprap (typically 75 mm to 450 mm or larger). Some silt infill within riprap also observed.
Depression/Sinkhole	None observed.
Erosion	As mentioned above.
Abutment Contacts	South abutment of dam blended well into the Talbot Canal Earthfill embankment.
Animal Burrow	None observed.
Vegetation	Generally grass and vegetation. Some tree growth at the upstream slope. Vegetation assessment for recommended control measures (as necessary) under review.

Downstream Slope and Toe Area	[Photos G21]
CONDITION	**OBSERVATION**
Slope Stability	Good- No signs of any active or historic slope failures. Overall the slope was in good condition with no stability issues observed.
Unusual Movement	None observed.
Slope Protection	Generally grass and vegetation cover – appeared intact.
Seepage	None observed.
Wet Area(s)	None observed.
Depression/Sinkhole	None observed.
Condition of toe	Intact.
Erosion	None observed. Erosion protection provided by intact vegetation cover.

DAM SAFETY GENERAL INSPECTION

Downstream Slope and Toe Area [Photos G21]	
CONDITION	**OBSERVATION**
Abutment Contacts	South abutment of dam blended well into the existing grade of the canal dyke slope and road.
Animal Burrow	None observed.
Vegetation	Generally grass and vegetation. Some tree growth at the Downstream slope. Vegetation assessment for recommended control measures (as necessary) under review.

Instrumentation	
CONDITION	**OBSERVATION**
Piezometer	None.
Survey Monuments	None.
Inclinometer	None.
Relief Well	None.
Weirs	None.

DAM SAFETY GENERAL INSPECTION

FIGURES

Figure G2- Talbot River Dam

DAM SAFETY GENERAL INSPECTION

PHOTOS

Photo G1- Headpond Shoreline (North Slope). Photo 1 of 2

Photo G2- Headpond Shoreline (North Slope). Photo 2 of 2

DAM SAFETY GENERAL INSPECTION

Photo G3- Headpond Shoreline (South Slope). Photo 1 of 1

Photo G4- Tailrace Channel (Talbot River Dam). Photo 1 of 2

DAM SAFETY GENERAL INSPECTION

Photo G5- Tailrace Shoreline (North Slope). Photo 2 of 2

Photo G6- Tailrace Shoreline (South Slope). Photo 1 of 1

DAM SAFETY GENERAL INSPECTION

Photo G7- North Abutment of the concrete structure at the dam (Upstream Side). Photo 1 of 2

Photo G8- North Abutment of the concrete structure at the dam (Downstream Side). Photo 2 of 2. Note some tree growth within the stacked stone wall

DAM SAFETY GENERAL INSPECTION

Photo G9- South Abutment of the concrete structure at the dam (Upstream Side). Photo 1 of 2

Photo G10- South Abutment of the concrete structure at the dam (Downstream Side). Photo 2 of 2. Note some tree growth within the stacked stone wall.

DAM SAFETY GENERAL INSPECTION

Photo G11- Crest (Right / North Earthfill Embankment). Photo 1 of 2

Photo G12- Crest (Right / North Earthfill Embankment). Photo 2 of 2

DAM SAFETY GENERAL INSPECTION

Photo G13- Upstream Slope of Right (North) Earthfill Embankment. Photo 1 of 3. Note some tree growth at the upstream slope.

Photo G14- Upstream Slope of Right (North) Earthfill Embankment. Photo 2 of 3. Note significant loss of upper portion of the slope, concrete core wall exposed and absence of riprap.

DAM SAFETY GENERAL INSPECTION

Photo G15- Upstream Slope of Right (North) Earthfill Embankment. Photo 3 of 3. Note steeper upper portion of the slope, finer size riprap and some tree growth at the upstream slope.

Photo G16- Downstream Slope of Right (North) Earthfill Embankment. Photo 1 of 3.

DAM SAFETY GENERAL INSPECTION

Photo G17- Downstream ditch at the toe of Right (North) Earthfill Embankment. Photo 2 of 3. Minor water flow and stagnant water observed in base of ditch.

Photo G18- Downstream Slope of Right (North) Earthfill Embankment. Photo 1 of 3. Note minor erosion and small gullying

DAM SAFETY GENERAL INSPECTION

Photo G19- Crest (Left / South Earthfill Embankment). Photo 1 of 1

Photo G20- Upstream Slope of Left (South) Earthfill Embankment. Photo 1 of 1. Note steeper upper portion of the slope and absence of riprap.

DAM SAFETY GENERAL INSPECTION

Photo G21- Downstream Slope of Left (South)Earthfill Embankment. Photo 1 of 1.

TALBOT CANAL

DAM SAFETY GENERAL INSPECTION

Northwest Canal Earthfill Embankment- STA 0+000 (Talbot River Dam) to STA 0+650

Crest	[Photos G1 and G2]
CONDITION	**OBSERVATION**
General	Intact. Embankment fill. The crest surface was uneven in some areas. The background information revealed that a steel sheet pile wall was installed along the crest of the embankment towards the upstream edge. As part of the 2013 geotechnical investigation, test pitting was performed by KGS Group at four selected locations in order to identify the sheet piles at the upstream side at Northwest and Southeast Canal embankments. The sheet piles were not encountered within the investigated depth and extent of the test pits. The discussion with PCA site staff during the investigation also suggests that the sheet piling was not installed between the Talbot River Dam and Lock 39; although, this has not been confirmed.
Surface Cracking	Snow covered. Some uneven areas observed but no evidence of surface cracking.
Horizontal Alignment	No evidence of misalignment.
Settlement	Some uneven areas although no significant settlement observed.
Depression/Sinkhole	Some depression areas as mentioned above. No evidence of sinkholes.
Ruts and/or Puddles	No significant rutting observed.
Surface Protection	Grass vegetation cover.
Vegetation	Generally grass vegetation. Some tree growth at / near the crest edge on both the upstream and downstream sides near the Talbot River Dam (Approx. STA 0+000 to 0+100). Vegetation assessment for recommended control measures (as necessary) under review.
Animal Burrow	None observed.

Upstream Slope	[Photos G3 to G10]
CONDITION	**OBSERVATION**
General	Earthfill slope. Material type is not known. A single gabion basket (900 X 900 mm) was installed along the canal near the mid portion of the upstream (canal side) slope. Riprap at the toe of the slope to the gabion basket provided protection from erosion. It appeared that granular fill (19 mm crushed stone) was placed behind and likely at the base of the gabion baskets. Some crushed stone was observed on the canal side of the baskets (Photo G4). No geotextile separator was observed between the gabion basket and the backfill. It was generally observed that the life expectancy of the gabion baskets has expired, with corrosion of the wire baskets frequently observed causing exposure of the inner rockfill (Photo G5). Note, however, that the baskets generally retained their shaped without significant collapse of the rockfill within the gabions.
Slope Stability	Poor to Fair. Some loss of material frequently observed behind the gabion basket (up to 600 mm deep from top of the gabion basket – see Photos G3 and G6). Significant loss of the upper portion of the slope behind the gabions was observed near STA 0+150. This area was approximately 10 m in length and was associated with erosion (Photo G9). The gabion baskets were frequently rotated and damaged / corroded, particularly from approximately STA 0+250 to ST 0+600 (Photo G6). Gabions from approximately STA 0+000 to STA 0+250 generally were in better condition compared to the other areas; although, there still was some rotation and basket damage observed. No signs of any significant active or historic deep seated failures of the overall slope observed.
Slope Protection	Riprap and gabion basket provided erosion protection.
Depression/Sinkhole	None observed except erosion mentioned above.
Erosion	Some local erosion around the ends of the concrete structure observed near STA 0+600. Some loss of material behind the gabion baskets as noted above.
Abutment Contacts	East side- blended well into the Talbot River Dam embankment. West side- gabion basket misalignment and rotation at abutments to concrete structure observed near STA 0+600 and some erosion around the ends of the concrete structure observed (Photos G7 and G8).
Animal Burrow	None observed.
Vegetation	Generally grass vegetation. Some tree growth and heavy vegetation on the upstream slope and within the gabion basket (Photo G10), particularly near the Talbot River Dam (approx. STA 0+100). Vegetation assessment for recommended control measures (as necessary) under review.

DAM SAFETY GENERAL INSPECTION

Downstream Slope and Toe Area [Photos G11]	
CONDITION	**OBSERVATION**
Slope Stability	Good- No signs of any active or historic slope failures. Primarily grass vegetation cover which provides erosion protection. Overall, the slope was in good condition with no stability issues observed.
Unusual Movement	None observed.
Slope Protection	Generally grass vegetation cover and a few trees.
Seepage	Snow covered. Difficult to observe although no seepage apparent.
Wet Area(s)	Snow covered. Difficult to observe although no wet areas apparent near the toe of the slope.
Depression/Sinkhole	None observed.
Condition of toe	Intact.
Erosion	Snow covered although no evidence of erosion.
Abutment Contacts	Intact.
Animal Burrow	None observed.
Vegetation	Generally grass vegetation.

Instrumentation	
CONDITION	**OBSERVATION**
Piezometer	None.
Survey Monuments	None.
Inclinometer	None.
Relief Well	None.
Weirs	None.

Northwest Canal Earthfill Embankment- STA 0+650 to STA 1+350 (Lock 39)

Crest [Photos G12 and G13]	
CONDITION	**OBSERVATION**
General	Intact. Access road to Lock 39. Embankment fill. Granular at surface. The crest surface was uneven in few areas.
Surface Cracking	Snow covered. Few uneven areas observed but no evidence of surface cracking.
Horizontal Alignment	No evidence of misalignment.
Settlement	Few uneven areas, although no significant settlement observed.
Depression/Sinkhole	Few depression areas as mentioned above. No evidence of sinkholes.
Ruts and/or Puddles	Minor ruts observed.
Surface Protection	Granular fill and grass.
Vegetation	Generally grass vegetation.
Animal Burrow	None observed.

Upstream Slope [Photos G14 to G17]	
CONDITION	**OBSERVATION**
General	Earthfill slope. Material type is not known. A single gabion basket (900 X 900 mm) was installed along the canal near the mid portion of the upstream (canal side) slope. Riprap at the toe of the slope to the gabion basket provided protection from erosion (Photos G15 and G16). It appeared that granular fill (19 mm crushed clear stone) was placed behind and likely at the base of the gabion basket. Some crushed stone was noted on the canal side of the baskets. No geotextile separator was observed between the gabion basket and the backfill. It was generally observed that the life expectancy of the gabion baskets has expired with corrosion of the wire baskets frequently observed causing exposure of the inner rockfill. Note however that the baskets generally retained their rectangular shape without significant collapse of the rockfill within the gabions.

DAM SAFETY GENERAL INSPECTION

Upstream Slope [Photos G14 to G17]	
CONDITION	**OBSERVATION**
Slope Stability	Poor to Fair. Some loss of material frequently observed behind the gabions (up to 600 mm deep from top of the baskets- see Photo G14), particularly from approximately STA 0+650 to STA 1+150. Loss of material could be associated with erosion due to canal water flow during high water season (water level reaches above the top of the gabion basket during high water season) or loss of fill through the baskets (crush stone frequently observed on canal side of the gabions within the riprap). The gabion baskets were rotated and damaged / corroded, particularly from approximately STA 0+650 to ST 1+150 (Photos G16 and G17). The baskets also appeared misaligned longitudinally at some locations from possible slope / gabion movement. No signs of any active or historic deep seated slope failures on the canal dyke were observed.
Slope Protection	Riprap and gabion basket provides erosion protection.
Depression/Sinkhole	None observed.
Erosion	Some loss of material behind the gabion basket as noted above.
Abutment Contacts	Blended well into the Lock 39 embankment.
Animal Burrow	None observed.
Vegetation	Generally grass vegetation. Some tree growth at the upstream slope and within the gabion basket. Vegetation assessment for recommended control measures (as necessary) under review.

Downstream Slope and Toe Area [Photos G18 and G19]	
CONDITION	**OBSERVATION**
Slope Stability	Good- No signs of any active or historic slope failures. Primarily grass vegetation cover which provides erosion protection. Overall, the slope was in good condition with no stability issues observed.
Unusual Movement	None observed.
Slope Protection	Generally grass.
Seepage	Snow covered. Difficult to observe although no seepage apparent.
Wet Area(s)	Snow covered. Difficult to observe although no wet areas apparent near the toe of the slope.
Depression/Sinkhole	None observed.
Condition of toe	Intact.
Erosion	Snow covered although no evidence of erosion.
Abutment Contacts	Intact.
Animal Burrow	None observed although vegetation and snow cover partially masked the slope.
Vegetation	Generally grass vegetation. Occasional tree growth on the downstream slope and toe area (Photos G18 and G19). Vegetation assessment for recommended control measures (as necessary) under review.

Instrumentation	
CONDITION	**OBSERVATION**
Piezometer	None.
Survey Monuments	None.
Inclinometer	None.
Relief Well	None.
Weirs	None.

DAM SAFETY GENERAL INSPECTION

Southeast Canal Earthfill Embankment- STA 0+000 (Talbot River Dam) to STA 0+650

Crest	[Photos G20 to G25]
CONDITION	**OBSERVATION**
General	Poor to fair. Embankment fill- Sandy silt / Clayey silt soils were observed in the disturbed areas within the embankment as shown in Figure G1. The crest surface was uneven. A few cuts were observed in the embankment between STA 0+050 and STA 0+250 (Photo G24). These disturbances appear to have been manmade, with an excavated trench along the crest of the dyke and the resulting material piled near the toe of the slope. It is speculated that the disturbances observed were made for farmland irrigation purposes, although this has not been confirmed.
Surface Cracking	Snow covered. Uneven areas observed but no evidence of surface cracking.
Horizontal Alignment	Significant erosion and loss of upper portion of the upstream slope and upstream edge of the crest approximately between STA +375 and STA 0+425 (Photo G25). Cuts were observed as specified above. Evidence of misalignment.
Settlement	Some depressions were observed between STA 0+050 and STA 0+450 (Photo G22).
Depression/Sinkhole	A few sinkholes were observed between STA 0+050 and STA 0+450 (Photo G23).
Ruts and/or Puddles	Man made cuts as specified above.
Surface Protection	Grass vegetation cover.
Vegetation	Generally grass vegetation.
Animal Burrow	None observed.

Upstream Slope	[Photos G25 to G28]
CONDITION	**OBSERVATION**
General	Earthfill slope- Sandy silt / Clayey Silt. A single gabion basket (900 X 900 mm) was installed along the canal near the mid portion of the upstream (canal side) slope. Riprap at the toe of the slope to the gabion basket provided protection from erosion. It appeared that granular fill (19 mm crushed clear stone) was placed behind and likely at the base of the gabion basket. Some crushed stone was noted on the canal side of the baskets. No geotextile separator was observed between the gabion basket and the backfill. It was generally observed that the life expectancy of the gabion baskets has expired with corrosion of the wire baskets frequently observed causing exposure of the inner rockfill. Note however that the baskets generally retained their rectangular shape without significant collapse of the rockfill within the gabions.
Slope Stability	Poor to Fair. Some loss of material frequently observed behind the gabion basket (up to 600 mm deep from top of the gabion basket- Photo G27). Significant loss of the upper portion of the slope behind the gabions was observed between approximately STA 0+375 and STA 0+425, associated with erosion (Photo G25). The gabion baskets were frequently rotated, mis-shapen and damaged / corroded (Photo G28). Cuts were observed between STA 0+050 and STA 0 + 250 as specified above. These could be associated with irrigation to farm field. No signs of any significant active or historic deep seated slope failures.
Slope Protection	Riprap and Gabion basket provided erosion protection.
Depression/Sinkhole	None observed except erosion as noted above.
Erosion	Some local erosion around the ends of the concrete structure observed near STA 0+600. Some loss of material behind the gabion basket as noted above.
Abutment Contacts	East side- blended into the existing grade of the Talbot River Dam shoreline -Elevation relative to design water levels unknown and should be confirmed. West side- gabion basket misalignment, mis-shapen and rotation at abutments to concrete structure observed near STA 0+600 and some erosion around the ends of the concrete structure observed (Photos G29 and G30).
Animal Burrow	None observed.
Vegetation	Generally grass vegetation. Some tree growth at the upstream slope between STA 0+500 and STA 0+650. Vegetation assessment for recommended control measures (as necessary) under review.

DAM SAFETY GENERAL INSPECTION

Downstream Slope and Toe Area [Photos G31 to G33]	
CONDITION	**OBSERVATION**
Slope Stability	Good- No signs of any active or historic slope failures except cuts as specified above and few depressions that could be associated with farm equipment movement within the embankment on the downstream slope (Photo G32 and G33). Grass vegetation cover provided erosion protection. Overall the slope was in good condition with no stability issues observed.
Unusual Movement	None observed.
Slope Protection	Generally grass vegetation cover.
Seepage	Snow covered. Difficult to observe although no seepage apparent.
Wet Area(s)	Snow covered. Difficult to observe although no wet areas apparent near the toe of the slope. There is a creek running near the toe of the dam from approximately STA 0+675 (culvert) to STA 0+550 (Photo G31).
Depression/Sinkhole	None observed.
Condition of toe	Intact. Man-made cuts and few depressions as noted above.
Erosion	Snow covered although no evidence of erosion.
Abutment Contacts	Intact.
Animal Burrow	None observed.
Vegetation	Generally grass vegetation.

Instrumentation	
CONDITION	**OBSERVATION**
Piezometer	None.
Survey Monuments	None.
Inclinometer	None.
Relief Well	None.
Weirs	None.

Southeast Canal Earthfill Embankment- STA 0+650 to STA 1+350 (Lock 39)

Crest [Photos G34 and G35]	
CONDITION	**OBSERVATION**
General	Intact. Embankment fill. Material type is not known. The crest surface was uneven in some areas. In order to by-pass the creek, a culvert was constructed below the canal near STA 0+675 (Photo G35).
Surface Cracking	Snow covered. Some uneven areas observed but no evidence of surface cracking.
Horizontal Alignment	No evidence of misalignment.
Settlement	Some uneven areas. No significant settlement observed.
Depression/Sinkhole	Some depression areas as mentioned above. No evidence of sinkholes.
Ruts and/or Puddles	No significant rutting observed.
Surface Protection	Grass vegetation cover.
Vegetation	Generally grass vegetation.
Animal Burrow	None observed.

Upstream Slope [Photos G36 to G39]	
CONDITION	**OBSERVATION**
General	Earthfill slope. Material type is not known. A single gabion basket (900 X 900 mm) was installed along the canal near the mid portion of the upstream (canal side) slope. Riprap at the toe of the slope to the gabion basket provided protection from erosion. It appeared that granular fill (19 mm crushed clear stone) was placed behind and likely at the base of the gabion basket. Some crushed stone was noted on the canal side of the baskets. No

DAM SAFETY GENERAL INSPECTION

Upstream Slope [Photos G36 to G39]	
CONDITION	**OBSERVATION**
	geotextile separator was observed between the gabion basket and the backfill. It was generally observed that the life expectancy of the gabion baskets has expired with corrosion of the wire baskets frequently observed causing exposure of the inner rockfill. Note however that the baskets generally retained their rectangular shape without significant collapse of the rockfill within the gabions.
Slope Stability	Poor to Fair. Some loss of material frequently observed behind the gabion basket (up to 600 mm deep from top of the gabion basket-Photos G36 to G38). Significant loss of the upper portion of the slope behind the gabion basket was observed between STA 0+750 and STA 0+800 associated with erosion (Photo G37). The gabion baskets were frequently rotated and damaged / corroded, particularly from approximately STA 0+650 to STA 1+250 (Photos G38 and G39). Gabions from approximately STA 1+250 to STA 1+350 generally were in better condition compared to the other remaining areas, although there still was some rotation and basket damage observed. No signs of any active or historic deep seated slope failures.
Slope Protection	Riprap and Gabion basket provide erosion protection.
Depression/Sinkhole	None observed except erosion mentioned above.
Erosion	Some loss of material and loss of the upper portion of the slope as noted above.
Abutment Contacts	Blended well into the Lock 39 embankment.
Animal Burrow	None observed.
Vegetation	Generally grass vegetation. Some tree growth at the upstream slope and within the gabion basket between STA 0+ 650 and STA 0+ 850. Vegetation assessment for recommended control measures (as necessary) under review.

Downstream Slope and Toe Area [Photos G40 and G41]	
CONDITION	**OBSERVATION**
Slope Stability	Good- No signs of any active or historic slope failures. Primarily grass vegetation cover which provides erosion protection. Overall the slope was in good condition with no stability issues observed.
Unusual Movement	None observed.
Slope Protection	Generally grass vegetation cover and trees.
Seepage	Snow covered. Difficult to observe although no seepage apparent.
Wet Area(s)	Snow covered. Difficult to observe although no wet areas apparent near the toe of the slope.
Depression/Sinkhole	None observed.
Condition of toe	Intact.
Erosion	Snow covered. Difficult to observe although no seepage apparent.
Abutment Contacts	Blend into the Lock 39 embankment.
Animal Burrow	None observed.
Vegetation	Generally trees, grass vegetation. Dense tree growth at the downstream slope and the toe, Impeded inspection. Vegetation assessment for recommended control measures (as necessary) under review.

Instrumentation	
CONDITION	**OBSERVATION**
Piezometer	None.
Survey Monuments	None.
Inclinometer	None.
Relief Well	None.
Weirs	None.

DAM SAFETY GENERAL INSPECTION

FIGURES

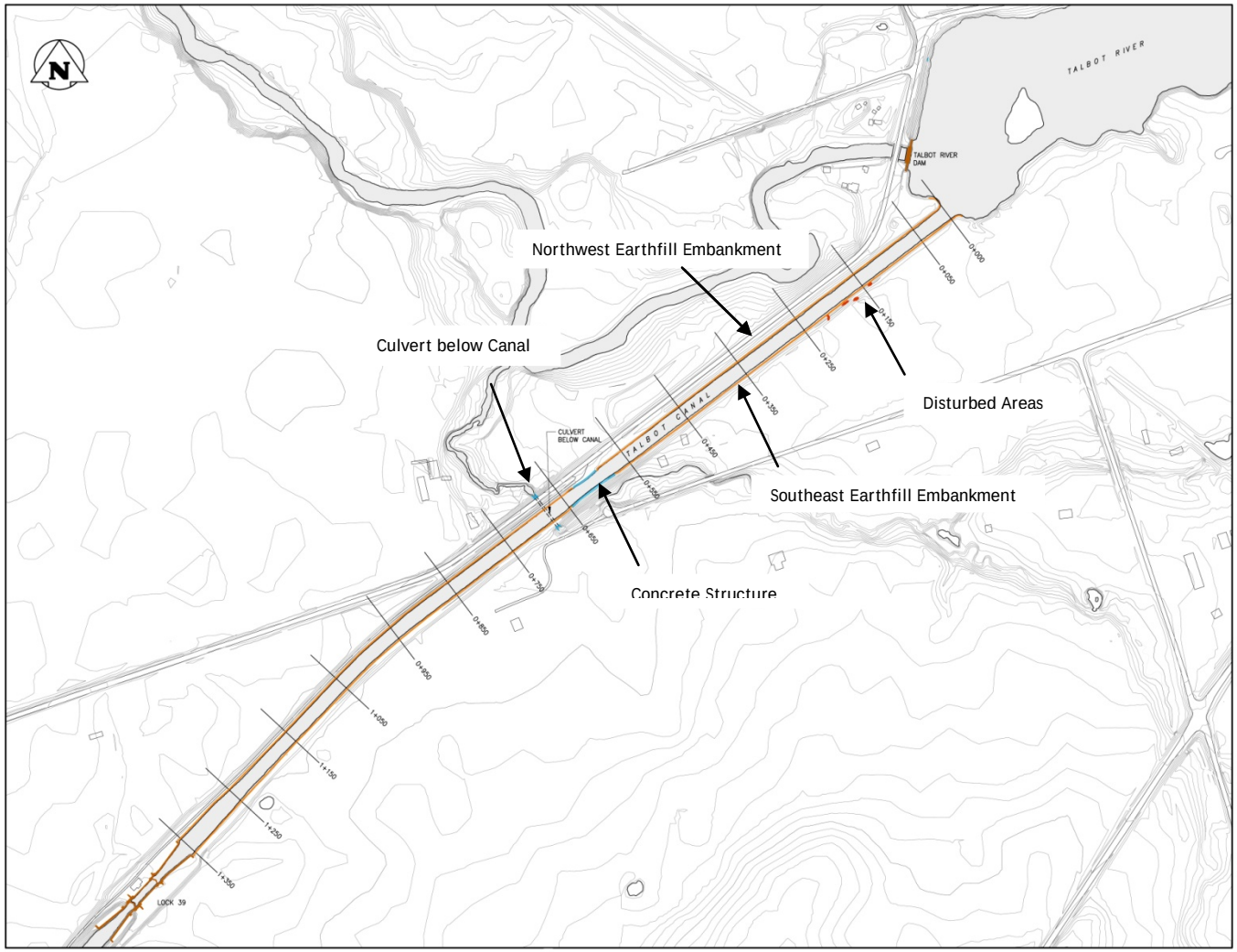

Figure G2- Talbot Canal (Talbot River Dam to Lock 39)

DAM SAFETY GENERAL INSPECTION
PHOTOS

Photo G1- Crest (Northwest Canal Earthfill Embankment) looking downstream from approximately STA 0+350.

Photo G2- Crest (Northwest Canal Earthfill Embankment) looking upstream from approximately STA 0+250 (Talbot River Dam in background on left)

DAM SAFETY GENERAL INSPECTION

Photo G3- Upstream slope (Northwest Canal Earthfill Embankment) looking downstream from approximately STA 0+300. Note Granular fill (19 mm crushed clear stone) behind the gabion basket and loss of fill material.

Photo G4- Lower portion of upstream slope (Northwest Canal Earthfill Embankment). Note 19 mm crushed stone at the base of the gabion basket.

DAM SAFETY GENERAL INSPECTION

Photo G5- Typical gabion basket damage and corrosion (Northwest Canal Earthfill Embankment near STA 0+050). Note exposure of the inner rockfill.

Photo G6- Upstream slope (Northwest Canal Earthfill Embankment). Note typical basket rotation near STA 0+400 and significant loss of material behind the gabions.

DAM SAFETY GENERAL INSPECTION

Photo G7- Upstream slope (Northwest Canal Earthfill Embankment). Note typical gabion basket rotation and misalignment at abutment to concrete structure (east end) near STA 0+ 600. Loss of material behind the gabions and some erosion around the end of the concrete structure observed.

Photo G8- Upstream slope (Northwest Canal Earthfill Embankment). Note gabion basket rotation / damage, mis-shapen and misalignment at abutment to concrete structure (west end) near STA 0+ 600. Loss of material behind the gabions and some erosion around the end of the concrete structure observed.

DAM SAFETY GENERAL INSPECTION

Photo G9- Upstream slope (Northwest Canal Earthfill Embankment). Note significant erosion of upper slope near STA 0+050.

Photo G10- Upstream slope (Northwest Canal Earthfill Embankment). Note tree growth within the gabion basket near STA 0+ 200.

DAM SAFETY GENERAL INSPECTION

Photo G11- Downstream slope (Northwest Canal Earthfill Embankment). Looking downstream from approximately STA 0+ 150. Note access road and ditch at toe of slope.

Photo G12- Crest (Northwest Canal Earthfill Embankment). Looking upstream from approximately STA 1+000.

DAM SAFETY GENERAL INSPECTION

Photo G13- Crest (Northwest Canal Earthfill Embankment).Looking downstream from approximately STA 0+700.

Photo G14- Upper portion of the upstream slope (Northwest Canal Earthfill Embankment) looking upstream from approximately STA 0+650. Note loss of material behind gabion basket.

DAM SAFETY GENERAL INSPECTION

Photo G15- Lower portion of the upstream slope (Northwest Canal Earthfill Embankment). Looking downstream from approximately STA 0+950.

Photo G16- Upstream slope (Northwest Canal Earthfill Embankment). Looking upstream from approximately STA 0+ 850. Note typical gabion rotation, corrosion damage, and misalignment of baskets.

DAM SAFETY GENERAL INSPECTION

Photo G17- Upstream slope (Northwest Canal Earthfill Embankment). Note gabion basket damage and corrosion, plus significant rotation and mis-shapen near STA 0+750.

Photo G18- Downstream slope (Northwest Canal Earthfill Embankment) looking downstream from approximately STA 1+150 with Lock 39 in background. Note brush and tree growth on the lower portion and at base of slope.

DAM SAFETY GENERAL INSPECTION

Photo G19- Downstream slope (Northwest Canal Earthfill Embankment). Looking downstream from approximately STA 0+ 750. Note tree growth on and at toe of slope.

Photo G20- Crest (Southeast Canal Earthfill Embankment). Looking upstream from approximately STA 0+350.

DAM SAFETY GENERAL INSPECTION

Photo G21- Crest (Southeast Canal Earthfill Embankment). Looking upstream from approximately STA 0+400.

Photo G22- Crest (Southeast Canal Earthfill Embankment). Looking downstream. Note significant depression near STA 0+450.

DAM SAFETY GENERAL INSPECTION

Photo G23- Crest (Southeast Canal Earthfill Embankment). Note sink hole near STA 0+ 250.

Photo G24- Crest (Southeast Canal Earthfill Embankment). Note typical man made cut across the embankment near STA 0+250, could be associated with irrigation to farm field.

DAM SAFETY GENERAL INSPECTION

Photo G25- Crest and upstream slope (Southeast Canal Earthfill Embankment). Note significant loss of slope behind the gabion basket associated with erosion approximately between STA 0+375 and STA 0+425.

Photo G26- Upstream slope (Southeast Canal Earthfill Embankment). Looking upstream from approximately 0+ 300. Note significant loss of material behind the gabions and significant disturbance near STA 0+250.

DAM SAFETY GENERAL INSPECTION

Photo G27- Upstream slope (Southeast Canal Earthfill Embankment). Looking downstream from approximately STA 0+200. Note Loss of material behind the gabions and typical gabion rotation, corrosion damage and misalignment of baskets.

Photo G28- Upstream slope (Southeast Canal Earthfill Embankment). Note Typical Gabion basket rotation, corrosion damage and misalignment near STA 0+000.

DAM SAFETY GENERAL INSPECTION

Photo G29- Upstream slope (Southeast Canal Earthfill Embankment). Note typical gabion basket rotation and misalignment at abutment to concrete structure (east end) near STA 0+ 600. Loss of material behind the gabions and some erosion around the end of the concrete structure observed.

Photo G30- Upstream slope (Southeast Canal Earthfill Embankment). Note gabion basket rotation / damage and misalignment at abutment to concrete structure (west end) near STA 0+ 600. Loss of material behind the gabions and some erosion around the end of the concrete structure observed.

DAM SAFETY GENERAL INSPECTION

Photo G31- Downstream slope (Southeast Canal Earthfill Embankment). Looking upstream from approximately STA 0 + 600. Note creek near toe of the dam

Photo G32- Downstream slope (Southeast Canal Earthfill Embankment). Looking upstream from approximately STA 0+ 250.

DAM SAFETY GENERAL INSPECTION

Photo G33- Downstream slope (Southeast Canal Earthfill Embankment). Note depression associated with farm equipment movement within the embankment near STA 1 + 150.

Photo G34- Crest (Southeast Canal Earthfill Embankment). Looking Upstream from approximately STA 1+250.

Photo G35- Upstream slope (Southeast Canal Earthfill Embankment). Looking downstream from approximately STA 0+950. Note loss of material behind the gabion baskets.

Photo G36- Upstream slope (Southeast Canal Earthfill Embankment). Looking downstream from approximately STA 0+700. Note significant loss of upper portion of the slope.

DAM SAFETY GENERAL INSPECTION

Photo G37- Upper portion of the upstream slope (Southeast Canal Earthfill Embankment). Looking upstream from approximately STA 1+ 050. Note gabion basket rotation and a significant loss of slope behind the Gabion basket associated with erosion.

Photo G38- Upstream slope (Southeast Canal Earthfill Embankment). Note typical gabion basket rotation and misalignment near concrete structure.

DAM SAFETY GENERAL INSPECTION

Photo G39- Culvert below the canal near STA 0+675.

Photo G40- Downstream slope (Southeast Canal Earthfill Embankment). Looking upstream from approximately STA 0+ 850

DAM SAFETY GENERAL INSPECTION

Photo G41- Downstream slope (Southeast Canal Earthfill Embankment). Looking upstream from approximately STA 1+050

LOCK 39

DAM SAFETY GENERAL INSPECTION

Right (North) Earthfill Embankment

Crest	[Photos G1]
CONDITION	**OBSERVATION**
General	Intact- Embankment fill and access road.
Surface Cracking	None observed.
Horizontal Alignment	Good. Upstream- Lock concrete wall. Downstream-crest edge was not even / continuously aligned, although no evidence of significant movement.
Settlement	No significant settlement observed.
Depression/Sinkhole	None observed.
Ruts and/or Puddles	None observed.
Surface Protection	Generally grass and some trees.
Vegetation	Generally grass and some trees.
Animal Borrow	None observed.

Upstream Slope (Canal Side)	[Photos G2]
CONDITION	**OBSERVATION**
General	Concrete wall at the Lock structure. Earthfill embankment extended downstream to canal. Note that there is little to no water retained by the earthfill embankment downstream from the lock structure. The embankment transitions from the lock area to the canal dykes, and provides vehicle access beyond the limits of the lock structure.
Slope Stability	No signs of any active or historic slope failures. Overall the slope was in good condition with no stability issues observed.
Slope Protection	Slope was generally grass covered and in good condition. Concrete retaining wall at toe of slope downstream from the structure provides slope protection.
Depression/Sinkhole	None observed.
Erosion	None observed.
Abutment Contacts	Blended well into the canal embankments.
Animal Borrow	None observed.
Vegetation	Generally grass.

Downstream (North) Slope and Toe Area	[Photos G3]
CONDITION	**OBSERVATION**
Slope Stability	No signs of any significant active or historic slope failures. Overall the slope was in good condition with no stability issues observed.
Unusual Movement	No significant or unusual movement observed.
Slope Protection	Generally heavy brush and grass with dense minor trees.
Seepage	None observed.
Wet Area(s)	None observed.
Depression/Sinkhole	Difficult to observe due to dense vegetation growth and snow cover, although none observed. Heavy brush with dense minor trees impeded inspection.
Condition of toe	Difficult to observe due to dense vegetation growth although no major concerns observed. Heavy brush with dense minor trees impeded inspection.
Erosion	Difficult to observe due to dense vegetation growth although no significant erosion observed. Heavy brush with dense minor trees impeded inspection.
Abutment Contacts	Blended well into the canal embankments.
Animal Borrow	Difficult to observe due to dense vegetation growth although none observed.
Vegetation	Heavy brush with dense minor trees on the slope. Some mature trees observed at the crest edge of the downstream slope. Heavy brush with dense minor trees on the slope impeded inspection. Vegetation assessment for recommended control measures (as necessary) under review.

DAM SAFETY GENERAL INSPECTION

Instrumentation	
CONDITION	**OBSERVATION**
Piezometer	None.
Survey Monuments	None.
Inclinometer	None.
Relief Well	None.
Weirs	None.

Left (South) Earthfill Embankment

Crest [Photos G4]	
CONDITION	**OBSERVATION**
General	Intact- Embankment fill.
Surface Cracking	Snow covered although none observed.
Horizontal Alignment	Good. Upstream- Lock concrete wall. Downstream-crest edge was not even or continuously aligned, however no evidence for significant movement.
Settlement	No significant settlement observed.
Depression/Sinkhole	None observed.
Ruts and/or Puddles	None observed.
Surface Protection	Generally grass and some trees.
Vegetation	Generally grass and some trees.
Animal Borrow	None observed.

Upstream Slope (Canal Side) [Photos G5]	
	OBSERVATION
General	Concrete wall at the Lock structure. Earthfill embankment extended downstream to canal. Note that there is little to no water retained by the earthfill embankment downstream from the lock structure. The embankment transitions from the lock area to the canal dykes, and provides vehicle access beyond the limits of the lock structure..
Slope Stability	No signs of any active or historic slope failures. Overall the slope was in good condition with no stability issues observed.
Slope Protection	Slope was generally grass covered and in good condition. Concrete retaining wall at toe of slope downstream from the structure provides slope protection
Depression/Sinkhole	None observed.
Erosion	None observed.
Abutment Contacts	Blended well into the canal embankments.
Animal Borrow	None observed.
Vegetation	Generally grass.

Downstream (South) Slope and Toe Area [Photos G6]	
CONDITION	**OBSERVATION**
Slope Stability	No signs of any significant active or historic slope failures. Overall the slope was in good condition with no stability issues observed.
Unusual Movement	No significant or unusual movement observed.
Slope Protection	Generally heavy brush and grass with dense minor trees.
Seepage	None observed.
Wet Area(s)	None observed.
Depression/Sinkhole	Difficult to observe due to dense vegetation growth although none observed. Heavy brush with dense minor trees impeded inspection.
Condition of toe	Difficult to observe due to dense vegetation growth although no major concerns observed. Heavy brush with dense minor trees impeded inspection.
Erosion	Difficult to observe due to dense vegetation growth although no significant erosion

DAM SAFETY GENERAL INSPECTION

Downstream (South) Slope and Toe Area [Photos G6]	
CONDITION	**OBSERVATION**
	observed. Heavy brush with dense minor trees impeded inspection.
Abutment Contacts	Blended well into the canal embankments.
Animal Borrow	Difficult to observe due to dense vegetation growth although none observed.
Vegetation	Heavy brush with dense minor trees on the slope. Some mature trees observed at the crest edge of the downstream slope. Heavy brush with dense minor trees on the slope impeded inspection. Vegetation assessment for recommended control measures (as necessary) under review.

Instrumentation	
CONDITION	**OBSERVATION**
Piezometer	None
Survey Monuments	None
Inclinometer	None
Relief Well	None
Weirs	None

DAM SAFETY GENERAL INSPECTION

FIGURES

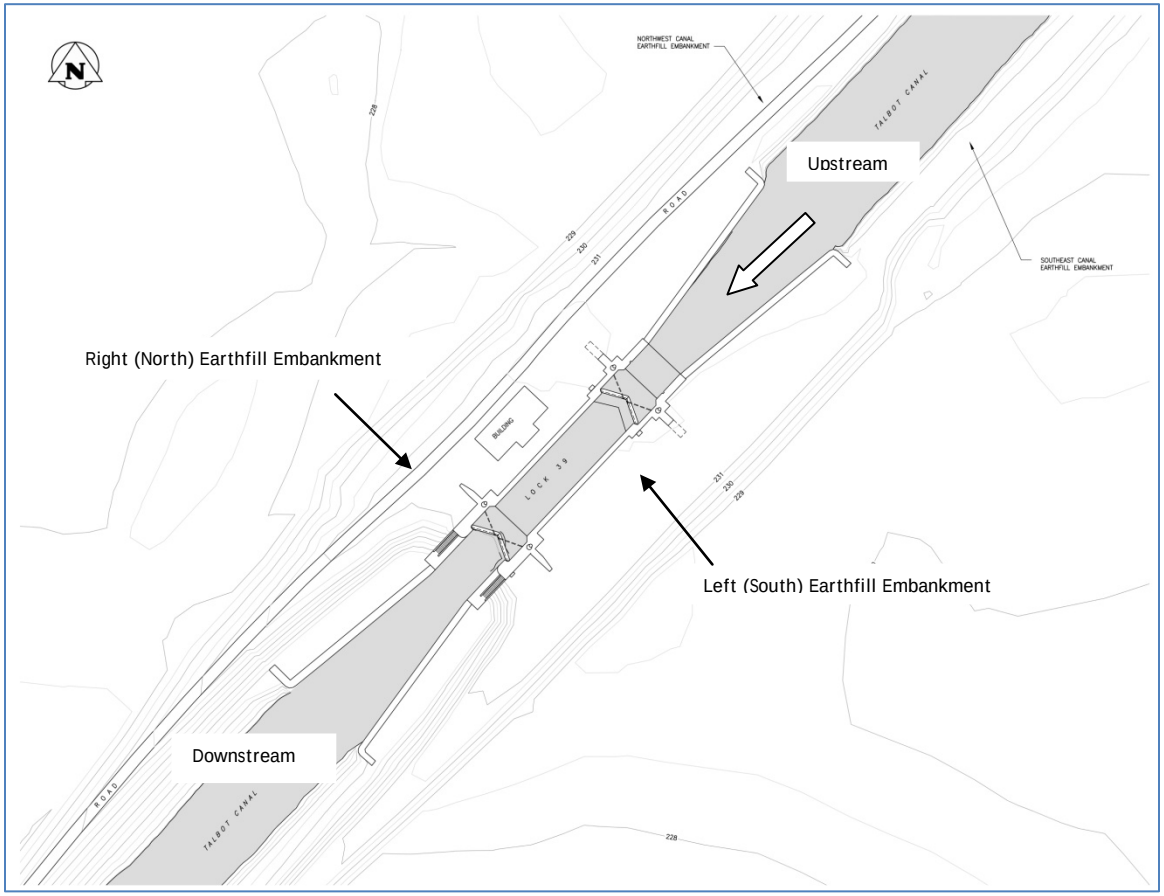

Figure G3- Lock 39

DAM SAFETY GENERAL INSPECTION

PHOTOS

Photo G1- Crest- Right (North) Earthfill Embankment. Photo 1 of 1

Photo G2- Upstream slope (canal side) of Right (North) Earthfill Embankment at downstream channel. Photo 1 of1

DAM SAFETY GENERAL INSPECTION

Photo G3- Downstream slope of Right (North) Earthfill Embankment. Photo 1 of 1

Photo G4- Crest- Left (South) Earthfill Embankment. Photo 1 of 1

DAM SAFETY GENERAL INSPECTION

Photo G5- Upstream slope (canal side) of Left (South) Earthfill Embankment of downstream channel. Photo 1 of 1

Photo G6- Downstream slope of Left (South) Earthfill Embankment. Photo 1 of 1

APPENDIX D

MECHANICAL EQUIPMENT

INSPECTION CHECK SHEETS

Stoplog Sluice
Condition Assessment Check List

Waterway: Trent-Severn **Site**: Talbot River Dam (near Lock 39)

Equipment: Stoplog Hoisting **Prepared by:** A. Garro / K. Besser **Date:** November 28, 2012 / September 24, 2013*
*= date of function test

Item	Rating	Reason for Rating	Remaining Life (Years)
Winches	2	Frames are structurally sound and coated. They are well secured and pad locked to the deck. Minor wear on spur gear tooth faces. Pinions have some wear on tooth faces. Gears are not lubricated, but shaft journals are. Some gears are not fully guarded. The wire rope is in good condition but is not lubricated. Winches operated well during functional testing in September 2013,with no unusual noise or excess effort.	25 (except wire ropes)
Wooden Logs	4	Twelve logs on the dam in December 2012 were in good condition and free of rot. The ends of one of the logs were slightly damaged. Remaining logs were installed in the sluice and were not visible for inspection. One of the logs on the dam is a spare. Five additional spare logs are stored at the Kirkfield shop for dams throughout the region.	10
Steel Half Log	4	Structurally sound and painted.	25
Gains	3	Sections visible from the dam deck appear in good condition except in one location. The south gain of the north sluice is missing some concrete adjacent to the downstream steel liner.	25
Sluice Crossers	4	Good condition. Some coating is missing in wear areas.	25
Sluice	4	Wooden stop logs are used as covers. Logs are in good condition.	15

Rating	Condition Description	Details
1	Unacceptable	Extensive deterioration. Barely functional. Maybe unsafe to operate.
2	Poor	Serious deterioration of at least some portions. Function is inadequate.
3	Fair	Moderate deterioration. Function is still adequate.
4	Good	Only minor deterioration or defects are evident.

Stoplog Sluice
Condition Assessment Check List

Item	Rating	Reason for Rating	Remaining Life (Years)
Covers		The six log support brackets are in good condition. Coating is missing in some areas.	
Rails	2	Rails are inadequately secured to the deck in some areas (> 6 feet between rail clips, maximum 3 feet recommended), some rail clips are loose, some are not clamping the rail and some clips are missing. Rails are misaligned over the north sluice.	25
Water Level Gauge	4	The water level gauge was installed in 2004. It appears to be in good condition and is well secured to the upstream face of the dam. The enclosure access door is locked. The gauge is calibrated every two years by the water control engineer and requires very little maintenance. It is in good functioning condition. It freezes in cold weather at which time data is inaccurate.	15
Winch Handles	4	Winch handles are also used for Talbot Dam winches. The two handles used during equipment testing in September 2013 were in good condition.	25
Pike Poles	4	Pike poles are not dedicated to this dam. One was brought to site for equipment testing in September 2013. It was in good condition.	15
Peavies	4	Peavies are not dedicated to this dam. The two peavies used during equipment testing in September 2013 were in good condition.	15
Log Stacking Ramps	N/A	Ramps are not required. There is sufficient space on the dam deck to store all 10 logs from each sluice.	-
Log Jack	4	The jack is not dedicated to this dam. It is stored at Lock 38 and is typically only used for winter sluice set up. It is in good condition with no oil leaks. Staff reported no known operating problems with the jack or the jacking ancillary equipment.	15

Rating	Condition Description	Details
1	Unacceptable	Extensive deterioration. Barely functional. Maybe unsafe to operate.
2	Poor	Serious deterioration of at least some portions. Function is inadequate.
3	Fair	Moderate deterioration. Function is still adequate.
4	Good	Only minor deterioration or defects are evident.

Lock Gates
Condition Assessment Check List

Waterway: Trent-Severn **Site**: Lock 39 **Gate:** Upstream

Description: Stacked timber mitre gates, manually operated with wire rope driven gate arms

Prepared by: A. Garro / K. Besser **Date:** November 28, 2012 / September 24, 2013 (functional test date)

1.0 Gate Leaves

Item	Rating	Reason for Rating	Remaining Life (Years)
Gate Leaves	4	Wooden gate timbers and end girders are in good condition. No leakage between timbers. The two gate leaves are coated.	20
Metal Tie Rods • Vertical • Diagonal	4	Diagonal tie rods are in good condition. Vertical tie rods are mostly concealed inside gate timbers preventing visual inspection. Exposed tie rod ends could not be accessed for close visual inspection.	20 for diagonal rods Unknown for vertical rods
Joints and Bolts		N/A	
Other Comments			

2.0 Lock Gate Supports

Item	Rating	Reason for Rating	Remaining Life (Years)
Gudgeons	4	Gudgeons appear solid and the pins have evidence of lubrication.	20
Pintles	4	Cannot access for visual inspection but operated smoothly during functional testing in September 2013.	20

Rating	Condition Description	Details
1	Unacceptable	Extensive deterioration. Barely functional. Maybe unsafe to operate.
2	Poor	Serious deterioration of at least some portions. Function is inadequate.
3	Fair	Moderate deterioration. Function is still adequate.
4	Good	Only minor deterioration or defects are evident.

Item	Rating	Reason for Rating	Remaining Life (Years)
Gudgeon Diagonal Tensioners	4	Tensioning rods are securely anchored to each steel bracket. Brackets appear to be in as new condition.	20
Other Comments		Gates operated smoothly during functional testing in September 2013, without unusual noise or vibration.	

3.0 Lock Gate Seals and Seal Seats

Item	Rating	Reason for Rating	Remaining Life (Years)
Centre Seals	4	No water leakage from centre of the gate.	20
Bottom Seals	4	No water leakage from bottom of the gate.	20
Outer Seals	4	No water leakage from sides of the gate.	20
Other Comments			

4.0 Lock Gate Actuators

Item	Rating	Reason for Rating	Remaining Life (Years)
Handles/Hand Wheels	3	Operator hand wheels and extensions are well coated. Hand wheels have a lot of play between the hub and the actuator square drive shaft. Currently no impact on performance but excessive play is accelerating wear. Operator handles are not locked when staff are off site thus facilitating removal by vandals.	10
Stands	4	Stands coated and in good condition.	25
Drive Systems	3	Wire ropes in good condition. Gate arm beams in good condition.	10

Rating	Condition Description	Details
1	Unacceptable	Extensive deterioration. Barely functional. Maybe unsafe to operate.
2	Poor	Serious deterioration of at least some portions. Function is inadequate.
3	Fair	Moderate deterioration. Function is still adequate.
4	Good	Only minor deterioration or defects are evident.

Item	Rating	Reason for Rating	Remaining Life (Years)
		Sheaves not accessible for visual inspection. Operate smoothly.	
Other Comments		Because of excess play between hand wheel hubs and actuator drive shafts, the hand wheels and their extensions tip off-horizontal. This presents an ergonomic problem for average to tall persons. Lock gates are chained and padlocked together over night to prevent lock gate actuation by vandals but are not locked during the day when staff are not present. Actuators operated well during functional testing in September 2013, but require a high level of force to get the gates moving from a stopped position.	

Rating	Condition Description	Details
1	Unacceptable	Extensive deterioration. Barely functional. Maybe unsafe to operate.
2	Poor	Serious deterioration of at least some portions. Function is inadequate.
3	Fair	Moderate deterioration. Function is still adequate.
4	Good	Only minor deterioration or defects are evident.

Lock Filling and Draining Valves
Condition Assessment Check List

Waterway: Trent-Severn **Site**: Lock 39 **Gates:** Upstream

Description: Rectangular butterfly valves located on the timber mitre gates, with manually operated actuators.

Prepared by: A. Garro / K. Besser **Date:** November 28, 2012 / September 24, 2013 (functional test date)

1.0 Lock Filling Valves

Item	Rating	Reason for Rating	Remaining Life (Years)
Hoist/Actuators	2	Actuator frames are coated and are in good condition. Gears are coated. Gears and pinions are well lubricated. There is some minor wear on the face of the pinions. The rack appears sound. Gears and pinions are not fully guarded. Actuator handles can easily be removed by vandals when staff are not present. Handles are removed by staff overnight to reduce the probability of loss. There are no means to lock the actuators and prevent undesired valve operation when staff are not present.	25
Operating Mechanisms	4	The operating shafts between the actuators and the valves appear in good condition from the lock gate walkway. Mounting of shafts to the gate leaves appears sound.	25
Valves	2	The valve leaves are not accessible and are far from the dam deck but appear to be solid. Leakage is excessive, even when high force is exerted by lock operators at the valve actuators.	5
Valve Supports	4	Valves appear to be solidly supported on the lock gate leaves.	25
Other Comments		Actuators operated well and valves moved smoothly during functional testing in September 2013.	

Rating	Condition Description	Details
1	Unacceptable	Extensive deterioration. Barely functional. Maybe unsafe to operate.
2	Poor	Serious deterioration of at least some portions. Function is inadequate.
3	Fair	Moderate deterioration. Function is still adequate.
4	Good	Only minor deterioration or defects are evident.

Lock Gates
Condition Assessment Check List

Waterway: Trent-Severn **Site**: Lock 39 **Gate:** Downstream

Description: Stacked timber mitre gates, manually operated with wire rope driven gate arms

Prepared by: A. Garro / K. Besser **Date:** November 28, 2012 / September 24, 2013 (functional test date)

1.0 Gate Leaves

Item	Rating	Reason for Rating	Remaining Life (Years)
Gate Leaves	4	Wooden gate timbers and end girders are in good condition. Very minor leakage between timbers in one location on one gate leaf. The two gate leaves do not appear to have been coated recently.	20
Metal Tie Rods • Vertical • Diagonal	4	Diagonal tie rods are in good condition. Vertical tie rods are mostly concealed inside gate timbers preventing visual inspection. Exposed tie rod ends could not be accessed for close visual inspection.	20 for diagonal rods Unknown for vertical rods
Joints and Bolts		N/A	
Other Comments			

2.0 Lock Gate Supports

Item	Rating	Reason for Rating	Remaining Life (Years)
Gudgeons	4	Gudgeons appear solid and the pins have evidence of lubrication.	20
Pintles	4	Cannot access for visual inspection but operated smoothly during functional testing in September 2013.	20

Rating	Condition Description	Details
1	Unacceptable	Extensive deterioration. Barely functional. Maybe unsafe to operate.
2	Poor	Serious deterioration of at least some portions. Function is inadequate.
3	Fair	Moderate deterioration. Function is still adequate.
4	Good	Only minor deterioration or defects are evident.

Item	Rating	Reason for Rating	Remaining Life (Years)
Gudgeon Diagonal Tensioners	4	Tensioning rods are securely anchored to each steel bracket. Brackets appear to be in as new condition.	20
Other Comments		Gates operated smoothly during functional testing in September 2013, without unusual noise or vibration.	

3.0 Lock Gate Seals and Seal Seats

Item	Rating	Reason for Rating	Remaining Life (Years)
Centre Seals	4	No water leakage from centre of the gate.	20
Bottom Seals	4	Bottom of gate submerged but appears to be no water leakage since no turbulence observed in water at bottom of gate.	20
Outer Seals	4	No water leakage from sides of the gate.	20
Other Comments			

4.0 Lock Gate Actuators

Item	Rating	Reason for Rating	Remaining Life (Years)
Handles/Hand Wheels	3	Operator hand wheels and extensions are well coated. Hand wheels have a lot of play between the hub and the actuator drive shaft. Currently no impact on performance but excessive play is accelerating wear. Operator handles are not locked when staff are off site thus facilitating removal by vandals.	10
Stands	4	Stands coated and in good condition.	25
Drive Systems	3	Wire ropes in good condition.	10

Rating	Condition Description	Details
1	Unacceptable	Extensive deterioration. Barely functional. Maybe unsafe to operate.
2	Poor	Serious deterioration of at least some portions. Function is inadequate.
3	Fair	Moderate deterioration. Function is still adequate.
4	Good	Only minor deterioration or defects are evident.

Item	Rating	Reason for Rating	Remaining Life (Years)
		Gate arm beams in good condition. Sheaves not accessible for visual inspection. Operate smoothly.	
Other Comments		Because of excess play between hand wheel hubs and actuator drive shafts, the hand wheels and their extensions tip off-horizontal. This presents an ergonomic problem for average to tall persons. Lock gates are chained and padlocked together over night to prevent lock gate actuation by vandals but are not locked during the day when staff are not present. Actuators operated well during functional testing in September 2013, but require a high level of force to get the gates moving from a stopped position.	

Rating	Condition Description	Details
1	Unacceptable	Extensive deterioration. Barely functional. Maybe unsafe to operate.
2	Poor	Serious deterioration of at least some portions. Function is inadequate.
3	Fair	Moderate deterioration. Function is still adequate.
4	Good	Only minor deterioration or defects are evident.

Lock Filling and Draining Valves
Condition Assessment Check List

Waterway: Trent-Severn **Site**: Lock 39 **Gates:** Downstream

Description: Rectangular butterfly valves located on the timber mitre gates, with manually operated actuators.

Prepared by: A. Garro / K. Besser **Date:** November 28, 2012 / September 24, 2013 (functional test date)

1.0 Lock Draining Valves

Item	Rating	Reason for Rating	Remaining Life (Years)
Hoist/Actuators	2	Actuator frames are coated and are in good condition. Gears are coated. Gears and pinions are well lubricated. There is some minor wear on the face of the pinions. The rack appears sound. Gears and pinions are not fully guarded. Actuator handles can easily be removed by vandals when staff are not present. Handles are removed by staff overnight to reduce the probability of loss. There are no means to lock the actuators and prevent undesired valve operation when staff are not present.	25
Operating Mechanisms	4	The operating shafts between the actuators and the valves appear in good condition from the lock gate walkway. Mounting of shafts to the gate leaves appears sound.	25
Valves	4	The valve leaves are not accessible and are far from the dam deck but appear to be solid. Leakage is minimal.	15
Valve Supports	4	Valves appear to be solidly supported on the lock gate leaves.	25
Other Comments		Actuators operated well and valves moved smoothly during functional testing in September 2013.	

Rating	Condition Description	Details
1	Unacceptable	Extensive deterioration. Barely functional. Maybe unsafe to operate.
2	Poor	Serious deterioration of at least some portions. Function is inadequate.
3	Fair	Moderate deterioration. Function is still adequate.
4	Good	Only minor deterioration or defects are evident.

APPENDIX E

MECHANICAL EQUIPMENT

FUNCTIONAL TEST CHECK SHEETS

Dam Safety Review
Sluice Stoplog Installation Test
Test Record Sheet

Date: Tuesday, 24 Sep 2013 Prepared by: Ken Besser

Site: Talbot River Dam Sluice: South

Weather Conditions (including temperature): Sunny, 14 °C

Headwater Level Start: 230.427 m Headwater Level Finish: 230.427 m

Number of Logs in Sluice: 8 Number of Logs to Install: 2

Prior to commencing the test:

- Confirm that stakeholders upstream and downstream of the dam have been previously notified that water levels and flow will change.
- Complete JSA and review work plan.

Task Description:	Start:	Finish:	Duration (min/sec)	Reasons for Delays, Comments (note condition of each log removed)
1. Obtain portable equipment from shop storage, inspect, load on truck *			1:00 ±	Obtain truck and pike pole. *
2. Drive to site *			16:00	Drive to Lock 38 control building. *
			1:05	Obtain winch handles and peavies from Lock 38 control building. *
			1:55	Drive from Lock 38 to dam. *
3. Transport equipment from truck to work area (winches, winch handles, pike poles, peavies) and confirm no upstream debris or obstructions *			1:30	Transport winch handles and peavies. *
4. Unlock and inspect site permanent equipment (winches, winch support frames and anchoring, sluice crosses, life ring) *			---	Nothing to unlock.
5. Ensure areas above			---	Log installation will not result in

and below dam are clear of public, contractors and staff				significant changes in water level upstream of the dam or in flow downstream of the dam that could impact public safety. Formal check upstream and downstream not required. Operators scan area immediately surrounding the dam as they perform Task #3.
6. Position winch frames on either side of sluice and anchor to deck *			---	Winch frames already in position.
7. Install winches *			---	Installation of handles is included in Task #3 above.
8. Remove sluice covers *			2:10	Task duration determined from log removal test. *
9. Remove jack stands			N/A	No jack stands in place during boating season. Jack stands only used in winter.
Mobilization:			23:40	
10. Install log #1	10:24:05	10:25:20	1:15	Log was already hanging from winch hooks, lifted above deck level, sluice crossers removed and ready to be lowered. Log lowered 1 m to 2 m with winches and then winch was released to free wheel and allow log to gravity drop to bottom. Log #1 was installed without difficulty.
11. Install log #2 **	10:25:20	10:27:55	2:35	Log lowered 1 m to 2 m with winches and then winch was released to free wheel and allow log to gravity drop to bottom. Log #2 was installed without difficulty.
Log Installation:			2:35	to install 1 log.
12. Jack logs down and install jack sands			N/A	Jack stands not used during boating season, only in winter.
13. Replace sluice covers	10:27:55	10:29:30	1:35	Sluice crossers are placed across the sluice opening, the cover log is moved into position on the crossers,

				the winches are used to raise the log off the crossers, the cover support brackets are placed in the opening and the sluice crossers are removed, the log is lowered onto the brackets in the opening.
14. Secure winch hooks, remove winch handles and gather portable equipment	10:29:30	10:30:20	0:50	
15. Lock equipment			N/A	
16. Transport portable equipment back to truck	10:31:50	10:32:40	0:50	Winch handles and peavies.
17. Drive to shop			1:55	Drive from dam to Lock 38 control building. *
			1:05	Return winch handles and peavies to storage at Lock 38 control building. *
			16:00	Drive from Lock 38 control building to Kirkfield shop. *
18. Return portable equipment to storage			1:00	Return pike pole to storage in Kirkfield shop. *
Demobilization:			23:15	
Log Installation Operation:			49 min ±	to install 1 log.

* Task duration determined from log removal test. See Sluice Stoplog Removal Test – Test Record Sheet for detailed comments.

** Install log typically includes: place sluice crossers across sluice opening, position log on sluice crossers, engage winch hooks in dees, raise log off crossers, remove crossers, lower log into sluice, disengage winch hooks, raise hooks.

Dam Safety Review
Sluice Stoplog Removal Test
Test Record Sheet

Date: Tuesday, 24 Sep 2013 Prepared by: Ken Besser

Site: Talbot River Dam Sluice: South

Weather Conditions (including temperature): Sunny, 14 °C

Headwater Level Start: 230.427 m Headwater Level Finish: 230.427 m

Number of Logs in Sluice: 10 Number of Logs to Remove: 2

Prior to commencing the test:

- Confirm that stakeholders upstream and downstream of the dam have been previously notified that water levels and flow will change.
- Complete JSA and review work plan.

Task Description	Start	Finish	Duration (min/sec)	Reasons for Delays, Comments (note condition of each log removed)
1. Obtain portable equipment from shop storage, inspect, load on truck			1:00 ±	Two Operators obtained truck and pike pole from Kirkfield shop. Remaining portable equipment stored at Lock 38 control building. Duration estimated since staff had already loaded the pike pole on the truck prior to KGS arrival.
2. Drive to site	9:06	9:22	16:00	Drove to Lock 38 control building, distance = 16.6 km.
	10:06:40	10:07:45	1:05	Winch handles and peavies were obtained from Lock 38 control building.
	10:08:50	10:10:45	1:55	Drive from Lock 38 to dam, distance = 1.3 km.
3. Transport equipment from truck to work area (winch handles, pike poles, peavies) and confirm no upstream debris or obstructions	10:10:45	10:12:15	1:30	Winch handles and peavies only were transported. It was not anticipated that pike poles would be needed.
4. Unlock and inspect site permanent	---	---	---	Nothing to unlock.

Task Description	Start	Finish	Duration (min/sec)	Reasons for Delays, Comments (note condition of each log removed)
equipment (winches, winch support frames and anchoring, sluice crosses, life ring)				
5. Ensure areas above and below dam are clear of public, contractors and staff	10:12:15	10:16:50	4:35	Two kayakers wanted to launch kayaks downstream of dam. Parks Canada staff asked them to wait until after log operations were completed.
6. Position winch frames on either side of sluice and anchor to deck	---	---	---	Winch frames were already in position upon arrival at dam.
7. Install winch handles	---	---	---	Installation of handles is included in Task #3 above.
8. Remove sluice covers *	10:17:10	10:19:20	2:10	Sluice cover is a log suspended from the dam deck by three steel hanger brackets. Log was removed using winches. Hanger brackets removed by hand. Log was in good condition.
9. Remove jack stands			N/A	No jack stands were in place.
Mobilization:			28:15	
10. Remove log #1 *	10:19:20	10:21:55	2:35	Minimal water flow overtopped the installed logs. Hook engagement was easy without using pike poles. Log #1 was removed without difficulty. Log was in good condition.
11. Remove log #2	10:21:55	10:23:55	2:00	Log was lifted above deck level and then removal operation was stopped, ie does not include lowering log onto sluice crossers, disengaging from hooks, moving log onto deck and removal of sluice crossers. Hook engagement slightly more tricky but pike pole was not needed. Log was in good condition.
Log Removal:			2:35	to remove 1 log.

Task Description	Start	Finish	Duration (min/sec)	Reasons for Delays, Comments (note condition of each log removed)
12. Jack logs down and install jack stands			N/A	Jack stands not used during boating season, only in winter.
13. Replace sluice covers **			1:35	These steps were performed after the log installation test. Durations and details are provided in the Sluice Stoplog Installation Test – Test Record Sheet.
14. Secure winch hooks, remove winch handles and gather portable equipment **			0:50	
15. Lock equipment **			N/A	
16. Transport portable equipment back to truck **			0:50	
17. Drive to shop			1:55	Drive from dam to Lock 38 control building. KGS unable to witness since PCA personnel had to travel to other sites before KGS able to follow. Duration is assumed equal to duration for Task #2.3.
			1:05	Return winch handles and peavies to storage at Lock 38 control building. Duration is assumed equal to duration for Task #2.2.
			16:00	Drive from Lock 38 control building to Kirkfield shop. Duration is assumed equal to duration for Task #2.1.
18. Return portable equipment to storage			1:00	Return pike pole to storage in Kirkfield shop. Duration is assumed equal to duration for Task #1.
Demobilization:			23:15	
Log Removal Operation:			55 min ±	to remove 1 log.

* Remove log typically includes: lower hooks into gain, engage dees, raise log, install sluice crossers, lower log on crossers, disengage winch hooks, move / stack log on deck, remove crossers from sluice.

** Task duration determined from log installation test.

APPENDIX F

QUESTIONNAIRE

Dam Safety - Operations Questionnaire

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

Part I - Background Data
Part II - General Site Information
Part III - Hydraulic Discharge and Operating Facilities
  A. Discharge Facilities
  B. Operating Equipment
  C. Operating Problems
Part IV - Public Safety

Throughout the questionnaire, the following definitions apply:

Spillway: A structure over which flood flows are discharged. The discharge is uncontrolled.
Sluice: A structure through which flood flows are discharged and the flow is controlled by gates, stop logs or valves.
Lock: A structure normally used for the passage of vessels, and not for spilling water.
Emergency: Severe flooding or possible dam failure conditions.

Organization: PCA

Watershed: Trent Severn Waterway

Site: **Talbot River Dam and Lock 39**

Prepared by: Holly Hampton (KGS Group) on behalf of Chuck Wilkinson (Manager – PCA Northern Sector

Date: November 28, 2012

Questions	Answers/Observations/Comments

Part I – Background Data

1. Flow Control Equipment Summary

	Type/Size	Number
Sluices –log -	*7.6 m W x 8.0 m H*	*2*
# of logs per sluice –		*10*
Material of logs -	*Douglas Fir*	
Height of logs -	*305 mm (12 in.) H x 356 mm (14 in.)*	
Lock size (l x w x h) -	*43.3 m x 10 m x 6.55 m*	
Lock lining material -	*Concrete*	
Lock gate-	*Stacked-timber mitre gates*	

	Type/Size	**Number**
Lock valves -	*Butterfly Valves*	
Lock gate motors -	*Manual*	
Other -		

2. Source of dimensions datum (CGD or other - specify) *CGD*

3. Are flashboards being used to raise the operating water levels? ☐ Yes ☒ No
   If yes, do they automatically collapse on high water levels? ☐ Yes ☐ No ☒ N/A

Part II - General Site Information

4. (a) Who operates the site? ☒ Owner ☐ Contractor ☐ Other

   (b) Who operates the lock? ☒ Owner ☐ Contractor ☐ Other

   (c) Where is the Operators' Office Located? *During lock operation season Operator's office is on site. Northern Sector office is in Kirkfield.*

   (d) How many staff are normally available to operate Dam and Lock?

   The dam and lock are manned by a crew that varies throughout the year. Schedule is as follows:

   - ***April 1st to May Long Weekend -*** *No Operators are specifically assigned to Talbot River Dam and Lock 39 but there is a pool of 5 Operators that are dedicated to the Northern Sector for controlling the water levels and maintenance activities.*
   - ***May Long Weekend to Thanksgiving –*** *Two Operators are dedicated to the operations Talbot Dam and Lock 38, Talbot River Dam and Lock 39. Students are also available to assist with the lock operations and cutting the lawn. They work 6 days per week and 11 hours per day.*
   - ***Thanksgiving to November 7th –*** *No Operators are specifically assigned to Talbot River Dam and Lock 39 but there is a pool of 5 Operators that are dedicated to the Northern Sector for controlling the water levels and maintenance activities**.***
   - ***November 8th to March 31st -*** *No Operators are specifically assigned to Talbot River Dam and Lock 39 but PCA maintenance staff are available if required.*

   (e) What is the area covered in km^2?

   Kirkfield and Washago offices make up the Northern Sector. Northern Sector covers from Bobcaygeon to Georgian Bay.

   (f) What is the distance from the Operators' office to this site?

During lock operation season, Operator's office is at site, the rest of the year the Operators' office is at Kirkfield. Kirkfield's office is approximately 15 km from the site.

(g) What is the distance from the Operators' office to furthest away site?

Approximately 70 km.

(h) How many dams per week does the operating team have to remove or install logs?

Spring: *~10 dams/ day* Summer: *~5 dams/day* Fall: *~4 dams/3 days* Winter:*~4 dams/3 days*

(i) What hours are the lock staffed and operated?

Spring: *N/A* Summer: *8:30 am to 7:30 pm, 7 days/wk* Fall: *N/A* Winter: *N/A*

(j) How often per week does the operating team have to service locks?

Spring: *N/A* Summer: *Daily* Fall: *N/A* Winter: *N/A*

(k) Is person/team responsible for operating other sites? ☒Yes ☐No

*During **navigation season**, 2 man crew is responsible for Talbot Dam and Lock 38, and Talbot River Dam and Lock 39.*

*For **non-navigation season**, see Question 4(d).*

(l) If answer to (k) is yes, is there sufficient staff to operate these sites simultaneously?

☒Yes ☒No *Typically yes, but very challenging*.

(m) If answer to (l) is no, is other assistance available?

☒Yes ☐No

If yes, from who and from where?

If required, can request assistance from Peterborough Headquarters maintenance Operators.

5. How often do the logs have to be removed or installed at Talbot River Dam?

Summer: ~ 1/ 3 days Fall: Initially 1 / wk for 5 wks Winter: Only for critical/unusual conditions
Spring: Daily

6. How often is the lock operated in one day?

Spring: N/A Summer: Every 20 mins (time to lower and raise) Fall: N/A Winter: N/A

7. How often does Lock 39 have to be serviced?

Spring: *N/A* Summer: *Daily* Fall: *N/A* Winter: *N/A*

NOTE: In the fall Operators identify what needs to be repaired or requires maintenance and then maintenance crew typically carries out the work in the fall.

8. Is an operations log book kept at:

   o Dam? ☒Yes ☐No

   If yes, where is it located and what information is logged?

   o Lock 39? ☒Yes ☐No

   If yes, where is it located and what information is logged?

   One log book is kept at site in the Operator's Building. This log book keeps information for both the lock and dam.

9. Do staff stay at a site during an emergency? ☐Yes ☒No

   Operators would do whatever needs to be done at the site and then move on to carry out other required tasks.

10. How are communications maintained with Owner's Office? *(Headquarters is in Peterborough)*

    (i) at site - *Always communicates with Chuck Wilkinson (Sector Manager) by cell phone.*

    (ii) travelling to/from site – *By cell phone*

11. Most likely means of access under emergency conditions during:

(a) Spring	☒Road	☒Water	☐Rail	☐Air	☒Walk
(b) Summer/Fall	☒Road	☒Water	☐Rail	☐Air	☒Walk
(c) Winter	☒Road	☐Water	☐Rail	☐Air	☒Walk

12. Are problems or restrictions for accessing the site in an emergency situation foreseen?

    ☐Yes ☒No

    If yes, please describe (e.g. Will access road be accessible if there is a major flood?)

    (a) Spring ☐Yes ☐No

    (b) Summer/Fall ☐Yes ☐No

    (c) Winter ☐Yes ☐No

13. How long will it take for staff to access the site under emergency conditions?

(a) Spring ☒Less than 1/2 h ☐1/2 to 2 h ☐2 h to 1/2 d ☐1/2 to 1 d ☐More than 1 d

(b) Summer/Fall ☒Less than 1/2 h ☐1/2 to 2 h ☐2 h to 1/2 d ☐1/2 to 1 d ☐More than 1 d

(c) Winter ☒Less than 1/2 h ☐1/2 to 2 h ☐2 h to 1/2 d ☐1/2 to 1 d ☐More than 1 d

14. In the event of a flood emergency, how is the requirement for log operation decided and what is the communication protocol that causes the log operation to be initiated? Is the protocol the same for non-work hours (evenings, weekends and statutory holidays)?

The Water Control Engineer (Dave Ness) or the Northern Sector Manager (Chuck Wilkinson) initiates what action is required by watching the water levels. Northern Sector Manager is not obligated to check water levels on non-working hours but unofficially he does. Dave Ness, also unofficially checks water levels on non-working hours. For Kirkfield region there are 2 Operators on standby and 2 Operators on standby for Washago region on weekends but no one for nights. If an emergency happened at night the Northern Sector Manager would try to find Operators.

15. How much are the response times affected during non-work hours (evenings, weekends and statutory holidays)?

Approximately 2 hrs: 1 hr to contact someone and another 1 hr to get equipment and get to site.

16. In the event of the need to operate several dams in a very short time frame (a few hours) how are the priorities established?

Dave and Chuck decide which dams are most critical.

17. Once at the site, how long will it take staff to achieve maximum spill capacity (assuming headwater level is at Maximum Operating Level)?

(a) Spring / Summer / Fall ☐Less than 1/2 h ☐1/2 to 1 h ☐1 h to 2 h ☒2 h to 1/2 d ☐1/2 d to 1 d ☐2 d

☐3 d ☐More than 3 d

Typically 2 hrs but problems can happen during all seasons. In winter, it would take longer if a tiger torch is needed to melt the ice at the gains.

18. How many staff members are required to achieve maximum spill capacity for the above time estimate?

Two (2) to four (4) Operators.

19. Are there any emergency procedures in place to deal with a dam accident or extreme flood condition? ☐Yes ☒No

If yes, what is the name of the document? Please provide a copy of the document.

20. On average, how many personnel work at the site year round or seasonal?

On weekdays

On weekends hours

Visits/week

Visits/month

See Question 4(d).

21. What recreational and business activities are in close proximity to the lock, head water, tailrace, and/or downstream areas?

Navigation, fishing, waterskiing, wakeboarding, swimming, canoeing, kayaking, and motor boat.

22. What other parties, outside of PCA, are involved with flow regulation along the river?

None.

Who are the contact persons?

N/A

Part III - Hydraulic Discharge and Operating Facilities

A. Discharge Facilities

23. Have any structural or channel modifications been made since the date on the rating table? (e. g., new stoplogs, dredging, etc.) ☐Yes ☒No

If yes, what type of modifications, and do they affect the rating table?

24. Have any structural, power equipment, gate or channel modifications been made to the lock in the last 10 years? (e.g., new gates, valves, dredging, etc.) ☐Yes ☒No

If yes, what type of modifications?

25. Have all log sluices ever been fully opened? ☐Yes ☒No

If yes, under what headwater elevation and when?

26. If all logs have never been removed from each sluice then what is the maximum number of logs that has been removed from each sluice?

o high tailwater level (specify)

o other conditions (specify)

Approximately 4 years ago at Talbot Dam and Talbot River Dam, seven (7) logs (out of 11 for Talbot Dam and 10 for Talbot River Dam) were removed. This resulted in flooding downstream.

B. Operating Equipment

27. What type of equipment is used to operate the

o Dam ☒crab winch

☐spud winch ☒other -specify

Miscellaneous equipment - tiger torches, peevies, pike poles, tongs, etc.

with:

o Lock 39 ☐Diesel ☐electric
☒hand ☐other -specify

28. What is the condition and age of equipment at the Dam?

Concrete is > 100 years old but there has been re-facing on the top half of Talbot Dam. Dam was constructed in about 1908. Winches are original (i.e., > 100 years old).

29. What is the condition and age of lock?

Built in 1908 – no significant issues.

30. How is the equipment maintained and how often at:

o Dam?

o Lock?

See question #7.

31. Who supplies the lifting equipment (i.e., winch and support frame)?

Supplier no longer exists. Replacement parts need to be custom made and retrofitted.

32. Does PCA have any specs and load ratings for the lifting equipment?

☐Yes ☒No - *Lifting capacity is not known.*

If yes, could you please provide documentation?

33. What are the procedures for removing, installing, and jacking down the logs?

Two to four Operators carry out operations. Refer to TSW Standing Orders for procedures.

34. What are the procedures for operating the lock?

Two Operators operate the lock. The lock is equipped with manual butterfly valves for discharging and filling the gates, and 2 gates at u/s end and 2 gates at d/s end are manually opened and closed.

35. What are the procedures for maintaining the lock?

Oil, greasing, painting, tighten cables on gate sweeps, maintain valves, etc.. Most maintenance is carried out in off season.

36. What special measures are taken during winter and night operations?

Use flashlights and/or use generator for lights.

37. Is auxiliary power available? ☐Yes ☒No
If yes, specify source.

38. Is the discharge facility operating equipment located at the site? ☒Yes ☐No
If no, where is it located?

39. Have any backup provisions been made should the equipment fail? ☒Yes ☐No
If yes, what are the provisions?
Would borrow equipment from another site.

If yes, is the backup located on site? ☐Yes ☒No

If no, where is backup located? *Next site, Lock 37 or 38.*

40. If backup located off-site, how much more time is required to achieve maximum discharge?
Two (2) additional hours (4 hours in total).

41. Has mechanical equipment ever failed for:
o Dam ☒Yes ☐No

o Lock 39 ☒Yes ☐No

If yes, when did failure occur? *Yearly.*

What was the nature and extent of failure?
Lock gate cables for opening the gate have failed. Takes approximately 3 hours to replace if a spare is at the shop, otherwise 1 week to get cable. Valves rarely fail, but when they do, it is the pins/arms. Crab winches gears break but mostly because of vandalism. PCA keep spare parts but not enough.

42. How is the water level monitored?

Automatic level gauge is at site and readings are recorded hourly and sent to Peterborough. Readings are also e-mailed daily to Dave Ness and Chuck Wilkinson.

43. How many spare logs are on site for each sluice?

None.

44. How many spare logs are stored off site for each sluice and where is the storage area?

Five (5) logs are stored at the Kirkfield office for the entire Kirkfield region.

45. What is the age and condition of the logs and how often are they replaced?

Replaced as required, varies from 5 yrs to 30 yrs. Chuck Wilkinson believes that Talbot River Dam needs more new logs.

46. What is the age and condition of the lock gates, seals, valves, hinges and operating equipment, how often are they replaced?

Valves, seals, valve hinges for the u/s end have been refurbished 10 years ago and for the d/s end were refurbished 30 years ago. U/S gates are 10 years old and d/s gates are 30 years old.

47. Where is the water level signal sent and by what path?

Is that location staffed 24/7? ☐Yes ☒No

If not, how is a response initiated?

Automatic readings sent to the Peterborough office.

48. How frequently is the water level gauge calibrated and by who?

Dave Ness calibrates the water level gauge approximately every 2 years but the gauge is monitored regularly for any issues.

C. Operating Problems

49. Are there problems that may reduce the number of stoplogs which can be removed during normal or flood conditions? ☒Yes ☐No

If yes, please describe.

D's can come out, logs can roll, logs can break and/or jam, winch gears can jump/fail and then free wheel.

50. What actions are taken if a log gets jammed?
If necessary, Operators cut log and let it go d/s and then collect log d/s.

51. Have any lock operation problems been experienced with gates, seals, valves, hinges or operating equipment? ☒Yes ☐No

If yes, please describe.
Maintenance issues that cause operating interruptions from 1 hr to 1 week long.

52. Is it possible for a log to turn 90 degrees in the gains? ☒Yes ☐No

53. Has debris blockage ever occurred at this site? ☒Yes ☐No
Debris blockage has disrupted normal logging operations (i.e., hooking logs), but no significant amount of debris to block the sluices. Debris is not an issue for this site.

54. If yes, when did blockage occur?
Yearly.

55. What was the nature & extent of blockage?
See Question # 53.

56. Is there potential for debris from upstream to interfere with operations at the site under:

(a) Normal Operation? ☒Yes ☐No
(b) Flood/Emergency Operation? ☒Yes ☐No

If answer to (a) or (b) is yes, please describe
See Question # 53.

57. Is there a debris management program in place (e.g. debris boom, regular removal of debris, etc.)? ☐Yes ☒No

If yes, briefly describe program.

58. Do ice jams occur on this river system? ☐Yes ☒No

(a) Do ice jams affect this site? ☐Yes ☒No

(b) Are there special operations to accommodate ice jam inflows? ☐Yes ☒No

(c) Do ice jams block/hinder discharge facilities? ☐Yes ☒No

If answer to (a), (b) or (c) is yes, please describe

59. Has ice sheet formation been observed:

(a) In the head pond or reservoir area? ☒Yes ☐No

(b) Against log sluices? ☒Yes ☐No

(c) Against lock gates? ☒Yes ☐No

(d) Against gravity wall/bulkhead? ☒Yes ☐No

Ice formation is minimal, only approximately 6 inches thick, but enough to affect the logging operations. Operators use tiger torches for winter logging operations when required.

60. Has frazil ice been observed:

(a) In the headpond or reservoir area? ☒Yes ☐No

(b) Against log sluices? ☒Yes ☐No

(c) Against lock gates? ☒Yes ☐No

(d) Against gravity wall/bulkhead? ☒Yes ☐No

Frazil ice has been observed but not an issue for this site.

61. Are there any measurements or other estimates of the ice thickness? ☐Yes ☒No

If yes, please provide

62. What is the duration of head pond/reservoir ice cover (months)?

__*December*__ to *April*__.

63. Is the frozen head pond generally covered with snow? ☐Yes ☐No ☒Maybe

64. Are any photographs of the head pond ice conditions available? ☐Yes ☒No

If so, where are they located and when were they taken?

65. Any other observations regarding ice cover? ☐Yes ☒No

Part IV – Public safety

66. In general, are there any public safety issues? ☒Yes ☐No

If yes, please describe?

Access to dam should be closed off. No kick plates for guardrails and Operators do work below in sluices for jacking down the logs.

67. Does the public access the dam decks and retaining walls and cross the lock on the top of the gates? ☒Yes ☐No

If yes, how often and how many people?

Regularly. Number of people not known.

68. Does the public come in close proximity to the sluices (from upstream and/or downstream)?

☒Yes ☐No

If yes, please describe?

69. Is there any security fencing at the dam to restrict access? ☐Yes ☒No

If yes, where?

70. Is there any security fencing at the lock to restrict access during non-operating periods?

☒Yes ☐No

If yes, where?

There is a gate across the road that leads to the lock.

71. Does the public come in close proximity to the shorelines near the dams? If yes, please describe.

☒Yes ☐No

72. Does anyone fish near the dam? ☒Yes ☐No

If yes, please describe.

Both u/s and d/s.

73. Is vandalism by the public an issue? ☒Yes ☐No

If yes, please describe.

There is vandalism but not major.

74. Are or have safety booms been deployed? ☐Yes ☒No

If yes:

Where and during what months of the year?

Instead of safety booms a double cable system is installed across the u/s face of the dam.

Have there been any issues with them? ☐Yes ☒No

If yes, please describe

75. Have there been any public safety incidents in the past 10 years, including use of the locks?

☐Yes ☒No

If yes, when and please describe.

No, but at Bolsover Dam (Lock 37), 2 years ago a drunk person dove off of the dam.

76. Have there been any employee or contractor safety incidents in the past 10 years?

☐Yes ☐No ☒Minor

If yes, when and please describe.

77. Are there any recent changes or foreseeable changes coming which may affect dam / lock operations? ☐Yes ☒No

If yes, please describe (this could include public use of the dam or nearby waterway, operating equipment deterioration, organizational change, change in the use of the dam, increased number of dams to be operated, deterioration of access roads, lengthening of navigation season, etc.).

APPENDIX G

INUNDATION MAPS - SUNNY DAY

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649000 649500 650000

4930000

4929500

N W E S

Talbot River Near End of Canal Rd.		
Distance from Talbot River Dam	0.1 km	
	Breach of Dam	No Breach
Peak Water Level (m)	228.6	225.4
Time to Peak Water Level (h)	0.3	
Peak Flow (m^3/s)	420	2
Peak Velocity (m/s)	2.2	0.3
Wave Arrival Time (h)	0.0	NA

Talbot River Upstream of Talbot River Dam		
Distance from Talbot River Dam	0 km	
	Breach of Dam	No Breach
Peak Water Level (m)	230.5	230.5
Time to Peak Water Level (h)		
Peak Flow (m^3/s)	460	2
Peak Velocity (m/s)	0.0	0.0
Wave Arrival Time (h)	NA	NA

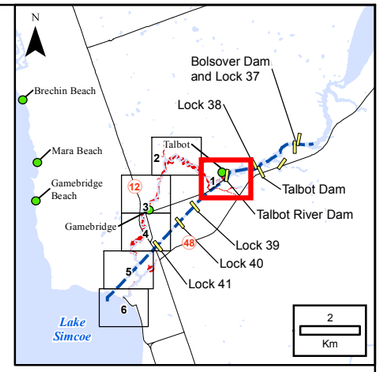
Talbot River Downstream of Talbot River Dam		
Distance from Talbot River Dam	0.9 km	
	Breach of Dam	No Breach
Peak Water Level (m)	227.8	225.0
Time to Peak Water Level (h)		
Peak Flow (m^3/s)	300	2
Peak Velocity (m/s)	2.2	0.5
Wave Arrival Time (h)	0.1	NA

SIDE RD 5
CANAL RD
CON RD A
REG RD 50
CON RD 11
THORAH SIDERD
HWY 48
SHIER AVE
BALL AVE W
BALL ST

9864
9713
8937
5850
5379
Culvert
Talbot River Dam

Talbot Canal Near Culvert Upstream of Lock 39		
Distance from Talbot Inlet Cannal	0.6 km	
	Breach of Dam	No Breach
Peak Water Level (m)	230.4	230.4
Time to Peak Water Level (h)		
Peak Flow (m^3/s)	0	0
Peak Velocity (m/s)	0.1	0.0
Wave Arrival Time (h)	NA	NA

Talbot Canal Near Canal Inlet		
Distance from Talbot Inlet Cannal	0.2 km	
	Breach of Dam	No Breach
Peak Water Level (m)	230.4	230.4
Time to Peak Water Level (h)		
Peak Flow (m^3/s)	0	0
Peak Velocity (m/s)	0.1	0.0
Wave Arrival Time (h)	NA	NA



LEGEND:

- Flooded Area Sunny Day Breach of Talbot River Dam
- Normal Lake or River Level
- Cross Section
- 2m Index Contour TIN
- 0.5m Contour TIN
- Dam
- Highway
- Major Road
- Local Street
- Flow Direction
- Building

Notes:

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50 0 50 100 150 200
Metres

SCALE: 1:5,000 METRIC 11"x17"

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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

SUNNY-DAY BREACH OF TALBOT RIVER DAM SHEET 1 OF 6

APRIL 2015 | FIGURE 01.01 | REV: 0

FileName: P:\Projects\2012\12-0006-028\Dwg\GIS\MXDs\Rev0\SDF_TRD\12-0006-028_Fig01_Rev0.mxd
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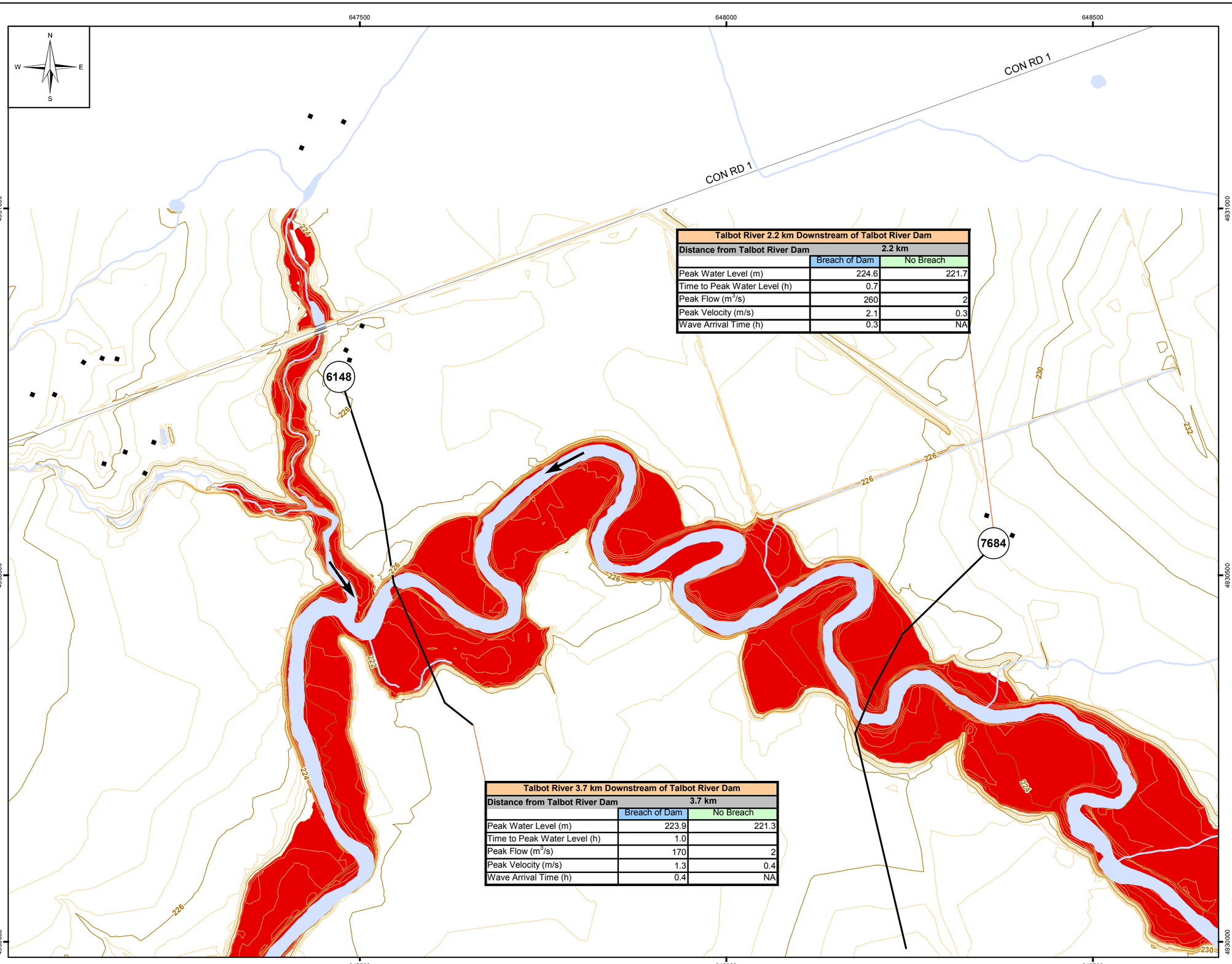
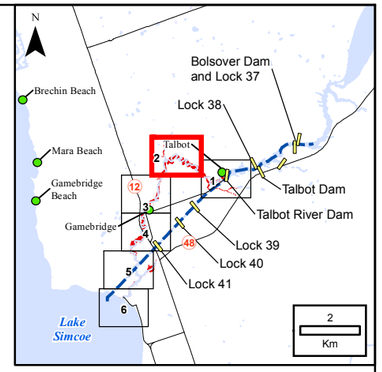


Talbot River 2.2 km Downstream of Talbot River Dam		
Distance from Talbot River Dam	2.2 km	
	Breach of Dam	No Breach
Peak Water Level (m)	224.6	221.7
Time to Peak Water Level (h)	0.7	
Peak Flow (m^3/s)	260	2
Peak Velocity (m/s)	2.1	0.3
Wave Arrival Time (h)	0.3	NA

Talbot River 3.7 km Downstream of Talbot River Dam		
Distance from Talbot River Dam	3.7 km	
	Breach of Dam	No Breach
Peak Water Level (m)	223.9	221.3
Time to Peak Water Level (h)	1.0	
Peak Flow (m^3/s)	170	2
Peak Velocity (m/s)	1.3	0.4
Wave Arrival Time (h)	0.4	NA



LEGEND:

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- Normal Lake or River Level
- Cross Section
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Metres

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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

SUNNY-DAY BREACH OF TALBOT RIVER DAM
SHEET 2 OF 6

APRIL 2015 | FIGURE 01.02 | REV: 0

FileName: P:\Projects\2012\12-0006-028\Dwg\GIS\MXDs\Rev0\SDF_TRD\12-0006-028_Fig01_Rev0.mxd
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646000 646500 647000 647500

4929500 4929000

N W E S

232 230 228 226 224 222 220

HWY 12

CON RD A

ETHEL DR

NEWTON AVE

HAZEL ST

RAMARA RD 51

REG RD 50

4723 3651 4008

Talbot River at Gamebridge Near Hazel St.

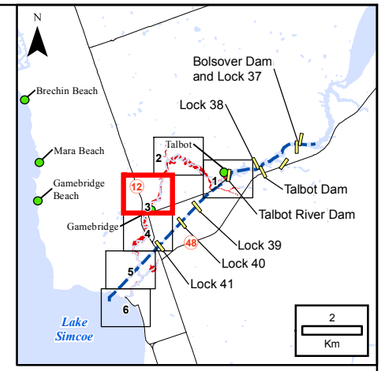
Distance from Talbot River Dam	5.8 km	
	Breach of Dam	No Breach
Peak Water Level (m)	222.5	220.0
Time to Peak Water Level (h)	1.5	
Peak Flow (m^3/s)	130	2
Peak Velocity (m/s)	1.4	0.2
Wave Arrival Time (h)	0.7	NA

Talbot River 5.1 km Downstream of Talbot River Dam

Distance from Talbot River Dam	5.1 km	
	Breach of Dam	No Breach
Peak Water Level (m)	222.7	220.1
Time to Peak Water Level (h)	1.4	
Peak Flow (m^3/s)	140	2
Peak Velocity (m/s)	1.3	0.2
Wave Arrival Time (h)	0.6	NA

Gamebridge - Upstream of Ramara Road 51 Bridge

Distance from Talbot River Dam	6.2 km	
	Breach of Dam	No Breach
Peak Water Level (m)	222.2	219.9
Time to Peak Water Level (h)	1.5	
Peak Flow (m^3/s)	120	2
Peak Velocity (m/s)	1.5	0.2
Wave Arrival Time (h)	0.8	NA



LEGEND:

- Flooded Area Sunny Day Breach of Talbot River Dam
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50 0 50 100 150 200
Metres

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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

SUNNY-DAY BREACH OF TALBOT RIVER DAM
SHEET 3 OF 6

APRIL 2015 | FIGURE 01.03 | REV: 0

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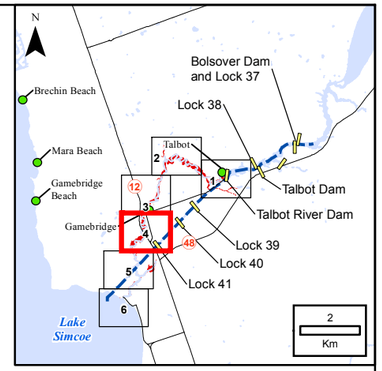
Talbot River at Gamebridge Near Talbot Rd.		
Distance from Talbot River Dam	**6.5 km**	
	Breach of Dam	No Breach
Peak Water Level (m)	221.9	219.8
Time to Peak Water Level (h)	1.5	
Peak Flow (m^3/s)	120	2
Peak Velocity (m/s)	1.9	0.4
Wave Arrival Time (h)	0.8	NA

Talbot River Upstream of Bridge at Hwy 12		
Distance from Talbot River Dam	**7.4 km**	
	Breach of Dam	No Breach
Peak Water Level (m)	221.1	219.1
Time to Peak Water Level (h)	1.8	
Peak Flow (m^3/s)	120	2
Peak Velocity (m/s)	1.5	0.2
Wave Arrival Time (h)	1.0	NA



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Metres

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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

SUNNY-DAY BREACH OF TALBOT RIVER DAM
SHEET 4 OF 6

APRIL 2015 | FIGURE 01.04 | REV: 0

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645500 646000 646500 647000

4927000

4926500

CON RD B

FURNISS DR

1010

1090

985

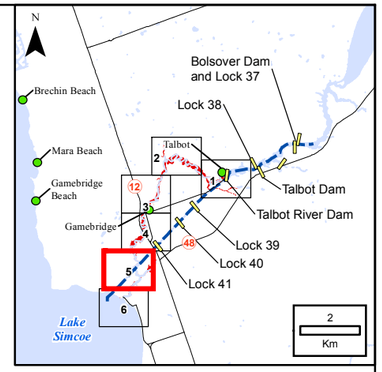
652

Talbot River Near Furniss Dr.		
Distance from Talbot River Dam	8.8 km	
	Breach of Dam	No Breach
Peak Water Level (m)	220.4	219.0
Time to Peak Water Level (h)	2.4	
Peak Flow (m^3/s)	100	2
Peak Velocity (m/s)	1.1	0.1
Wave Arrival Time (h)	1.1	NA

Talbot Canal Upstream of Railway Bridge		
Distance from Talbot River Dam	10.2 km	
	Breach of Dam	No Breach
Peak Water Level (m)	219.1	219.0
Time to Peak Water Level (h)	1.9	
Peak Flow (m^3/s)	60	1
Peak Velocity (m/s)	0.6	0.0
Wave Arrival Time (h)	1.2	NA

Talbot River Upstream of Railway Bridge		
Distance from Talbot River Dam	10.7 km	
	Breach of Dam	No Breach
Peak Water Level (m)	219.1	219.0
Time to Peak Water Level (h)	2.4	
Peak Flow (m^3/s)	40	1
Peak Velocity (m/s)	0.4	0.0
Wave Arrival Time (h)	1.2	NA

Talbot River Near Trent Talbot Marina		
Distance from Talbot River Dam	10.4 km	
	Breach of Dam	No Breach
Peak Water Level (m)	219.1	219.0
Time to Peak Water Level (h)	2.4	
Peak Flow (m^3/s)	40	1
Peak Velocity (m/s)	0.4	0.0
Wave Arrival Time (h)	1.2	NA



LEGEND:

- Flooded Area Sunny Day Breach of Talbot River Dam
- Normal Lake or River Level
- Cross Section
- 2m Index Contour TIN
- 0.5m Contour TIN
- Dam
- Highway
- Major Road
- Local Street
- Flow Direction
- Building

Notes:

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3. NA indicates "Not Applicable".
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6. All units are metric and in metres unless otherwise specified. Transverse Mercator Projection, NAD 1983, Zone 17. Elevations are in metres above sea level (MSL).

50 0 50 100 150 200
Metres

SCALE: 1:5,000 METRIC 11"x17"

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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

SUNNY-DAY BREACH OF TALBOT RIVER DAM SHEET 5 OF 6

APRIL 2015 | FIGURE 01.05 | REV: 0

FileName: P:\Projects\2012\12-0006-028\Dwg\GIS\MXDs\Rev0\SDF_TRD\12-0006-028_Fig01_Rev0.mxd
11"x17" PLOT SCALE 1:1

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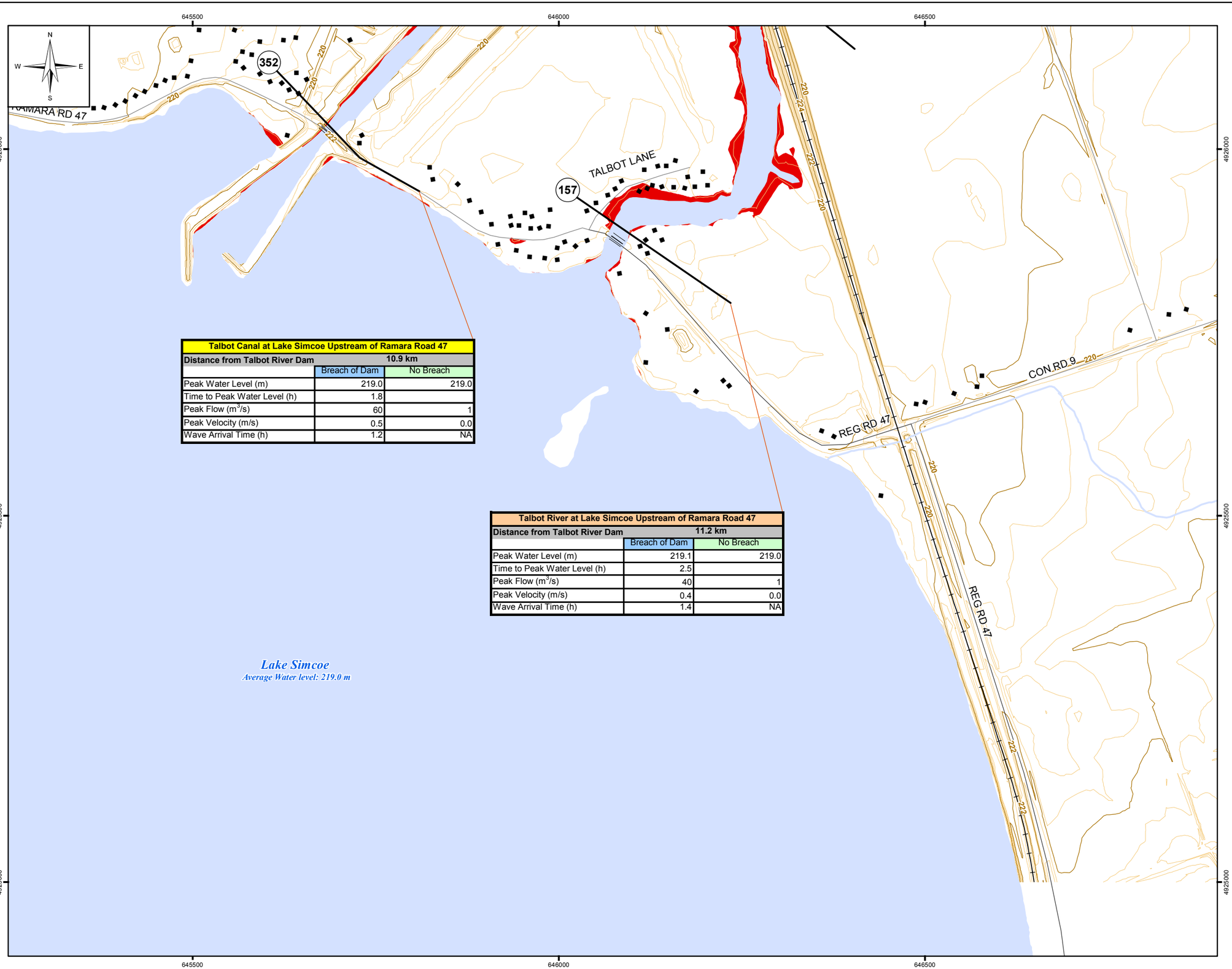
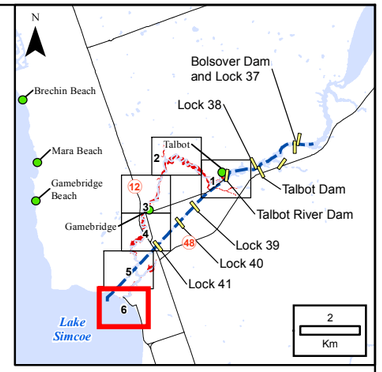


Talbot Canal at Lake Simcoe Upstream of Ramara Road 47		
Distance from Talbot River Dam	10.9 km	
	Breach of Dam	No Breach
Peak Water Level (m)	219.0	219.0
Time to Peak Water Level (h)	1.8	
Peak Flow (m^3/s)	60	1
Peak Velocity (m/s)	0.5	0.0
Wave Arrival Time (h)	1.2	NA

Talbot River at Lake Simcoe Upstream of Ramara Road 47		
Distance from Talbot River Dam	11.2 km	
	Breach of Dam	No Breach
Peak Water Level (m)	219.1	219.0
Time to Peak Water Level (h)	2.5	
Peak Flow (m^3/s)	40	1
Peak Velocity (m/s)	0.4	0.0
Wave Arrival Time (h)	1.4	NA



LEGEND:

- Flooded Area Sunny Day Breach of Talbot River Dam
- Normal Lake or River Level
- Cross Section
- 2m Index Contour TIN
- 0.5m Contour TIN
- Dam
- Highway
- Major Road
- Local Street
- Flow Direction
- Building

Notes:

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50 0 50 100 150 200
Metres

SCALE: 1:5,000 METRIC 11"x17"

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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

SUNNY-DAY BREACH OF TALBOT RIVER DAM
SHEET 6 OF 6

APRIL 2015 | FIGURE 01.06 | REV: 0

FileName: P:\Projects\2012\12-0006-028\Dwg\GIS\MXDs\Rev0\SDF_TRD\12-0006-028_Fig01_Rev0.mxd
11"x17" PLOT SCALE 1:1

APPENDIX H

INUNDATION MAPS – IDF

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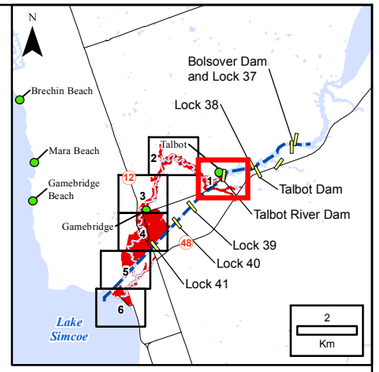
Talbot River Near End of Canal Rd.		
Distance from Talbot River Dam	0.1 km	
	Breach of Dam	No Breach
Peak Water Level (m)	229.2	228.1
Time to Peak Water Level (h)	0.9	
Peak Flow (m^3/s)	510	220
Peak Velocity (m/s)	2.4	1.8
Wave Arrival Time (h)	0.0	NA

Talbot River Upstream of Talbot River Dam		
Distance from Talbot River Dam	0 km	
	Breach of Dam	No Breach
Peak Water Level (m)	231.2	231.2
Time to Peak Water Level (h)		
Peak Flow (m^3/s)	520	220
Peak Velocity (m/s)	0.2	0.2
Wave Arrival Time (h)	NA	NA

Talbot River Downstream of Talbot River Dam		
Distance from Talbot River Dam	0.9 km	
	Breach of Dam	No Breach
Peak Water Level (m)	228.4	227.4
Time to Peak Water Level (h)	0.9	
Peak Flow (m^3/s)	510	210
Peak Velocity (m/s)	2.0	2.1
Wave Arrival Time (h)	0.2	NA

Talbot Canal Near Culvert Upstream of Lock 39		
Distance from Talbot Inlet Cannal	0.6 km	
	Breach of Dam	No Breach
Peak Water Level (m)	231.2	231.2
Time to Peak Water Level (h)		
Peak Flow (m^3/s)	10	10
Peak Velocity (m/s)	0.1	0.1
Wave Arrival Time (h)	NA	NA

Talbot Canal Near Canal Inlet		
Distance from Talbot Inlet Cannal	0.2 km	
	Breach of Dam	No Breach
Peak Water Level (m)	231.2	231.2
Time to Peak Water Level (h)		
Peak Flow (m^3/s)	10	10
Peak Velocity (m/s)	0.1	0.1
Wave Arrival Time (h)	NA	NA



LEGEND:

- Flooded Area IDF No Breach of Talbot River Dam
- Flooded Area Breach of Talbot River Dam during the IDF
- Normal Lake or River Level
- Cross Section
- 2m Index Contour TIN
- 0.5m Contour TIN
- Dam
- Highway
- Major Road
- Local Street
- Flow Direction
- Building

Notes:

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7. The IDF obtained for Talbot River Dam was the flood 37% of the way between the 1000 years flood and the PMF.

50 0 50 100 150 200
Metres

SCALE: 1:5,000 METRIC 11"x17"

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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

BREACH OF THE TALBOT RIVER DAM DURING THE INFLOW DESIGN FLOOD (IDF) SHEET 1 OF 6

APRIL 2015 | FIGURE 01.01 | REV: 0

FileName: P:\Projects\2012\12-0006-028\Dwg\GIS\MXDs\Rev0\DF_TRD\12-0006-028_Fig01_Rev0.mxd
11"x17" PLOT SCALE 1:1

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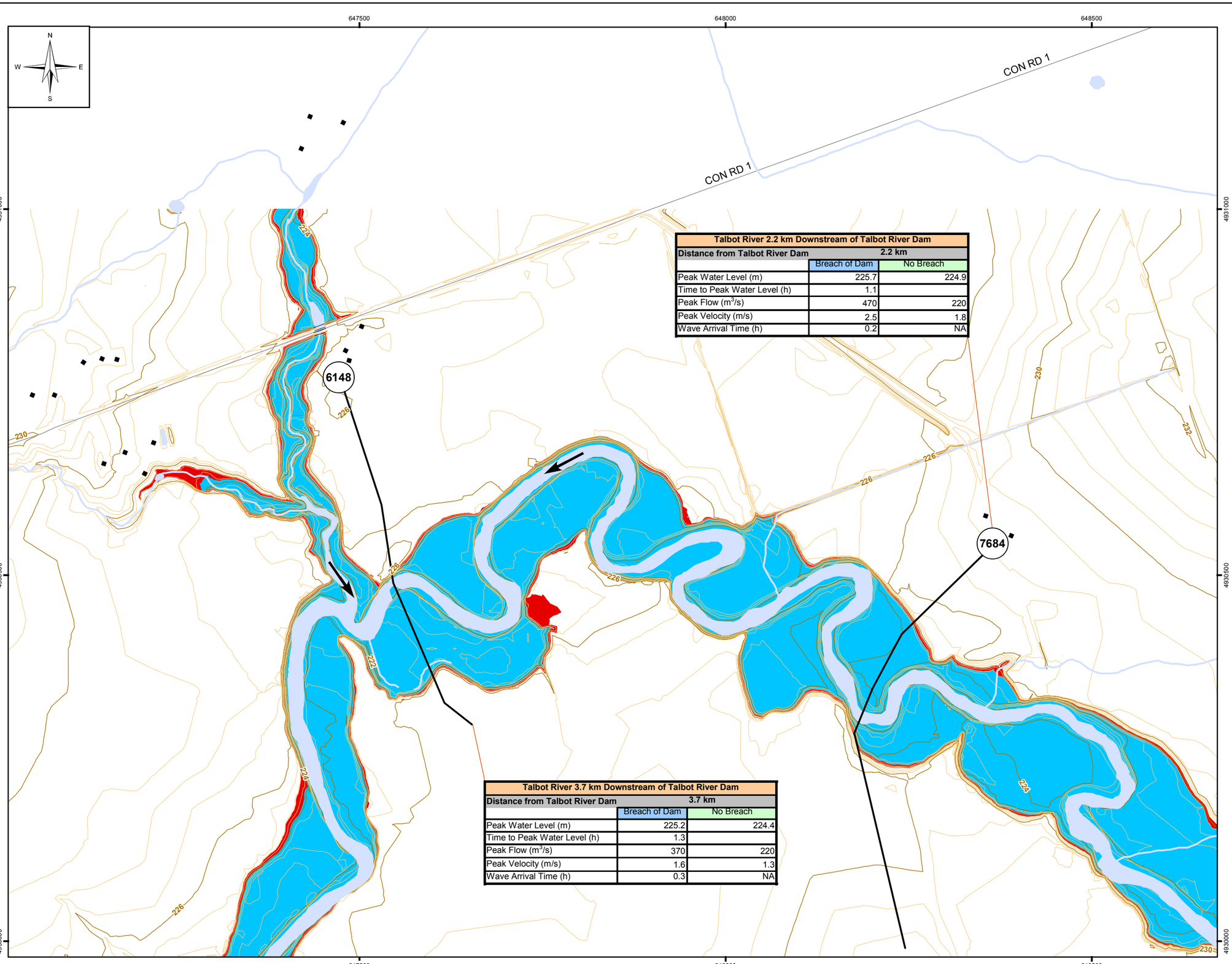
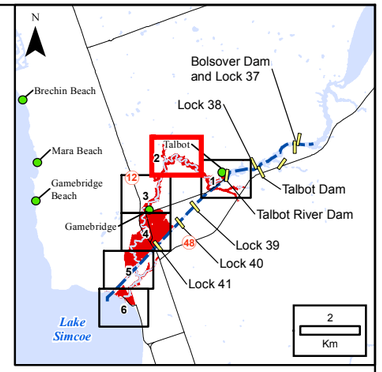


Talbot River 2.2 km Downstream of Talbot River Dam		
Distance from Talbot River Dam	2.2 km	
	Breach of Dam	No Breach
Peak Water Level (m)	225.7	224.9
Time to Peak Water Level (h)	1.1	
Peak Flow (m^3/s)	470	220
Peak Velocity (m/s)	2.5	1.8
Wave Arrival Time (h)	0.2	NA

Talbot River 3.7 km Downstream of Talbot River Dam		
Distance from Talbot River Dam	3.7 km	
	Breach of Dam	No Breach
Peak Water Level (m)	225.2	224.4
Time to Peak Water Level (h)	1.3	
Peak Flow (m^3/s)	370	220
Peak Velocity (m/s)	1.6	1.3
Wave Arrival Time (h)	0.3	NA



LEGEND:

- Flooded Area IDF No Breach of Talbot River Dam
- Flooded Area Breach of Talbot River Dam during the IDF
- Normal Lake or River Level
- Cross Section
- 2m Index Contour TIN
- 0.5m Contour TIN
- Dam
- Highway
- Major Road
- Local Street
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- Building

Notes:

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7. The IDF obtained for Talbot River Dam was the flood 37% of the way between the 1000 years flood and the PMF.

50 0 50 100 150 200

Metres

SCALE: 1:5,000 METRIC 11"x17"

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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

BREACH OF THE TALBOT RIVER DAM DURING THE INFLOW DESIGN FLOOD (IDF) SHEET 2 OF 6

APRIL 2015 | FIGURE 01.02 | REV: 0

FileName: P:\Projects\2012\12-0006-028\Dwg\GIS\MXDs\Rev0\DF_TRD\12-0006-028_Fig01_Rev0.mxd
11"x17" PLOT SCALE 1:1

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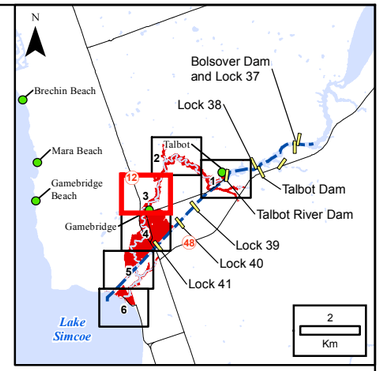
Talbot River at Gamebridge Near Hazel St.		
Distance from Talbot River Dam	5.8 km	
	Breach of Dam	No Breach
Peak Water Level (m)	223.4	223.0
Time to Peak Water Level (h)	1.6	
Peak Flow (m^3/s)	340	220
Peak Velocity (m/s)	2.4	1.9
Wave Arrival Time (h)	0.4	NA

Gamebridge - Upstream of Ramara Road 51 Bridge		
Distance from Talbot River Dam	6.2 km	
	Breach of Dam	No Breach
Peak Water Level (m)	223.2	222.8
Time to Peak Water Level (h)	1.6	
Peak Flow (m^3/s)	330	220
Peak Velocity (m/s)	2.2	1.9
Wave Arrival Time (h)	0.5	NA

Talbot River 5.1 km Downstream of Talbot River Dam		
Distance from Talbot River Dam	5.1 km	
	Breach of Dam	No Breach
Peak Water Level (m)	223.9	223.4
Time to Peak Water Level (h)	1.5	
Peak Flow (m^3/s)	340	220
Peak Velocity (m/s)	2.2	1.7
Wave Arrival Time (h)	0.3	NA



LEGEND:

- Flooded Area IDF No Breach of Talbot River Dam
- Flooded Area Breach of Talbot River Dam during the IDF
- Normal Lake or River Level
- Cross Section
- 2m Index Contour TIN
- 0.5m Contour TIN
- Dam
- Highway
- Major Road
- Local Street
- Flow Direction
- Building

Notes:

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7. The IDF obtained for Talbot River Dam was the flood 37% of the way between the 1000 years flood and the PMF.

50 0 50 100 150 200
Metres

SCALE: 1:5,000 METRIC 11"x17"

NO.	YY/MM/DD	DESCRIPTION	BY
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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

BREACH OF THE TALBOT RIVER DAM DURING THE INFLOW DESIGN FLOOD (IDF) SHEET 3 OF 6

APRIL 2015 | FIGURE 01.03 | REV: 0

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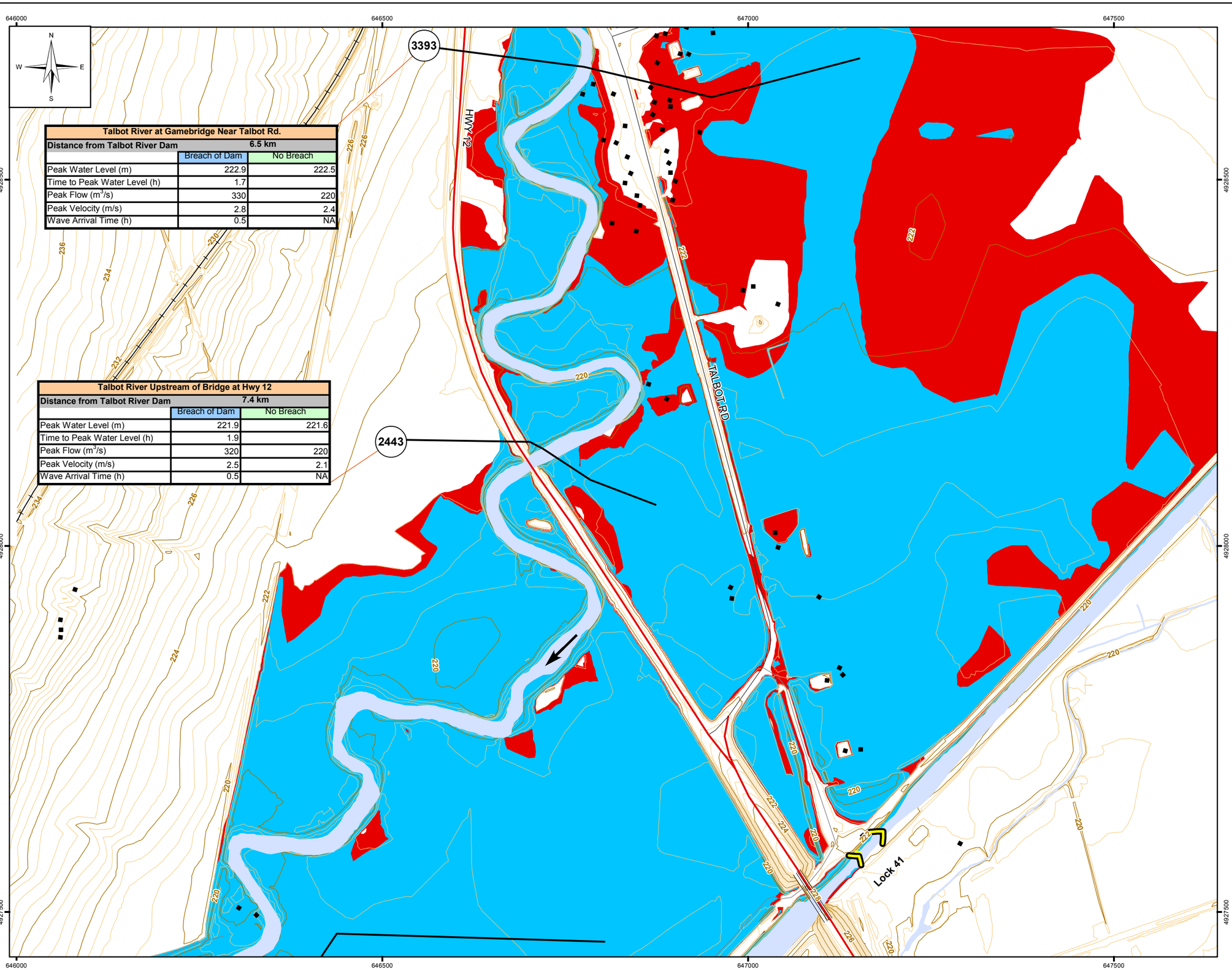
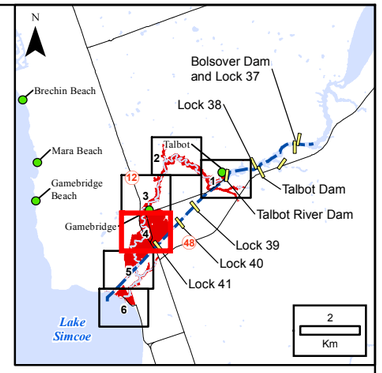


Talbot River at Gamebridge Near Talbot Rd.		
Distance from Talbot River Dam	**6.5 km**	
	Breach of Dam	No Breach
Peak Water Level (m)	222.9	222.5
Time to Peak Water Level (h)	1.7	
Peak Flow (m^3/s)	330	220
Peak Velocity (m/s)	2.8	2.4
Wave Arrival Time (h)	0.5	NA

Talbot River Upstream of Bridge at Hwy 12		
Distance from Talbot River Dam	**7.4 km**	
	Breach of Dam	No Breach
Peak Water Level (m)	221.9	221.6
Time to Peak Water Level (h)	1.9	
Peak Flow (m^3/s)	320	220
Peak Velocity (m/s)	2.5	2.1
Wave Arrival Time (h)	0.5	NA



LEGEND:

- Flooded Area IDF No Breach of Talbot River Dam
- Flooded Area Breach of Talbot River Dam during the IDF
- Normal Lake or River Level
- Cross Section
- 2m Index Contour TIN
- 0.5m Contour TIN
- Dam
- Highway
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7. The IDF obtained for Talbot River Dam was the flood 37% of the way between the 1000 years flood and the PMF.

50 0 50 100 150 200

Metres

SCALE: 1:5,000 METRIC 11"x17"

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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

BREACH OF THE TALBOT RIVER DAM DURING THE INFLOW DESIGN FLOOD (IDF) SHEET 4 OF 6

APRIL 2015 | FIGURE 01.04 | REV: 0

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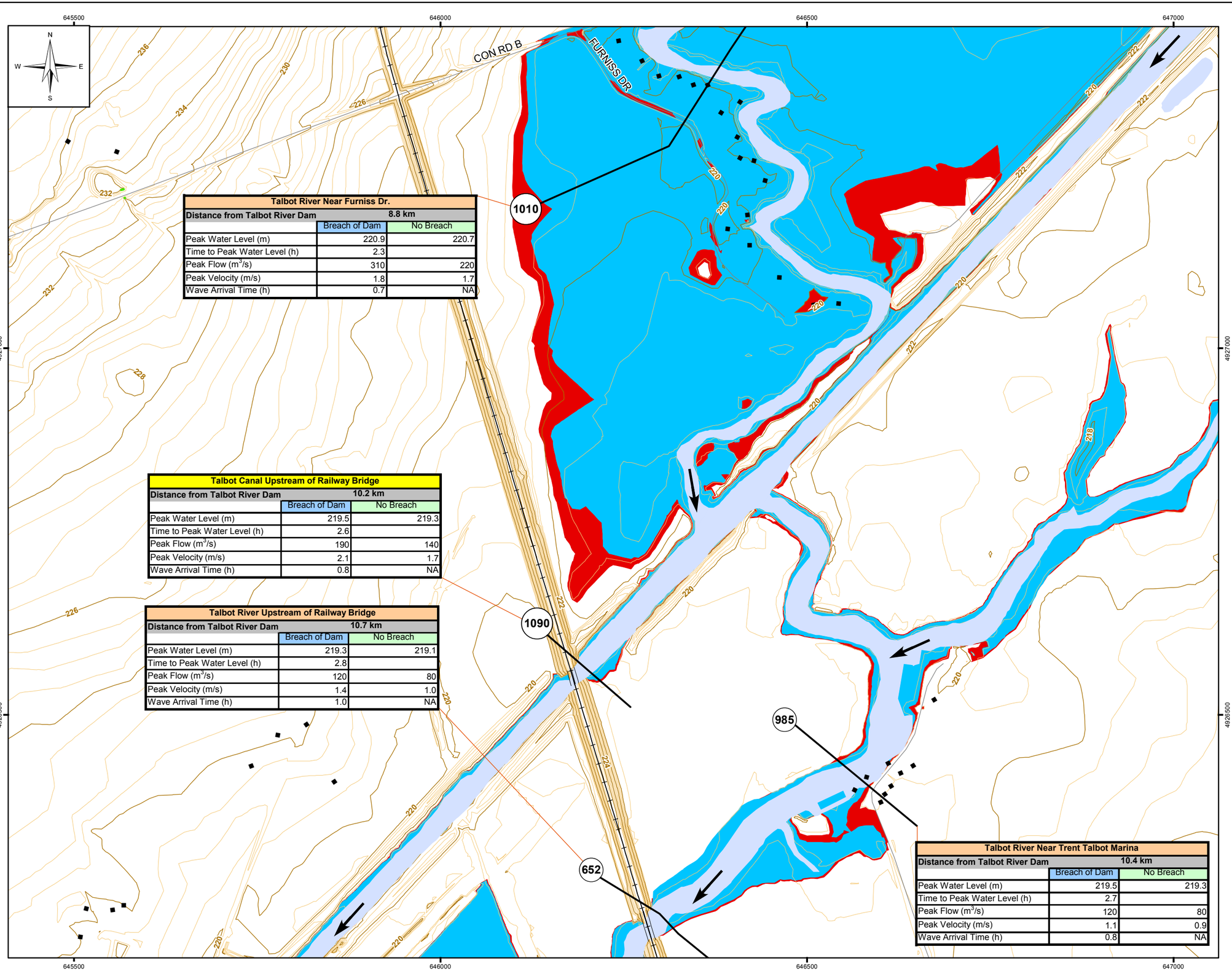
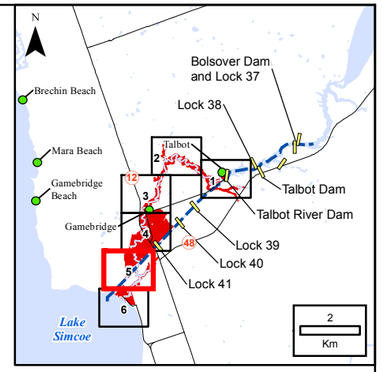


Talbot River Near Furniss Dr.		
Distance from Talbot River Dam	**8.8 km**	
	Breach of Dam	No Breach
Peak Water Level (m)	220.9	220.7
Time to Peak Water Level (h)	2.3	
Peak Flow (m^3/s)	310	220
Peak Velocity (m/s)	1.8	1.7
Wave Arrival Time (h)	0.7	NA

Talbot Canal Upstream of Railway Bridge		
Distance from Talbot River Dam	**10.2 km**	
	Breach of Dam	No Breach
Peak Water Level (m)	219.5	219.3
Time to Peak Water Level (h)	2.6	
Peak Flow (m^3/s)	190	140
Peak Velocity (m/s)	2.1	1.7
Wave Arrival Time (h)	0.8	NA

Talbot River Upstream of Railway Bridge		
Distance from Talbot River Dam	**10.7 km**	
	Breach of Dam	No Breach
Peak Water Level (m)	219.3	219.1
Time to Peak Water Level (h)	2.8	
Peak Flow (m^3/s)	120	80
Peak Velocity (m/s)	1.4	1.0
Wave Arrival Time (h)	1.0	NA

Talbot River Near Trent Talbot Marina		
Distance from Talbot River Dam	**10.4 km**	
	Breach of Dam	No Breach
Peak Water Level (m)	219.5	219.3
Time to Peak Water Level (h)	2.7	
Peak Flow (m^3/s)	120	80
Peak Velocity (m/s)	1.1	0.9
Wave Arrival Time (h)	0.8	NA



LEGEND:

- Flooded Area IDF No Breach of Talbot River Dam
- Flooded Area Breach of Talbot River Dam during the IDF
- Normal Lake or River Level
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- 2m Index Contour TIN
- 0.5m Contour TIN
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50 0 50 100 150 200
Metres

SCALE: 1:5,000 METRIC 11"x17"

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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

BREACH OF THE TALBOT RIVER DAM DURING THE INFLOW DESIGN FLOOD (IDF) SHEET 5 OF 6

APRIL 2015 | FIGURE 01.05 | REV: 0

FileName: P:\Projects\2012\12-0006-028\Dwg\GIS\MXDs\Rev0\IDF_TRD\12-0006-028_Fig01_Rev0.mxd
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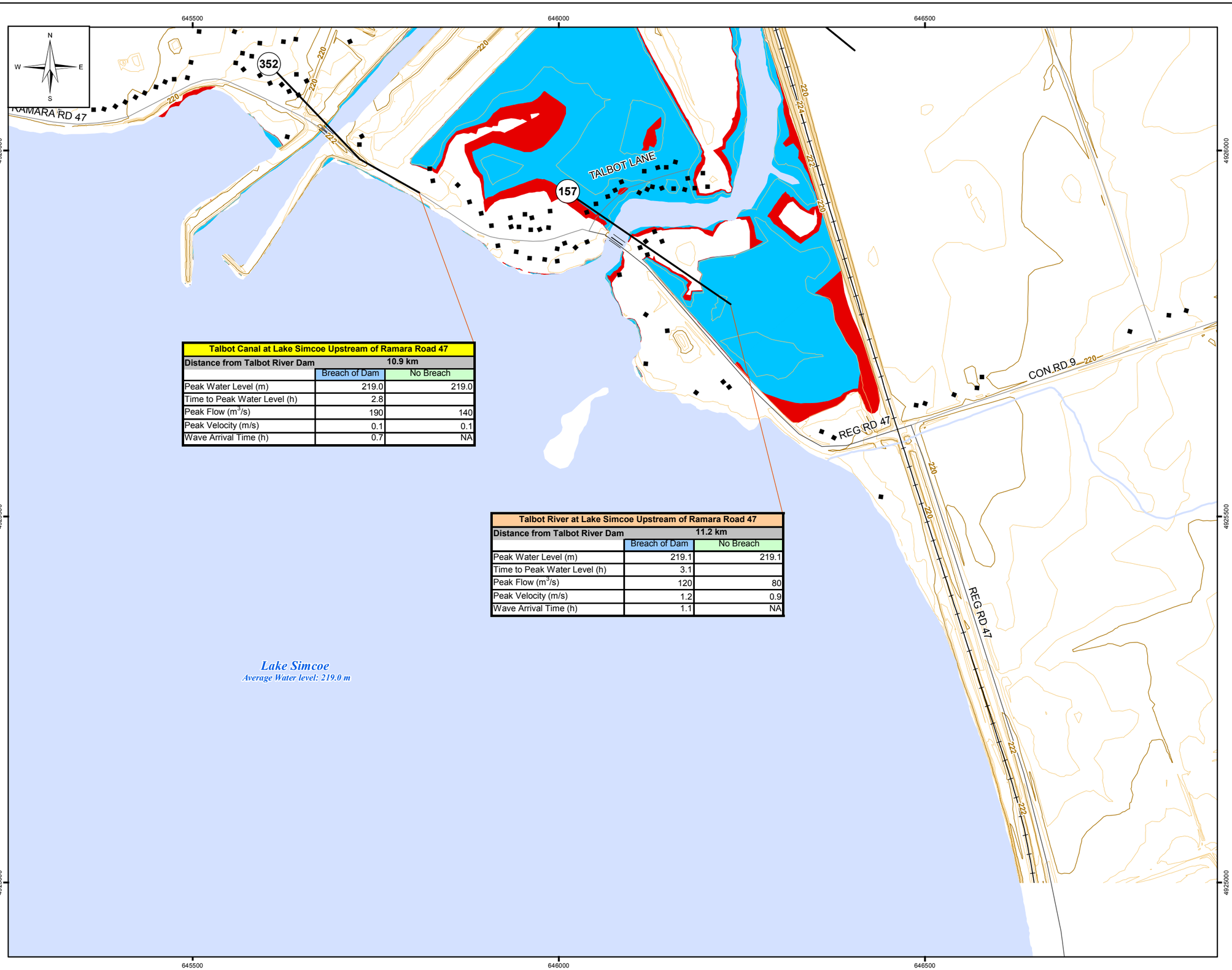
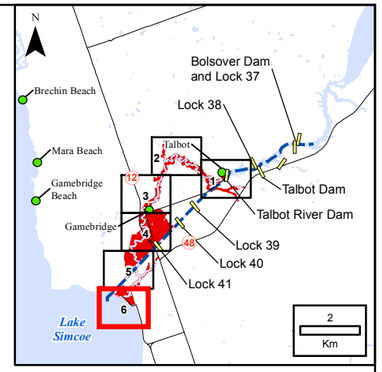


Talbot Canal at Lake Simcoe Upstream of Ramara Road 47		
Distance from Talbot River Dam	10.9 km	
	Breach of Dam	No Breach
Peak Water Level (m)	219.0	219.0
Time to Peak Water Level (h)	2.8	
Peak Flow (m^3/s)	190	140
Peak Velocity (m/s)	0.1	0.1
Wave Arrival Time (h)	0.7	NA

Talbot River at Lake Simcoe Upstream of Ramara Road 47		
Distance from Talbot River Dam	11.2 km	
	Breach of Dam	No Breach
Peak Water Level (m)	219.1	219.1
Time to Peak Water Level (h)	3.1	
Peak Flow (m^3/s)	120	80
Peak Velocity (m/s)	1.2	0.9
Wave Arrival Time (h)	1.1	NA



LEGEND:

- Flooded Area IDF No Breach of Talbot River Dam
- Flooded Area Breach of Talbot River Dam during the IDF
- Normal Lake or River Level
- Cross Section
- 2m Index Contour TIN
- 0.5m Contour TIN
- Dam
- Highway
- Major Road
- Local Street
- Flow Direction
- Building

Notes:

1. The limits of the flooding and the travel times shown on this map are approximate and should be used as guidelines for establishing evacuation zones and procedures in any dam break event, including extreme floods and normal flow conditions.
2. Indicated times in tables are relatives to the time of initiation of the breach of the Talbot River Dam for the dam breach case.
3. NA indicates "Not Applicable".
4. Base topographic features are from LIO (Land Information Ontario).
5. Elevation information is based on Province of Ontario Greater Toronto Area (GTA) topographic data 2002, provided by Public Works and Government Services Canada.
6. All units are metric and in metres unless otherwise specified. Transverse Mercator Projection, NAD 1983, Zone 17. Elevations are in metres above sea level (MSL).
7. The IDF obtained for Talbot River Dam was the flood 37% of the way between the 1000 years flood and the PMF.

50 0 50 100 150 200

Metres

SCALE: 1:5,000 METRIC 11"x17"

NO.	YY/MM/DD	DESCRIPTION	BY
0	15/04/08	ISSUED WITH FINAL REPORT	FGC

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DAM SAFETY REVIEW TALBOT RIVER DAM AND LOCK 39

BREACH OF THE TALBOT RIVER DAM DURING THE INFLOW DESIGN FLOOD (IDF) SHEET 6 OF 6

APRIL 2015 | FIGURE 01.06 | REV: 0

FileName: P:\Projects\2012\12-0006-028\Dwg\GIS\MXDs\Rev0\DF_TRD\12-0006-028_Fig01_Rev0.mxd
11"x17" PLOT SCALE 1:1

APPENDIX I

WATER IMPOUNDMENT

649500
650000
650500
4930500
4930000
4929500
Talbot River
Talbot
240
238
236
234
232
230
228
226

NO.	YY/MM/DD	DESCRIPTION	BY
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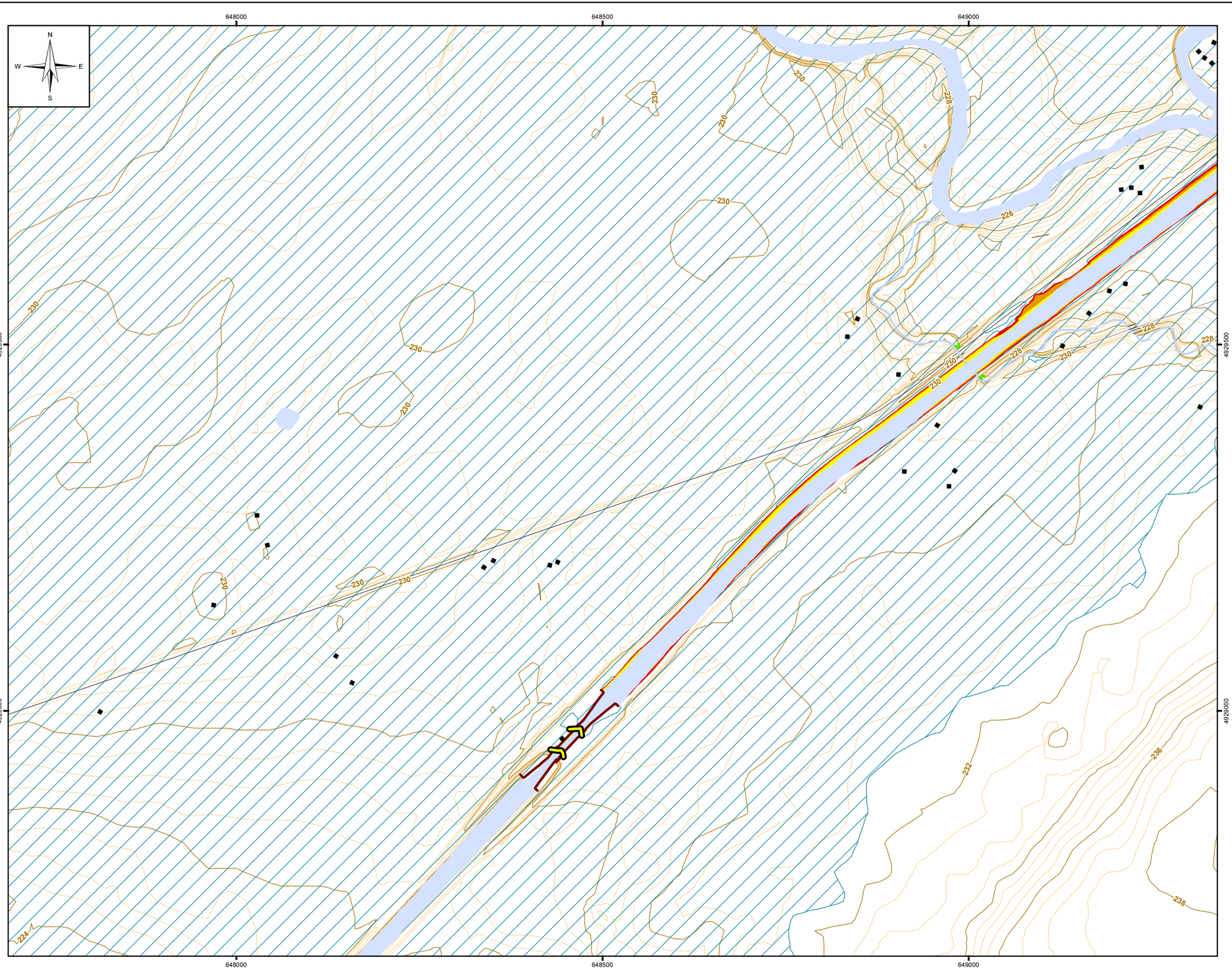
CLIENT

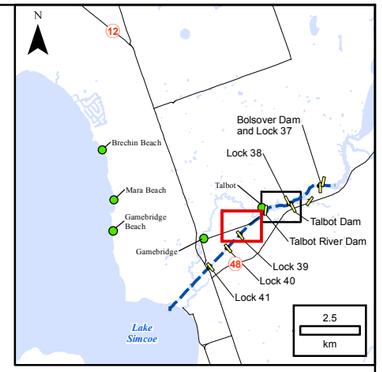
DAM SAFETY REVIEWS TALBOT

TALBOT RIVER DAM
WATER IMPOUNDMENT REVIEW

APRIL 2015	FIGURE I1	0

Portions of data Produced by KGS Group, under Licence with the Ontario Ministry of Natural Resources © Queen's Printer for Ontario, 2015.





LEGEND:

- Building
- Dam

INUNDATION TALBOT RIVER DAM

- 230.5m
- 231m
- 231.5m
- Area of Potential Overflow
- Normal Lake or River Level
- 2m Index Contour TIN
- 0.5m Contour TIN

Notes:

1. Base topographic features are from LIO (Land Information Ontario).
2. Elevation information is based on Province of Ontario Greater Toronto Area (GTA) topographic data 2002, provided by Public Works and Government Services Canada.
3. All units are metric and in metres unless otherwise specified. Transverse Mercator Projection, NAD 1983, Zone 17. Elevations are in metres above sea level (MSL).

0.05 0 0.05 0.1 0.15 0.2

Kilometers

SCALE: 1:5,000 METRIC 11"x17"

NO.	YY/MM/DD	DESCRIPTION	BY
0	15/04/08	ISSUED WITH FINAL REPORT	FGC

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DAM SAFETY REVIEWS TALBOT RIVER DAM AND LOCK 39

TALBOT RIVER DAM
WATER IMPOUNDMENT REVIEW
SHEET 2 OF 2

APRIL 2015 | FIGURE I1.02 | REV: 0

FileName:P:\Projects\2012\12-0006-028\Dwg\GIS\MXDs\Rev0\Water_Impoundment_Review_TRD\12-0006-028_Fig11_Rev0.mxd
11"x17" PLOT SCALE 1:1

APPENDIX J

STRUCTURAL STABILITY ANALYSES

KGS GROUP

DESIGN CALCULATIONS COVER SHEET

Project No. :	12-0006-028	**Project Name :**	Talbot River Dam (at Lock 39)		
File No. :		**Discipline :**	Civil Engineering		
Calculation Title :	**Gravity Bulkhead**				
Calculation No. :	CIV-001	**Prepared by :**	Y.F.	**Date :**	Mar. 22, 2013
No. of Sheets :	41	**Checked by :**	H.H.	**Date :**	OCT 8, 2013
Supersedes Calc. No. :		**Approved by :**	H. H.	**Date :**	OCT 8, 2013

Calculation Description :

The pier has been reviewed against 2007 CDA Dam Safety Guidelines

Related Design Concept :

Stability analysis for the structures is carried out by using "Gravity Method"
Five loading cases are utilized in the analyses based 2007 CDA Guidelines

Reference Codes and Standards :

1.Design of Small Dams, Third Edition, U.S. Government Printing Office, Washington, D.C. 1999.
2. 2007 CDA Dam Safety Guidelines
3. 2009 Parks Canada Directive for Dam Safety Program of Dams and Water-Retaining Structures

ENGINEER'S SEAL

Rev. #	Rev. Description	Rev. Author	Date Revised	Checked by	Approved by	Approved Date

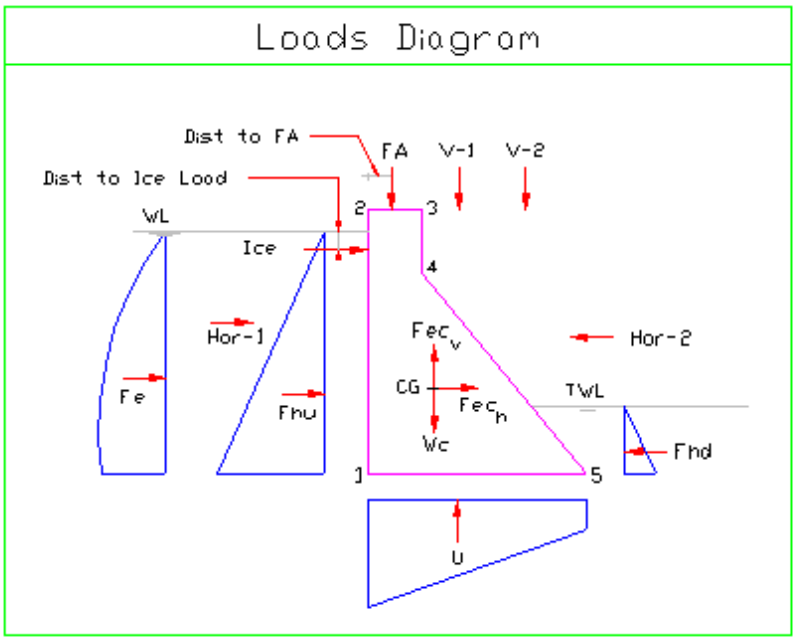
PROGRAM DESCRIPTION SECTION

This Mathcad file can calculate factors of safety against sliding and perform cracked section analysis for any shaped structure. Loading combinations conforms to CDA Dam Safety Guidelines 2007
Features

- metric or imperial units
- ice, anchor, hydrostatic and gravity loads
- crack analysis
- calculates percentage of effective base
- bearing pressures
- volume and position of center of gravity of structure (including voids)
- summary of forces and moments (including resultant and eccentricity)
- additional point loads (e.g.. backfill, vertical hydrostatic, etc.)
- hydrodynamic forces by formulas of Small Dam Design
- equivalent horizontal and vertical inertia forces on structure during earthquake

Assumptions and Limitations

- 2-dimensional
- horizontal base only
- vertical hydrostatic and backfill loads must be input as point loads
- lowest point on structure is level of base
- all vertical point loads are referenced to most upstream point of structure
- Evaluation of cracking during analyses are for unusual and extreme loading cases only.



INPUT SECTION

Unit System

1 - metric
2 - imperial

Select unit system

units := 1 <... select unit system

unL := if(units = 1, 1 · m, 1 · ft)

unM := if(units = 1, 1 · kN, 1 · lb)

Geometry of Structure defined by perimeter coordinates

There is no need to "close" the perimeter ,i.e., so that last point equal to first point

$$X := \begin{pmatrix} 0.000 \\ 0.000 \\ 1.68 \\ 1.68 \\ 0.99 \\ 0.99 \\ 1.68 \\ 2.31 \\ 3.05 \\ 3.05 \\ 0 \end{pmatrix} \cdot unL \qquad Y := \begin{pmatrix} 224.68 \\ 232.0 \\ 232.0 \\ 231.54 \\ 231.54 \\ 230.32 \\ 230.32 \\ 227.73 \\ 227.73 \\ 224.68 \\ 224.68 \end{pmatrix} \cdot unL$$

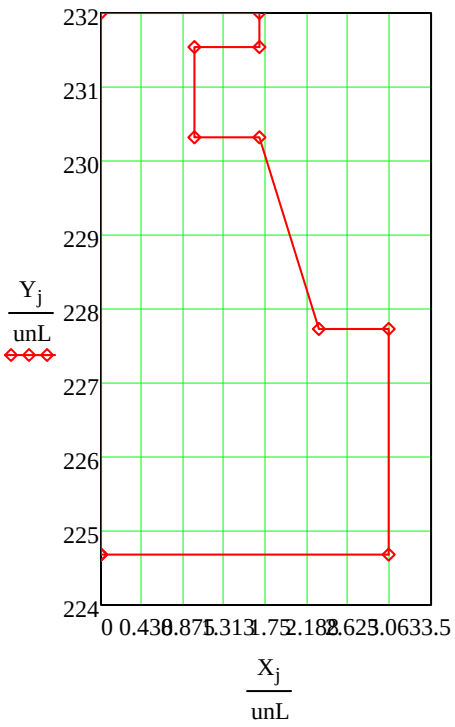
<... input X & Y coordinates

$$maxY := \frac{max(Y)}{unL} + 5 \qquad minY := \frac{min(Y)}{unL} - 5 \qquad i := 1 .. length(X)$$

$$maxX := \frac{max(X)}{unL} + 5 \qquad minX := \frac{min(X)}{unL} - 5 \qquad j := 1 .. length(X) + 1$$

$$X_{length(X)+1} := X_1 \qquad Y_{length(Y)+1} := Y_1$$

Graphical Representation of Structure defined by X & Y Coordinates



Densities of Water and Concrete, Width of Structure, of Structure defined by perimeter coordinates

$\gamma w := \text{if}\left(\text{units} = 1, 9.81 \cdot \frac{kN}{m^3}, 62.5 \cdot \frac{lb}{ft^3}\right)$ <... water density

$\gamma_{silt} := \text{if}\left(\text{units} = 1, 7.7 \cdot \frac{kN}{m^3}, 49 \cdot \frac{lb}{ft^3}\right)$ <... Silt density

$\gamma_{backfill} := \text{if}\left(\text{units} = 1, 19 \cdot \frac{kN}{m^3}, 120 \cdot \frac{lb}{ft^3}\right)$ <... Backfill density

$\gamma c := \text{if}\left(\text{units} = 1, 23.5 \cdot \frac{kN}{m^3}, 145 \cdot \frac{lb}{ft^3}\right)$ <... concrete density

$\gamma_{timber} := \text{if}\left(\text{units} = 1, 10 \cdot \frac{kN}{m^3}, 62.4 \cdot \frac{lb}{ft^3}\right)$ <... timber density (for Stoplogs)

$\gamma_{steel} := \text{if}\left(\text{units} = 1, 77 \cdot \frac{kN}{m^3}, 490 \cdot \frac{lb}{ft^3}\right)$ <... Steel density

$UNI_{floor} := \text{if}\left(\text{units} = 1, 0.5 \cdot \frac{kN}{m^2}, 10.4 \cdot \frac{lb}{ft^2}\right)$ <... Steel deck unit area weight

$f'_c := \text{if}(\text{units} = 1, 17 \cdot MPa, 2030 \cdot psi)$ <... concrete compressive strength

$f'_{rock} := if(units = 1, 30 \cdot MPa, 4350 \cdot psi)$ <... mass rock compressive strength

$Structure_Width := 1.0$ <... width of structure

$\phi := 36 \cdot deg$ <... angle of internal friction for foundation material

$c := 0.0 \cdot f'_c$ $c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

Computation of Area and Coordinates of Center of Gravity of Structure

$deltax_i := X_{i+1} - X_i$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot (deltay_i \cdot xplusx_i)$

$Yginc_i := \frac{deltax_i}{8} \cdot \left[(yplusy_i)^2 + \frac{(deltay_i)^2}{3} \right]$ $Xginc_i := \frac{deltay_i}{8} \cdot \left[(xplusx_i)^2 + \frac{(deltax_i)^2}{3} \right]$

$Area := \sum_i Areainc_i$ $Area := if\left[\frac{Area}{unL^2} > 0, Area, Area \cdot (-1) \right]$ <... Area of the Structure (defined by X & Y)

$Xg := \sum_i Xginc_i$ $Xg := \frac{if\left[\frac{Xg}{unL^3} > 0, Xg, Xg \cdot (-1) \right]}{Area}$ <... X coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$Yg := \sum_i Yginc_i$ $Yg := \frac{if\left[\frac{Yg}{unL^3} > 0, Yg, Yg \cdot (-1) \right]}{Area}$ <... Y coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$B := Structure_Width \cdot (unL)$ $L := \max(X) - \min(X) = 3.05\,m$

Computation of Concrete Volume and Weight in Structure

$Vol_conc := Area \cdot B$ $Vol_conc = 16 \cdot m^3$ <... volume of concrete in structure

$Wc := Area \cdot B \cdot \gamma c$ $Wc = 386.6 \cdot kN$ <... weight of concrete in structure

Load Case 1. Usual Loading Summer Case (D+H+S+U)

US_Water_Level := 230.44 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

CREST_ELE := 232.0 <...Crest Elevation

US_L_ELE := 224.68 <...Lowest Dam Elevation at upstream

DS_L_ELE := 224.68 <...Lowest Dam Elevation at downstream

ROLLWAY_TOP_ELE := 224.68 <...Rollways Top Elevation

$$\alpha := \operatorname{atan}\left(\frac{DS_L_ELE - US_L_ELE}{\frac{L}{unL}}\right) = 0 \cdot deg$$

<...Angle of inclined rock foundation

SF := 2.0 <... Minimum Safety Factor

Deck Slab and Gate weight

$Sp_{gate} := 0$ <... Clear opening for gate/stoplogs

$N_s := 0$ <... Number of Stoplogs

Lsize := 0.305 <... Size of stoplog

$$W_{gate} := \gamma_{timber} \cdot \left[Sp_{gate} + 2(Lsize)\right] \cdot unL \cdot (Lsize \cdot unL)^2 \cdot N_s$$

$W_{gate} = 0 \cdot kN$ <... Weight of gate/stoplogs

$S_{wid} := 4.572 - 0.305$ <... Slab width

$S_{thk} := 0.16$ <... Equivalent slab thickness

$$W_{slab} := \gamma c \cdot Sp_{gate} \cdot S_{wid} \cdot S_{thk} \cdot (unL)^3$$

$W_{slab} = 0 \cdot kN$ <... Weight of slab

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.** **Otherwise,** i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

$$US_Hor_Force := \frac{1}{2} \cdot \gamma w \cdot (US_Water_Level - ROLLWAY_TOP_ELE)^2 \cdot Sp_{gate} \cdot \frac{(unL)^3}{kN}$$

US_Hor_Force = 0 <...Hydrostatic force on Gate / Stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

EL_US_Hor_F = 226.6 <...Elevation of the Hydrostatic force on Gate / Stoplogs

$\phi' := 25 \cdot deg$ <... angle of internal friction for Silt at rest condition

$Ko := 1 - \sin(\phi') = 0.58$

SILT_ELE := 227.73 <... Silt top elevation

$$Silt_Pressure := \frac{1}{2} \cdot (\gamma_{silt}) \cdot Ko \cdot (SILT_ELE \cdot unL - DS_L_ELE \cdot unL)^2 \cdot B = 20.68 \cdot kN$$

$$E_{silt} := DS_L_ELE + \frac{1}{3}(SILT_ELE - DS_L_ELE) = 225.7$$

$\phi 1' := 30 \cdot deg$ <... angle of internal friction for Backfill at rest condition

$Ko1 := 1 - \sin(\phi 1') = 0.5$

Backfill_ELE := 227.73 <... Backfill top elevation

$$Backfill_Pressure := \frac{1}{2} \cdot (\gamma_{backfill}) \cdot Ko1 \cdot (Backfill_ELE \cdot unL - DS_L_ELE \cdot unL)^2 \cdot B = 44.19 \cdot kN$$

$soil_pressure := Silt_Pressure - Backfill_Pressure = -23.51 \cdot kN$

$$E_{backfillt} := DS_L_ELE + \frac{1}{3}(Backfill_ELE - DS_L_ELE) = 225.7$$

$$E_{eq} := \frac{Silt_Pressure \cdot \left(\frac{SILT_ELE - DS_L_ELE}{3}\right) - Backfill_Pressure \cdot \left(\frac{Backfill_ELE - DS_L_ELE}{3}\right)}{soil_pressure} \ldots + DS_L_ELE = 225.7$$

$$DS_Hor_Force := \frac{-soil_pressure}{unM} = 23.51$$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$$EL_DS_Hor_F := E_{eq}$$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$$Vert_Force_1 := \frac{W_{slab}}{unM}$$

<... Weight of Deck slab

$$Dist_to_F1 := 0$$

<... distance from most upstream point of structure to point of application of vertical force

$$Vert_Force_2 := 0$$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$$Dist_to_F2 := 0$$

<... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$$WL := US_Water_Level \cdot (unL) \qquad F_EL := US_L_ELE \cdot (unL) \qquad F_EL_D := DS_L_ELE \cdot (unL)$$

$$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL) \qquad F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$$

$$F_ANC := Anchor_Force \cdot (unM) \qquad Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$$

$$V1 := Vert_Force_1 \cdot (unM) \qquad Dist_V1 := L - Dist_to_F1 \cdot (unL)$$

$$V2 := Vert_Force_2 \cdot (unM) \qquad Dist_V2 := L - Dist_to_F2 \cdot (unL)$$

$$USHF := US_Hor_Force \cdot (unM) \qquad EL_USF := EL_US_Hor_F \cdot (unL)$$

$DSHF := DS_Hor_Force \cdot (unM)$ $EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 163 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 86.17 \cdot kN$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF - DSHF) = 139.23 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces <u>without uplift</u>

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ ... = 694.25 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fi \cdot (EL_ICE - F_EL_D) \ ... = 288.55 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) - DSHF \cdot (EL_DSF - F_EL_D)$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ $x_o = 0.77\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ $E = 0.76\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 245.32 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -48.33 \cdot kPa$

$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$ Calculation_to_be = "stoped"

(To ensure un cracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 0.75\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \\ (L - PI) & \text{otherwise} \end{cases}$ $= 2.3\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 2.3\,m^2$

$L_1 := L_c + L_t = 2.3\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 300.41 \cdot kN$ $Mr := Ms - Mo_T = 230.48 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 0.77\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 0.76\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 245.32 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -48.33 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 261.03 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 0.75\,m$ <...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$ <...Stress at Foundation

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.57$ <...Factor of Safety Against Sliding.

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

$\Sigma M := Ms - Mo_1$

$\Sigma W := Wc + Fa + V1 + V2$

$A3 := Phu \cdot B$ $T := L$

$T1 := \begin{cases} \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} & \text{if } \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases}$ $T1 = 2\,m$ <...Uncracked section depth

$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$ $T1 = 2\,m$ <...Full base cracked

$$B5 := \begin{cases} \dfrac{2 \cdot \dfrac{\Sigma W - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases} \qquad B5 = 270.66 \cdot kPa$$

<...Stress at downstream face

$$Cr := T - T1 \qquad Cr = 1.05 \cdot m$$

<...Cracked section depth

$$T1 := T - Cr = 2\,m$$

$$A_c := \min(B \cdot T1, A_c) = 2\,m^2$$

$$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] + U_d = 115.82 \cdot kN$$

$$F_{ver} := Wc + Fa + V1 + V2 - U = 270.76 \cdot kN$$

$$F_{ver} := F_{ver} + N_{ten} = 270.76 \cdot kN$$

$$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2}\right) + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] \cdot \frac{2}{3} \cdot (T1)\right] + U_d \cdot \frac{L}{2} = 225 \cdot kN \cdot m$$

$$Mo_T := Mo_1 + Mo_3 = 514 \cdot kN \cdot m \qquad Mr := Ms - Mo_T$$

$$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases} \qquad fmax := B5$$

$$x_o := \begin{cases} \dfrac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \dfrac{T1}{3} & \text{otherwise} \end{cases} \qquad x_o = 0.67\,m$$

$$EFF_BASE := \frac{T1}{L}$$

$$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.413$$

<...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 1

Initial point of inflection

$$PI = 0.75 \cdot m$$

Initial point of inflection as % of the base

$$\frac{PI}{L} = 24.53 \cdot \%$$

Result := "ok" if $E \le \frac{L}{6}$; "to increase base size" otherwise

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

CALCULATED CRACK LENGTH | **% OF CRACKED BASE** | **% OF EFFECTIVE BASE** (portion not cracked)

$Cr = 1.049 \cdot m$ | $\frac{Cr}{T} = 34.4 \cdot \%$ | $EFF_BASE = 65.6 \cdot \%$

$q_{1min} = -48.3 \cdot kPa$

When c is taken into account

Result := "ok" if $q_{1min} \ge \frac{f_t}{SF}$; "to increase base size" otherwise

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

fmin = $0 \cdot kPa$

fmax = $270.66 \cdot kPa$

$x_o = 0.67\ m$

<...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

Normal_stresses := "OK" if $f \le 0.3 \cdot \min(f'_c, f'_{rock})$; "not good" otherwise

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

FSS = 1.41

Safety_against_sliding := "OK" if $FSS \ge 1.5$; "to increase dam size" otherwise

Safety_against_sliding = "to increase dam

When c is taken into account

Safety_against_sliding := "OK" if $FSS \ge SF$; "to increase dam size" otherwise

Safety_against_sliding = "to increase dam s

Summary of Forces and Moments

1) Vertical Forces

$Wc = 386.58 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 0 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U = 115.82 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 386.58 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 270.76 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 162.74 \cdot kN$ <... upstream hydrostatic force

$Fhd = 0 \cdot kN$ <... downstream hydrostatic force

$Fi = 0 \cdot kN$ <... Ice Force

$USHF = 0 \cdot kN$ <...horizontal force acting on the Downstream Direction

$DSHF = 23.51 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 139.23 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 694.25 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 513.68 \cdot kN \cdot m$ <... overturning moment

$Mr = 180.58 \cdot kN \cdot m$ <... resultant moment

Load Case 2. Usual Loading Winter Case (D+H+S+U+I)

US_Water_Level := 230.14 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

SF := 2.0 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 77 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0.3 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

$$US_Hor_Force := \frac{1}{2} \cdot \gamma w \cdot (US_Water_Level - ROLLWAY_TOP_ELE)^2 \cdot Sp_{gate} \cdot \frac{(unL)^3}{kN}$$

US_Hor_Force = 0 <...Hydrostatic force on Gate / Stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

EL_US_Hor_F = 226.5 <...Elevation of the Hydrostatic force on Gate / Stoplogs

US_Hor_Force1 := Ice_Load · Sp_{gate} US_Hor_Force1 = 0 <... Ice Load on gate / stoplogs

EL_US_Hor_F1 := US_Water_Level − Dist_WL_to_Ice_Load

EL_US_Hor_F1 = 229.84 <...Elevation of the ice load

$DS_Hor_Force := \frac{-soil_pressure}{unM} = 23.51$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{ec}$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{slab}}{unM}$

<... Weight of Deck Slab

$Dist_to_F1 := 0$

<... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := 0$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := 0$

<... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$

$F_EL := US_L_ELE \cdot (unL)$

$F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$

$F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$

$EL_USF := EL_US_Hor_F \cdot (unL)$

$USHF1 := US_Hor_Force1 \cdot (unM)$

$EL_USF1 := EL_US_Hor_F1 \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$

$EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$

$TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 146.23 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Net uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Net uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF + USHF1 - DSHF)$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ ... = 694.25 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fi \cdot (EL_ICE - F_EL_D) \ ... = 639.55 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) + USHF1 \cdot (EL_USF1 - F_EL_D) \ ...$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 471.71 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -271.77 \cdot kPa$

$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$

Calculation_to_be = "stoped"

(To ensure uncracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 4.15\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \\ (L - PI) & \text{otherwise} \end{cases} = -1.1\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 0\,m^2$

$L_1 := L_c + L_t = -1.1\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 304.9 \cdot kN$ $Mr := Ms - Mo_T = -111.39 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = -0.37\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.89\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 471.71 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -271.77 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = -556.39 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$

<...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 4.15\,m$

<...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$

<...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } fmin \leq 0 \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 4.15\,m$

<...Point of inflection of stresses at foundation from heel

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.11$

<...Factor of Safety Against Sliding.

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

$\Sigma M := Ms - Mo_1$

$\Sigma W := Wc + Fa + V1 + V2$

$A3 := Phu \cdot B$ $T := L$

$T1 := \begin{cases} 3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right) & \text{if } \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ \Sigma W - A3 \cdot T & \\ T & \text{otherwise} \end{cases}$ $T1 = -2.61\,m$

<...Uncracked section depth

$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$ $T1 = 0\,m$

<...Full base cracked

$$B5 := \begin{cases} 2 \cdot \dfrac{\Sigma W - A3 \cdot T}{T1} + A3 \\ \hline B \end{cases} \quad \text{if } T1 < T \qquad \text{fmax otherwise}$$

$B5 = 4464251953.56 \cdot kPa$ ...Stress at downstream face

$Cr := T - T1$ $\quad Cr = 3.05 \cdot m$ <...Cracked section depth

$T1 := T - Cr = 0\,m$ $\quad A_c := \min(B \cdot T1, A_c) = 0\,m^2$

$$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] + U_d = 163.37 \cdot kN$$

$F_{ver} := Wc + Fa + V1 + V2 - U = 223.21 \cdot kN$

$F_{ver} := F_{ver} + N_{ten} = 223.21 \cdot kN$

$$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2}\right) + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] \cdot \frac{2}{3} \cdot (T1)\right] + U_d \cdot \frac{L}{2} = 249 \cdot kN \cdot m$$

$Mo_T := Mo_1 + Mo_3 = 889 \cdot kN \cdot m$ $\quad Ms := Ms + M_{ten}$ $\quad Mr := Ms - Mo_T$

$$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases} \qquad fmax := B5$$

$$x_o := \begin{cases} \dfrac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \dfrac{T1}{3} & \text{otherwise} \end{cases} \qquad x_o = 0\,m$$

$$EFF_BASE := \frac{T1}{L}$$

$$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.81$$

<...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 2

Initial point of inflection

$PI = 4.15 \cdot m$

Initial point of inflection as % of the base

$$\frac{PI}{L} = 135.93 \cdot \%$$

$$Result := \begin{cases} \text{"ok"} & \text{if } E \leq \dfrac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

CALCULATED CRACK LENGTH | **% OF CRACKED BASE** | **% OF EFFECTIVE BASE** (portion not cracked)

$Cr = 3.05 \cdot m$ | $\frac{Cr}{T} = 100 \cdot \%$ | $EFF_BASE = 3.28 \times 10^{-6} \cdot \%$

$q_{1min} = -271.8 \cdot kPa$

When c is taken into account

$$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \geq \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 4.46 \times 10^{9} \cdot kPa$

$x_o = 0\,m$

<...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.3 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "not good"

SAFETY FACTOR AGAINST SLIDING

$FSS = 0.81$

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \geq 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam

When c is taken into account

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \geq SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam s

Summary of Forces and Moments

1) Vertical Forces

$Wc = 386.58 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 0 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U = 163.37 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 386.58 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 223.21 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 146.23 \cdot kN$ <... upstream hydrostatic force

$Fhd = 0 \cdot kN$ <... downstream hydrostatic force

$Fi = 77 \cdot kN$ <... Ice Force

$USHF = 0 \cdot kN$ <...horizontal force **due to hydrostatic pressure from stoplogs**

$USHF1 = 0 \cdot kN$ <...horizontal force **due to ice pressure from stoplogs**

$DSHF = 23.51 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 199.72 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 694.25 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 888.68 \cdot kN \cdot m$ <... overturning moment

$Mr = -194.43 \cdot kN \cdot m$ <... resultant moment

Load Case 3. Unusual Loading IDF ($D+H_{IDF}+S+U_{IDF}$)

IDF_Level := 231.03 <... IDF level

Tailwater_Level_IDF := 230.44 <... tailwater level at IDF

SF := 1.5 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0

- Drag Force

<... Horizontal Force acting on the Downstream Direction. For instance, this force can be used to input effect of soil pressure on the upstream side of Structure

EL_US_Hor_F := 0 <...Elevation at which the Upstream Horizontal Force is applied to the Structure

$DS_Hor_Force := \frac{-Silt_Pressure}{unM} = -20.68$ Silt pressure

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{silt}$ <...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{slab}}{unM}$ <... Weight of Deck Slab

$Dist_to_F1 := 0$ <... distance from most upstream point of structure to point of application of vertical force

$\tan\theta := \frac{8.5}{12}$ <... Downstream face slope

$h1 := Tailwater_Level_IDF - DS_L_ELE = 5.76$ <... height of the tailwater

$W_{tailwater} := \frac{1}{2} \cdot \frac{B}{unL} \cdot (h1)^2 \cdot \tan\theta \cdot \frac{\gamma w}{\left(\frac{kN}{m^3}\right)} = 115.27$ <... Weight of tailwater on dam

$L_{tailwater} := \frac{1}{3} \cdot h1 \cdot \tan\theta = 1.36$ <... Moment arm of Weight of tailwater to toe

$Vert_Force_2 := W_{tailwater} = 115.27$ <... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := \frac{L}{unL} - L_{tailwater} = 1.69$ <... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL_{IDF} := IDF_Level \cdot (unL)$

$TWL_{IDF} := Tailwater_Level_IDF \cdot (unL)$

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$

$EL_USF := EL_US_Hor_F \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$

$EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu_{IDF} := (WL_{IDF} - F_EL) \cdot \gamma w$ $F_EL = 224.68\,m$ <... upstream hydrostatic pressure

$Phd_{IDF} := (TWL_{IDF} - F_EL_D) \cdot \gamma w$ $F_EL_D = 224.68\,m$ <... downstream hydrostatic pressure

$Fhu_{IDF} := Phu_{IDF} \cdot \frac{WL_{IDF} - F_EL}{2} \cdot B$ $Fhu_{IDF} = 197.78 \cdot kN$ <... upstream hydrostatic force

$Fhd_{IDF} := Phd_{IDF} \cdot \frac{TWL_{IDF} - F_EL_D}{2} \cdot B$ $Fhd_{IDF} = 162.74 \cdot kN$ <... downstream hydrostatic force

$Pu_{IDF} := Phu_{IDF} - Phd_{IDF}$ <... Uplift pressure at upstream end

$U_{uIDF} := \frac{(Pu_{IDF} \cdot Ueff) \cdot L \cdot B}{2}$ $U_{uIDF} = 8.83 \cdot kN$ <... Uplift force due to upstream head water

$U_{dIDF} := Phd_{IDF} \cdot L \cdot B$ $U_{dIDF} = 172.34 \cdot kN$ <... Uplift force due to downstream tailwater

$U_{IDF} := U_{uIDF} + U_{dIDF} = 181.17 \cdot kN$ <... Total Uplift force

$F_{horIDF} := (Fhu_{IDF} - Fhd_{IDF}) + (USHF - DSHF)$ <... Resultant of Horizontal Forces

$F_{verIDF} := Wc + Fa + V1 + V2 - U_{IDF} = 320.68 \cdot kN$ <... Resultant of Vertical Forces

$Ms_{IDF} := Wc \cdot (L - Xg) + Fhd_{IDF} \cdot \left(\frac{TWL_{IDF} - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 1163.48 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_{1IDF} := Fhu_{IDF} \cdot \left[\frac{WL_{IDF} - F_EL}{3} + (F_EL_D - F_EL)\right] + USHF \cdot (EL_USF - F_EL_D) \ldots = 439.66 \cdot kN \cdot m$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$ <... Overturning Moment (uplift not included)

$Mo_{2IDF} := U_{uIDF} \cdot \left(\frac{2 \cdot L}{3}\right) + U_{dIDF} \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_{TIDF} := Mo_{1IDF} + Mo_{2IDF}$ <... Total Overturning Moment

$Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$ <... Resultant Moment

$x_{oIDF} := \frac{Mr_{IDF}}{F_{verIDF}}$ $x_{oIDF} = 1.38\,m$ <...Resultant location. Distance from the toe.

$E_{IDF} := \frac{L}{2} - x_{oIDF}$ $E_{IDF} = 0.14\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E_{IDF}}{L}\right) = 134.81 \cdot kPa$ $q_{1min} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E_{IDF}}{L}\right) = 75.48 \cdot kPa$

(To consider cohesion and tensile

$$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$$ $L_t = 0\,m$ <...Length of base under tention

$$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 0\,m$$

$$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \\ (L - PI) & \text{otherwise} \end{cases} = 3.05\,m$$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases} = 3.05\,m^2$ $L_1 := L_c + L_t = 3.05\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms_{IDF} := Ms_{IDF} + M_{ten}$ $F_{verIDF} := F_{verIDF} + N_{ten} = 320.68 \cdot kN$ $Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$

$x_{oIDF} := \frac{Mr_{IDF}}{F_{verIDF}}$ $x_{oIDF} = 1.38\,m$

$E_{IDF} := \frac{L}{2} - x_{oIDF}$ $E_{IDF} = 0.14\,m$

$q_{1max} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E_{IDF}}{L}\right) = 134.81 \cdot kPa$ $q_{1min} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E_{IDF}}{L}\right) = 75.48 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \dfrac{2 \cdot F_{verIDF}}{3 \cdot x_{oIDF} \cdot B} & \text{otherwise} \end{cases} = 134.81 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 75.48 \cdot kPa$

<...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot (x_{oIDF}) & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 0\,m$

<...Point of inflection of stresses at foundation from heel

$$FSS := \frac{(F_{verIDF} \cdot \cos(\alpha) + F_{horIDF} \cdot \sin(\alpha)) \cdot \tan(\phi) + c \cdot A_c}{F_{horIDF} \cdot \cos(\alpha) - F_{verIDF} \cdot \sin(\alpha)} = 4.18$$

<...Factor of Safety Against Sliding.

STRESSES AT FOUNDATION

fmin = 75.5 · kPa

fmax = 134.8 · kPa

initial point of inflection

$PI = 0 \cdot m$

initial point of inflection as % of the base

$\frac{PI}{L} = 0 \cdot \%$

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

Static Method of Analysis.-*Assumptions associated with a static, cracked-section analysis are:*
1 Stress distribution along a horizontal section,without uplift, varies linearly between upstream and downstream faces.
2 Once a crack occurs, uplift pressure equivalent to reservoir pressure above the crack exists throughout the entire crack depth. Uplift is then assumed to vary linearly from crack tip to tailwater pressure at downstream face.
3 Crack penetrates to point of zero stress. This assumes no tensile strength at crack tip, which means the entire untracked length is entirely in compression.

Refer to "Design of small dam" P 331-332

$\Sigma M_{IDF} := Ms_{IDF} - Mo_{1IDF}$ $\Sigma W_{IDF} := Wc + Fa + V1 + V2$ $A3 := Phu_{IDF} \cdot B$

$T := L$

$$T1 := \begin{cases} \dfrac{3 \cdot \left(\Sigma M_{IDF} - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W_{IDF} - A3 \cdot T} & \text{if } \dfrac{3 \cdot \left(\Sigma M_{IDF} - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W_{IDF} - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases}$$

$T1 = 3.05\,m$

<...Un cracked section depth

$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$

<...Full base cracked

$$B5 := \begin{cases} \dfrac{2 \cdot \dfrac{\Sigma W_{IDF} - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases}$$

<...Stress at downstream face

$Cr := T - T1$ $\quad Cr = 0 \cdot m$ <...Cracked section depth

$T1 := T - Cr = 3.05\,m$ $\quad A_c := \min(B \cdot T1, A_c) = 3.05\,m^2$

$U_{IDF} := (Pu_{IDF} \cdot Ueff) \cdot Cr \cdot B + (Pu_{IDF} \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] + U_{dIDF} = 181.17 \cdot kN$

$F_{ver_UIDF} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces without uplift

$F_{verIDF} := Wc + Fa + V1 + V2 - U_{IDF} = 320.68 \cdot kN$ <... Resultant of Vertical Forces

$F_{verIDF} := F_{verIDF} + N_{ten} = 320.68 \cdot kN$

$$Mo_{3IDF} := \left[(Pu_{IDF} \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2}\right) + (Pu_{IDF} \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] \cdot \frac{2}{3} \cdot (T1)\right] + U_{dIDF} \cdot \frac{L}{2}$$

$Mo_{TIDF} := Mo_{1IDF} + Mo_{3IDF}$ $\quad Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$

$$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$$

$fmax := B5$ $\quad EFF_BASE := \dfrac{T1}{L}$

$$x_{oIDF} := \begin{cases} \dfrac{Mr_{IDF}}{F_{verIDF}} & \text{if } Cr = 0 \\ \dfrac{T1}{3} & \text{otherwise} \end{cases} \qquad x_{oIDF} = 1.38\,m$$

$$FSS := \frac{(F_{verIDF} \cdot \cos(\alpha) + F_{horIDF} \cdot \sin(\alpha)) \cdot \tan(\phi) + c \cdot A_c}{F_{horIDF} \cdot \cos(\alpha) - F_{verIDF} \cdot \sin(\alpha)} = 4.18$$

<...Factor of Safety Against Sliding.

RESULTS for Loading Case 3

Initial point of inflection

$PI = 0 \cdot m$

Initial point of inflection as % of the base

$\dfrac{PI}{L} = 0 \cdot \%$

CALCULATED CRACK LENGTH	% OF CRACKED BASE	% OF EFFECTIVE BASE (portion not cracked)

$Cr = 0 \cdot m$ $\quad \frac{Cr}{T} = 0 \cdot \%$ $\quad EFF_BASE = 100 \cdot \%$

Result := "ok" if $EFF_BASE \geq 75 \cdot \%$; "to increase base size" otherwise

Result = "ok"

<...Resultant required to be is within base.

$q_{1min} = 75.5 \cdot kPa$

When c is taken into account

Result := "ok" if $q_{1min} \geq \frac{f_t}{SF}$; "to increase base size" otherwise

<...Resultant shall be within middle third of the base.

Result = "ok"

STRESSES AT FOUNDATION

fmin = $75.5 \cdot kPa$ $\quad$ fmax = $134.8 \cdot kPa$

$f := \max(fmin, fmax)$

$x_{oIDF} = 1.382\ m$

Normal_stresses := "OK" if $f \leq 0.5 \cdot \min(f'_c, f'_{rock})$; "not good" otherwise

<... Allowable compressive strength =0.5 x f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST OVERTURNING and SLIDING

FSS = 4.18

Safety_against_sliding := "OK" if $FSS \geq 1.3$; "to increase dam size" otherwise

Safety_against_sliding = "OK"

When c is taken into account

Safety_against_sliding := "OK" if $FSS \geq SF$; "to increase dam size" otherwise

Safety_against_sliding = "OK"

Summary of Forces and Moments

1) Vertical Forces

$Wc = 386.58 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 0 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 115.27 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U_{IDF} = 181.17 \cdot kN$ <... uplift force acting upwards

$F_{ver_UIDF} = 501.85 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{verIDF} = 320.68 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu_{IDF} = 197.78 \cdot kN$ <... upstream hydrostatic force

$Fhd_{IDF} = 162.74 \cdot kN$ <... downstream hydrostatic force

$USHF = 0 \cdot kN$ <...Drag Force

$DSHF = -20.68 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{horIDF} = 55.72 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms_{IDF} = 1163.48 \cdot kN \cdot m$ <... stabilizing moment

$Mo_{TIDF} = 720.43 \cdot kN \cdot m$ <... overturning moment

$Mr_{IDF} = 443.04 \cdot kN \cdot m$ <... resultant moment

Load Case 4. Extreme Loading Earthquake (D+H+S+Q+U_Q)

Horizontal & Vertical Components of the earthquake Intensity and Coefficient of Hydrodynamic Pressure Distribution

$\lambda_{hor} := 0.05$ <... **horizontal** component of earthquake intensity = ratio of earthquake acceleration to acceleration due to gravity

$\lambda_{vert} := \frac{2}{3} \cdot \lambda_{hor}$ <... **vertical** component of earthquake intensity

<... from owner's requirement 21.2.5.1

$C_m := 0.74$ <... from fig 8-6, page 325 of "Design of Small Dams", for a structure with **vertical face**

US_Water_Level := 230.44 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

$f_t := 0$ <... tensile strength at interface of concrete /rock

SF := 1.3 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **acceleration,** i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

$$US_Hor_Force := \frac{1}{2} \cdot \gamma w \cdot (US_Water_Level - ROLLWAY_TOP_ELE)^2 \cdot Sp_{gate} \cdot \frac{(unL)^3}{kN}$$

US_Hor_Force = 0 <...Hydrostatic on stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

<...Elevation at which the Upstream Horizontal Force is applied to the Structure

EL_US_Hor_F = 226.6

$$DS_Hor_Force := \frac{-soil_pressure}{unM} = 23.51$$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{ed}$ <...Elevation at which the Downstream Horizontal Force is applied to the Structure

$W_{tower} := 0$ <...Weight of tower

$H_{tower} := 0$ <...Height of tower

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **acceleration,** i.e., metric units, **Vertical Forces** will be in **kN**

$$Vert_Force_1 := \frac{W_{slab}}{unM}$$ <... Weight of Deck Slab

Dist_to_F1 := 0 <... distance from most upstream point of structure to point of application of vertical force

Vert_Force_2 := 0 <... Vertical Force acting on the Downward Direction. It is a stabilizing force

Dist_to_F2 := 0 <... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$ $F_EL := US_L_ELE \cdot (unL)$ $F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$ $F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

$F_ANC := Anchor_Force \cdot (unM)$ $Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$ $EL_USF := EL_US_Hor_F \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$ $EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces and Uplift

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 162.74 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d$ <... Total Uplift force

Computation of Hydrodynamic Force Due to Earthquake

$h := US_Water_Level \cdot (unL) - F_EL$ $h = 5.76\,m$ <...total depth of the reservoir at section being analyses

$y := h$ <....vertical distance from reservoir surface to the elevation in question

$C := \frac{C_m}{2} \cdot \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right) + \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right)\right]^{0.5}\right]$ <...Coefficient of Hydrodynamic Pressure Distribution

$Pe := C \cdot \lambda_{hor} \cdot \gamma w \cdot h \cdot Structure_Width \cdot (unL)$ $Pe = 2.09 \cdot \frac{kN}{m}$ <...hydrodynamic pressure

$Fe1 := 0.726 \cdot Pe \cdot y$ $Fe1 = 8.7 \cdot kN$ <...hydrodynamic force at pier

$Me := 0.299 \cdot Pe \cdot y^2$ $Me = 21 \cdot kN \cdot m$ <...moment at the base of the structure due to hydrodynamic force

$P_{appl} := \begin{cases} F_EL + \frac{Me}{Fe1} & \text{if } Fe1 > 0kN \\ 0 \cdot m & \text{otherwise} \end{cases}$ $P_{appl} = 227.052 \cdot m$ <...elevation of point of application of hydrodynamic force

$Fe2 := 0.726 \cdot Pe \cdot y \cdot \frac{Sp_{gate}}{(Structure_Width)}$ $Fe2 = 0 \cdot kN$ <...hydrodynamic force at gate/ stoplogs

$Fe := Fe1 + Fe2$ $Fe = 8.74 \cdot kN$ <...Total hydrodynamic force

$Fec1_h := Wc \cdot \lambda_{hor}$ $Fec1_h = 19.33 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec_v := Wc \cdot \lambda_{vert}$ $Fec_v = 12.89 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec2_h := W_{gate} \cdot \lambda_{hor}$ $Fec2_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Gate/stologs due to earthquake

$Fec_h := Fec1_h + Fec2_h$ $Fec_h = 19.33 \cdot kN$ <... Equivalent Horizontal Inertia Force on pier and gate due to earthquake

$Fec3_h := W_{slab} \cdot \lambda_{hor}$ $Fec3_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$Fec3_v := W_{slab} \cdot \lambda_{vert}$ $Fec3_v = 0 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

$Fec4_h := W_{tower} \cdot \lambda_{hor}$ $Fec4_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Tower due to earthquake

$Fec4_v := W_{tower} \cdot \lambda_{vert}$ $Fec4_v = 0 \cdot kN$ <... Equivalent Vertical Inertia Force on Tower due to earthquake

$F_ANC := Anchor_Force \cdot (unM)$ $Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$F_{hor} := (Fhu - Fhd) + Fe + Fec_h + Fec3_h + Fec4_h + (USHF - DSHF) = 167.3 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - \left(U + Fec_v + Fec3_v + Fec4_v\right) = 287.52 \cdot kN$ <... Resultant of Vertical Forces

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 694.25 \cdot kN \cdot m$ <... Stabilizing Moment

$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$

$$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fe \cdot \left(P_{appl} - F_EL_D\right) \ldots$$
$$+ Fec_h \cdot (Yg - F_EL_D) + Fec_v \cdot (L - Xg) + USHF \cdot (EL_USF - F_EL_D) \ldots$$
$$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$$

<... Overturning Moment (uplift and EQ due to slab/tower inertial force not included)

$Mo_1 = 390.38 \cdot kN \cdot m$

$$Mo_{11} := Fec3_h \cdot (CREST_ELE \cdot unL - F_EL_D) + Fec3_v \cdot (L - Dist_to_F1 \cdot unL) \ldots$$
$$+ Fec4_h \cdot \left(CREST_ELE \cdot unL - F_EL_D + \frac{H_{tower}}{2}\right) + Fec4_v \cdot (L - Dist_to_F2 \cdot unL)$$

<... Overturning Moment due to slab and tower inertial force

$Mo_{11} = 0 \cdot kN \cdot m$

$$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$$

$Mo_2 = 175 \cdot kN \cdot m$

<... Overturning Moment due to Uplift

$$Mo_T := \left(Mo_1 + Mo_{11}\right) + Mo_2$$

<... Total Overturning Moment

$$Mr := Ms - Mo_T$$

<... Resultant Moment

$$x_o := \frac{Mr}{F_{ver}}$$

$x_o = 0.45\,m$

<...Resultant location. Distance from the toe.

$$E := \frac{L}{2} - x_o$$

$E = 1.08\,m$

<...Eccentricity of Resultant

$$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 294.09 \cdot kPa \qquad q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -105.55 \cdot kPa$$

(To consider cohesion and tensile strength)

$$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$$

$L_t = 0\,m$

<...Length of base under tention

$$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$$

$PI = 1.71\,m$

$$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \\ (L - PI) & \text{otherwise} \end{cases} = 1.34\,m$$

<...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 1.34\,m^2$

$L_1 := L_c + L_t = 1.34\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t \right) \right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 287.52 \cdot kN$ $Mr := Ms - Mo_T = 128.66 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 0.45\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.08\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L} \right) = 294.09 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L} \right) = -105.55 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 428.36 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 1.71\,m$ <...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$ <...Stress at Foundation

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha) \right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.25$ <...Factor of Safety Against Sliding.

RESULTS for Loading case 4

Initial point of inflection

$PI = 1.71 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 55.99 \cdot \%$

$EFF_BASE := 1 - \left(\frac{PI}{L}\right)$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 44 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \cdot \% \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant required to be is within base.

Result = "ok"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 428.4 \cdot kPa$

$f := \max(fmin, fmax)$

$x_o = 0.45\,m$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.9 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.9 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

Not Required. The Earthquake load case is used to establish post-earthquake condition of the dam

Summary of Forces and Moments

1) Vertical Forces

$Wc = 386.58 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 0 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$Fec_v = 12.89 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec3_v = 0 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

$U = 86.17 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 386.58 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 287.52 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 162.74 \cdot kN$ <... upstream hydrostatic force

$USHF = 0 \cdot kN$ <...hydrostatic force on gate/stoplogs

$Fhd = 0 \cdot kN$ <... downstream hydrostatic force

$Fe = 8.74 \cdot kN$ <...hydrodynamic force on pier + gate/stoplogs

$Fec_h = 19.33 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec3_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$DSHF = 23.51 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 167.3 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 694.25 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 565.59 \cdot kN \cdot m$ <... overturning moment

$Mr = 128.66 \cdot kN \cdot m$ <... resultant moment

Load Case 5. Post-Earthquake (D+H+S+U_{PQ})

Computation of crack length based on earthquake loading case

$\Sigma M := Ms - Mo_1 + Mo_{11} = 303.87 \cdot kN \cdot m$

$\Sigma W := (Wc + Fa + V1 + V2) - \left(Fec_v + Fec3_v + Fec4_v\right) = 373.69 \cdot kN$

$A3 := Phu \cdot B$

$$T1 := \begin{cases} \dfrac{3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} & \text{if } \dfrac{3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases}$$

<..Uncracked section depth

$$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$$

$T1 = 0.61\,m$

<...Full base cracked when T1=0

$A_c := T1 \cdot B = 0.61\,m^2$

$Cr := \max(T - T1, PI)$ $\quad Cr = 2.44 \cdot m$ <...Cracked section depth

$T1 := T - Cr = 0.61\,m$

$$B5 := \begin{cases} \dfrac{2 \cdot \dfrac{\Sigma W_{IDF} - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases}$$

<...Stress at downstream face

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] + U_d = 155.06 \cdot kN$

$F_{ver_U} := Wc + Fa + V1 + V2$ <... Downwards Vertical Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver} := F_{ver} + N_{ten} = 231.52 \cdot kN$

$F_{hor} := (Fhu - Fhd) + Fi + (USHF - DSHF)$

$$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fi \cdot (EL_ICE - F_EL_D) \ldots + USHF \cdot (EL_USF - F_EL_D) - DSHF \cdot (EL_DSF - F_EL_D) = 288.55 \cdot kN \cdot m$$

$$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2}\right) + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] \cdot \frac{2}{3} \cdot (T1)\right] + U_d \cdot \frac{L}{2}$$

<... Overturning Moment due to increased uplift force

$Mo_T := Mo_1 + Mo_3 = 548 \cdot kN \cdot m$ $\quad Mr := Ms - Mo_T$

$$x_o := \begin{cases} \frac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \frac{T1}{3} & \text{otherwise} \end{cases} \qquad x_o = 0.2\,m$$

$$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.21$$

<...Factor of Safety Against Sliding.

$$EFF_BASE := \frac{T - Cr}{L} \qquad fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases} \qquad fmax := B5$$

RESULTS for Loading case 5

CALCULATED CRACK LENGTH

$Cr = 2.438 \cdot m$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 20.1 \cdot \%$

$$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant within the base .

Result = "ok"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 1133.9 \cdot kPa$

$f := \max(fmin, fmax)$

$x_o = 0.2\,m$

$$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.5 \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$$

<... Allowable compressive strength = 0.5 x f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST OVERTURNING and SLIDING

FSS = 1.21

Safety_against_sliding := "OK" if FSS ≥ 1.1
"to increase dam size" otherwise

Safety_against_sliding = "OK"

Summary of Forces and Moments

1) Vertical Forces

Wc = 386.58 · kN <... weight of concrete in structure

Fa = 0 · kN <... anchor Force

V1 = 0 · kN <... Vertical Force 1 acting on the Downward Direction.

V2 = 0 · kN <... Vertical Force 2 acting on the Downward Direction.

U = 155.06 · kN <... uplift force acting upwards

F_{ver_U} = 386.58 · kN <... resultant of the vertical forces without uplift

F_{ver} = 231.52 · kN <... resultant of the vertical forces with uplift

2) Horizontal Forces

Fhu = 162.74 · kN <... upstream hydrostatic force

USHF = 0 · kN <...hydrostatic force on gate/stoplogs

Fhd = 0 · kN <... downstream hydrostatic force

DSHF = 23.51 · kN <...horizontal force acting on the Upstream Direction

F_{hor} = 139.23 · kN <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

Ms = 694.25 · kN · m <... stabilizing moment

Mo_T = 547.85 · kN · m <... overturning moment

Mr = 146.4 · kN · m <... resultant moment

KGS GROUP

DESIGN CALCULATIONS COVER SHEET

Project No. :	12-0006-028	**Project Name :**	Talbot River Dam (at Lock 39)		
File No. :		**Discipline :**	Civil Engineering		
Calculation Title :	**Spillway**				
Calculation No. :	CIV-002	**Prepared by :**	Y.F.	**Date :**	Mar. 22, 2013
No. of Sheets :	41	**Checked by :**	H.H.	**Date :**	Oct. 8, 2013
Supersedes Calc. No. :		**Approved by :**	H. H.	**Date :**	Oct. 8, 2013

Calculation Description :

The pier has been reviewed against 2007 CDA Dam Safety Guidelines

Related Design Concept :

Stability analysis for the structures is carried out by using "Gravity Method"
Five loading cases are utilized in the analyses based 2007 CDA Guidelines

Reference Codes and Standards :

1.Design of Small Dams, Third Edition, U.S. Government Printing Office, Washington, D.C. 1999.
2. 2007 CDA Dam Safety Guidelines
3. 2009 Parks Canada Directive for Dam Safety Program of Dams and Water-Retaining Structures

ENGINEER'S SEAL

Rev. #	Rev. Description	Rev. Author	Date Revised	Checked by	Approved by	Approved Date

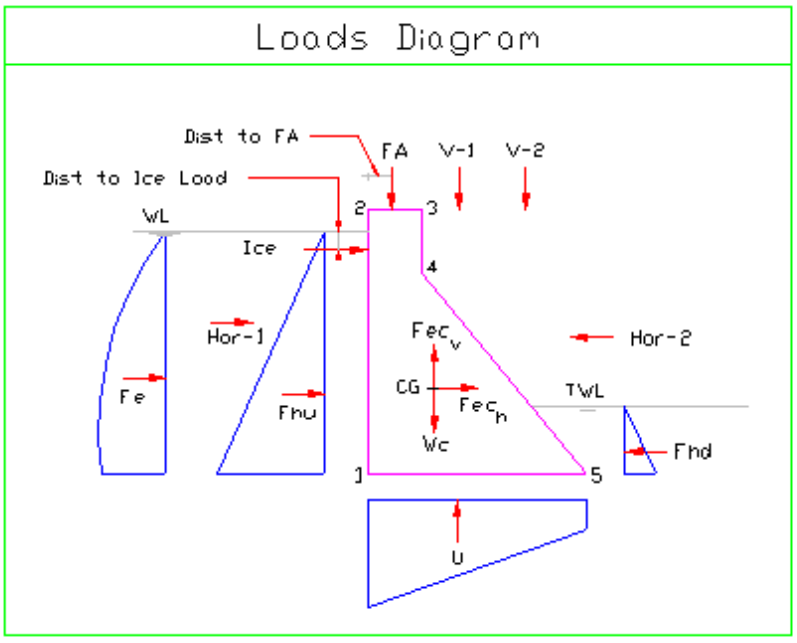
PROGRAM DESCRIPTION SECTION

This Mathcad file can calculate factors of safety against sliding and perform cracked section analysis for any shaped structure. Loading combinations conforms to CDA Dam Safety Guidelines 2007
Features

- metric or imperial units
- ice, anchor, hydrostatic and gravity loads
- crack analysis
- calculates percentage of effective base
- bearing pressures
- volume and position of center of gravity of structure (including voids)
- summary of forces and moments (including resultant and eccentricity)
- additional point loads (e.g.. backfill, vertical hydrostatic, etc.)
- hydrodynamic forces by formulas of Small Dam Design
- equivalent horizontal and vertical inertia forces on structure during earthquake

Assumptions and Limitations

- 2-dimensional
- horizontal base only
- vertical hydrostatic and backfill loads must be input as point loads
- lowest point on structure is level of base
- all vertical point loads are referenced to most upstream point of structure
- Evaluation of cracking during analyses are for unusual and extreme loading cases only.



INPUT SECTION

Unit System

1 - metric
2 - imperial

Select unit system

units := 1 <... select unit system

unL := if(units = 1, 1 · m, 1 · ft)

unM := if(units = 1, 1 · kN, 1 · lb)

Geometry of Structure defined by perimeter coordinates

There is no need to "close" the perimeter ,i.e., so that last point equal to first point

$$X := \begin{pmatrix} 0.000 \\ 0.000 \\ 1.68 \\ 2.31 \\ 3.05 \\ 3.05 \\ 0 \end{pmatrix} \cdot unL \qquad Y := \begin{pmatrix} 224.68 \\ 230.32 \\ 230.32 \\ 227.73 \\ 227.73 \\ 224.68 \\ 224.68 \end{pmatrix} \cdot unL$$

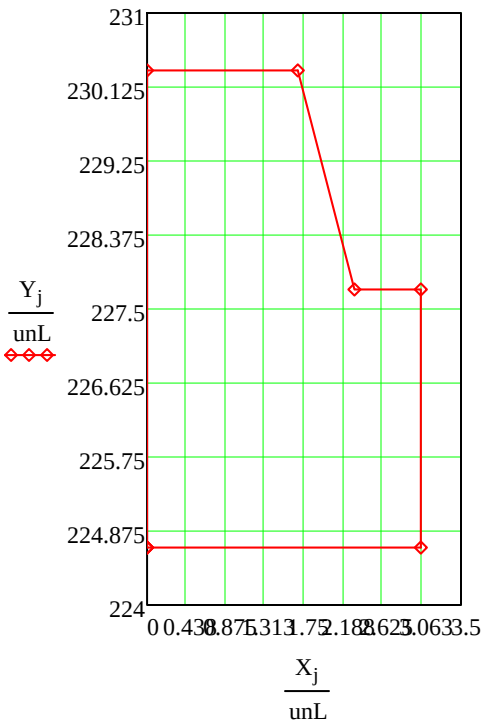
<... input X & Y coordinates

$$maxY := \frac{\max(Y)}{unL} + 5 \qquad minY := \frac{\min(Y)}{unL} - 5 \qquad i := 1 .. length(X)$$

$$maxX := \frac{\max(X)}{unL} + 5 \qquad minX := \frac{\min(X)}{unL} - 5 \qquad j := 1 .. length(X) + 1$$

$$X_{length(X)+1} := X_1 \qquad Y_{length(Y)+1} := Y_1$$

Graphical Representation of Structure defined by X & Y Coordinates



Densities of Water and Concrete, Width of Structure, of Structure defined by perimeter coordinates

$\gamma w := \text{if}\left(\text{units} = 1, 9.81 \cdot \frac{kN}{m^3}, 62.5 \cdot \frac{lb}{ft^3}\right)$ <... water density

$\gamma_{silt} := \text{if}\left(\text{units} = 1, 7.7 \cdot \frac{kN}{m^3}, 49 \cdot \frac{lb}{ft^3}\right)$ <... Silt density

$\gamma_{backfill} := \text{if}\left(\text{units} = 1, 19 \cdot \frac{kN}{m^3}, 120 \cdot \frac{lb}{ft^3}\right)$ <... Backfill density

$\gamma c := \text{if}\left(\text{units} = 1, 23.5 \cdot \frac{kN}{m^3}, 145 \cdot \frac{lb}{ft^3}\right)$ <... concrete density

$\gamma_{timber} := \text{if}\left(\text{units} = 1, 10 \cdot \frac{kN}{m^3}, 62.4 \cdot \frac{lb}{ft^3}\right)$ <... timber density (for Stoplogs)

$\gamma_{steel} := \text{if}\left(\text{units} = 1, 77 \cdot \frac{kN}{m^3}, 490 \cdot \frac{lb}{ft^3}\right)$ <... Steel density

$UNI_{floor} := \text{if}\left(\text{units} = 1, 0.5 \cdot \frac{kN}{m^2}, 10.4 \cdot \frac{lb}{ft^2}\right)$ <... Steel deck unit area weight

$f'_c := \text{if}(\text{units} = 1, 17 \cdot MPa, 2030 \cdot psi)$ <... concrete compressive strength

$f'_{rock} := if(units = 1, 30 \cdot MPa, 4350 \cdot psi)$ <... mass rock compressive strength

$Structure_Width := 1.0$ <... width of structure

$\phi := 36 \cdot deg$ <... angle of internal friction for foundation material

$c := 0 \cdot f'_c$ $c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

Computation of Area and Coordinates of Center of Gravity of Structure

$deltax_i := X_{i+1} - X_i$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot (deltay_i \cdot xplusx_i)$

$Yginc_i := \frac{deltax_i}{8} \cdot \left[(yplusy_i)^2 + \frac{(deltay_i)^2}{3} \right]$ $Xginc_i := \frac{deltay_i}{8} \cdot \left[(xplusx_i)^2 + \frac{(deltax_i)^2}{3} \right]$

$Area := \sum_i Areainc_i$ $Area := if\left[\frac{Area}{unL^2} > 0, Area, Area \cdot (-1) \right]$ <... Area of the Structure (defined by X & Y)

$Xg := \sum_i Xginc_i$ $Xg := \frac{if\left[\frac{Xg}{unL^3} > 0, Xg, Xg \cdot (-1) \right]}{Area}$ <... X coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$Yg := \sum_i Yginc_i$ $Yg := \frac{if\left[\frac{Yg}{unL^3} > 0, Yg, Yg \cdot (-1) \right]}{Area}$ <... Y coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$B := Structure_Width \cdot (unL)$ $L := \max(X) - \min(X) = 3.05\,m$

Computation of Concrete Volume and Weight in Structure

$Vol_conc := Area \cdot B$ $Vol_conc = 14 \cdot m^3$ <... volume of concrete in structure

$Wc := Area \cdot B \cdot \gamma c$ $Wc = 340 \cdot kN$ <... weight of concrete in structure

Load Case 1. Usual Loading Summer Case (D+H+S+U)

US_Water_Level := 230.34 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

CREST_ELE := 230.32 <...Crest Elevation

US_L_ELE := 224.68 <...Lowest Dam Elevation at upstream

DS_L_ELE := 224.68 <...Lowest Dam Elevation at downstream

ROLLWAY_TOP_ELE := 224.68 <...Rollways Top Elevation

$$\alpha := \operatorname{atan}\left(\frac{DS_L_ELE - US_L_ELE}{\frac{L}{unL}}\right) = 0 \cdot deg$$

<...Angle of inclined rock foundation

SF := 2.0 <... Minimum Safety Factor

Deck Slab and Gate weight

$Sp_{gate} := 0$ <... Clear opening for gate/stoplogs

$N_s := 0$ <... Number of Stoplogs

Lsize := 0.305 <... Size of stoplog

$$W_{gate} := \gamma_{timber} \cdot \left[Sp_{gate} + 2(Lsize)\right] \cdot unL \cdot (Lsize \cdot unL)^2 \cdot N_s$$

$W_{gate} = 0 \cdot kN$ <... Weight of gate/stoplogs

$S_{wid} := 4.572 - 0.305$ <... Slab width

$S_{thk} := 0.16$ <... Equivalent slab thickness

$$W_{slab} := \gamma c \cdot Sp_{gate} \cdot S_{wid} \cdot S_{thk} \cdot (unL)^3$$

$W_{slab} = 0 \cdot kN$ <... Weight of slab

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

$$US_Hor_Force := \frac{1}{2} \cdot \gamma w \cdot (US_Water_Level - ROLLWAY_TOP_ELE)^2 \cdot Sp_{gate} \cdot \frac{(unL)^3}{kN}$$

US_Hor_Force = 0 <...Hydrostatic force on Gate / Stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

EL_US_Hor_F = 226.57 <...Elevation of the Hydrostatic force on Gate / Stoplogs

$\phi' := 25 \cdot deg$ <... angle of internal friction for Silt at rest condition

$Ko := 1 - \sin(\phi') = 0.58$

SILT_ELE := 227.73 <... Silt top elevation

$$Silt_Pressure := \frac{1}{2} \cdot (\gamma_{silt}) \cdot Ko \cdot (SILT_ELE \cdot unL - DS_L_ELE \cdot unL)^2 \cdot B = 20.68 \cdot kN$$

$$E_{silt} := DS_L_ELE + \frac{1}{3}(SILT_ELE - DS_L_ELE) = 225.7$$

$\phi 1' := 30 \cdot deg$ <... angle of internal friction for Backfill at rest condition

$Ko1 := 1 - \sin(\phi 1') = 0.5$

Backfill_ELE := 227.73 <... Backfill top elevation

$$Backfill_Pressure := \frac{1}{2} \cdot (\gamma_{backfill}) \cdot Ko1 \cdot (Backfill_ELE \cdot unL - DS_L_ELE \cdot unL)^2 \cdot B = 44.19 \cdot kN$$

$soil_pressure := Silt_Pressure - Backfill_Pressure = -23.51 \cdot kN$

$$E_{backfillt} := DS_L_ELE + \frac{1}{3}(Backfill_ELE - DS_L_ELE) = 225.7$$

$$E_{eq} := \frac{Silt_Pressure \cdot \left(\frac{SILT_ELE - DS_L_ELE}{3}\right) - Backfill_Pressure \cdot \left(\frac{Backfill_ELE - DS_L_ELE}{3}\right)}{soil_pressure} \ldots = 225.7$$
$$+ DS_L_ELE$$

$$DS_Hor_Force := \frac{-soil_pressure}{unM} = 23.51$$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$$EL_DS_Hor_F := E_{eq}$$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$$Vert_Force_1 := \frac{W_{slab}}{unM}$$

<... Weight of Deck slab

$$Dist_to_F1 := 0$$

<... distance from most upstream point of structure to point of application of vertical force

$$Vert_Force_2 := 0$$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$$Dist_to_F2 := 0$$

<... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$$WL := US_Water_Level \cdot (unL) \qquad F_EL := US_L_ELE \cdot (unL) \qquad F_EL_D := DS_L_ELE \cdot (unL)$$

$$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL) \qquad F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$$

$$F_ANC := Anchor_Force \cdot (unM) \qquad Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$$

$$V1 := Vert_Force_1 \cdot (unM) \qquad Dist_V1 := L - Dist_to_F1 \cdot (unL)$$

$$V2 := Vert_Force_2 \cdot (unM) \qquad Dist_V2 := L - Dist_to_F2 \cdot (unL)$$

$$USHF := US_Hor_Force \cdot (unM) \qquad EL_USF := EL_US_Hor_F \cdot (unL)$$

$DSHF := DS_Hor_Force \cdot (unM)$ $EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 157 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 84.68 \cdot kN$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF - DSHF) = 133.63 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ ... = 581.6 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fi \cdot (EL_ICE - F_EL_D) \ ... = 272.56 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) - DSHF \cdot (EL_DSF - F_EL_D)$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ $x_o = 0.54\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ $E = 0.99\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 246.62 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -79.17 \cdot kPa$

$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$ Calculation_to_be = "stoped"

(To ensure un cracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 1.44\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \\ (L - PI) & \text{otherwise} \end{cases} = 1.61\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 1.61\,m^2$

$L_1 := L_c + L_t = 1.61\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 255.36 \cdot kN$ $Mr := Ms - Mo_T = 136.86 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 0.54\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 0.99\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 246.62 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -79.17 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 317.63 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$

<...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 1.44\,m$ <...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$ <...Stress at Foundation

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.39$ <...Factor of Safety Against Sliding.

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

$\Sigma M := Ms - Mo_1$

$\Sigma W := Wc + Fa + V1 + V2$

$A3 := Phu \cdot B$ $T := L$

$T1 := \begin{cases} \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} & \text{if } \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases}$ $T1 = 0.89\,m$ <...Uncracked section depth

$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$ $T1 = 0.89\,m$ <...Full base cracked

$$B5 := \begin{cases} \dfrac{2 \cdot \dfrac{\Sigma W - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases} \qquad B5 = 438.01 \cdot kPa$$

<...Stress at downstream face

$Cr := T - T1 \qquad Cr = 2.16 \cdot m$

<...Cracked section depth

$T1 := T - Cr = 0.89\,m$

$A_c := B \cdot T1 = 0.89\,m^2$

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] + U_d = 144.57 \cdot kN$

$F_{ver} := Wc + Fa + V1 + V2 - U = 195.46 \cdot kN$

$F_{ver} := F_{ver} + N_{ten} = 195.46 \cdot kN$

$$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2}\right) + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] \cdot \frac{2}{3} \cdot (T1)\right] + U_d \cdot \frac{L}{2} = 251 \cdot kN \cdot m$$

$Mo_T := Mo_1 + Mo_3 = 523 \cdot kN \cdot m \qquad Mr := Ms - Mo_T$

$$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases} \qquad fmax := B5$$

$$x_o := \begin{cases} \dfrac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \dfrac{T1}{3} & \text{otherwise} \end{cases} \qquad x_o = 0.3\,m$$

$$EFF_BASE := \frac{T1}{L}$$

$$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.06$$

<...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 1

Initial point of inflection

$PI = 1.44 \cdot m$

Initial point of inflection as % of the base

$\dfrac{PI}{L} = 47.28 \cdot \%$

$$Result := \begin{cases} \text{"ok"} & \text{if } E \leq \dfrac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

CALCULATED CRACK LENGTH | **% OF CRACKED BASE** | **% OF EFFECTIVE BASE** (portion not cracked)

$Cr = 2.158 \cdot m$ | $\frac{Cr}{T} = 70.74 \cdot \%$ | $EFF_BASE = 29.26 \cdot \%$

$q_{1min} = -79.2 \cdot kPa$

When c is taken into account

$$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \geq \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$ $fmax = 438.01 \cdot kPa$

$x_o = 0.3\,m$

<...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.3 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

$FSS = 1.06$

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \geq 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam

When c is taken into account

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \geq SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam s

Summary of Forces and Moments

1) Vertical Forces

$Wc = 340.03 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 0 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U = 144.57 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 340.03 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 195.46 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 157.13 \cdot kN$ <... upstream hydrostatic force

$Fhd = 0 \cdot kN$ <... downstream hydrostatic force

$Fi = 0 \cdot kN$ <... Ice Force

$USHF = 0 \cdot kN$ <...horizontal force acting on the Downstream Direction

$DSHF = 23.51 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 133.63 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 581.6 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 523.45 \cdot kN \cdot m$ <... overturning moment

$Mr = 58.15 \cdot kN \cdot m$ <... resultant moment

Load Case 2. Usual Loading Winter Case (D+H+S+U+I)

US_Water_Level := 230.04 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

SF := 2.0 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 77 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0.3 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

$$US_Hor_Force := \frac{1}{2} \cdot \gamma w \cdot (US_Water_Level - ROLLWAY_TOP_ELE)^2 \cdot Sp_{gate} \cdot \frac{(unL)^3}{kN}$$

US_Hor_Force = 0 <...Hydrostatic force on Gate / Stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

EL_US_Hor_F = 226.47 <...Elevation of the Hydrostatic force on Gate / Stoplogs

US_Hor_Force1 := Ice_Load · Sp_{gate} US_Hor_Force1 = 0 <... Ice Load on gate / stoplogs

EL_US_Hor_F1 := US_Water_Level − Dist_WL_to_Ice_Load

EL_US_Hor_F1 = 229.74 <...Elevation of the ice load

$$DS_Hor_Force := \frac{-soil_pressure}{unM} = 23.51$$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{ec}$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$$Vert_Force_1 := \frac{W_{slab}}{unM}$$

<... Weight of Deck Slab

$Dist_to_F1 := 0$

<... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := 0$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := 0$

<... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$

$F_EL := US_L_ELE \cdot (unL)$

$F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$

$F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$

$EL_USF := EL_US_Hor_F \cdot (unL)$

$USHF1 := US_Hor_Force1 \cdot (unM)$

$EL_USF1 := EL_US_Hor_F1 \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$

$EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$

$TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 140.92 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Net uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Net uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF + USHF1 - DSHF)$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ ... = 581.6 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fi \cdot (EL_ICE - F_EL_D) \ ... = 617.49 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) + USHF1 \cdot (EL_USF1 - F_EL_D) \ ...$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 469.1 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -298.71 \cdot kPa$

$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$ Calculation_to_be = "stoped"

(To ensure uncracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 5.35\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases}$ $= -2.3\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 0\,m^2$

$L_1 := L_c + L_t = -2.3\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 259.85 \cdot kN$ $Mr := Ms - Mo_T = -198.94 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = -0.77\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 2.29\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 469.1 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -298.71 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = -226.26 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 5.35\,m$ <...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } fmin \leq 0 \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 5.35\,m$ <...Point of inflection of stresses at foundation from heel

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.97$ <...Factor of Safety Against Sliding.

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

$\Sigma M := Ms - Mo_1$

$\Sigma W := Wc + Fa + V1 + V2$

$A3 := Phu \cdot B$ $T := L$

$T1 := \begin{cases} 3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right) & \text{if } \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ \Sigma W - A3 \cdot T & \\ T & \text{otherwise} \end{cases}$ $T1 = -4.68\,m$ <...Uncracked section depth

$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$ $T1 = 0\,m$ <...Full base cracked

$B5 := \begin{cases} \dfrac{2 \cdot \dfrac{\Sigma W - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases}$ $B5 = 3593210952.58 \cdot kPa$ <...Stress at downstream face

$Cr := T - T1$ $Cr = 3.05 \cdot m$ <...Cracked section depth

$T1 := T - Cr = 0\,m$ $A_c := B \cdot T1 = 0\,m^2$

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] + U_d = 160.37 \cdot kN$

$F_{ver} := Wc + Fa + V1 + V2 - U = 179.66 \cdot kN$

$F_{ver} := F_{ver} + N_{ten} = 179.66 \cdot kN$

$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \dfrac{Cr}{2}\right) + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] \cdot \dfrac{2}{3} \cdot (T1)\right] + U_d \cdot \dfrac{L}{2} = 245 \cdot kN \cdot m$

$Mo_T := Mo_1 + Mo_3 = 862 \cdot kN \cdot m$ $Ms := Ms + M_{ten}$ $Mr := Ms - Mo_T$

$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$ $fmax := B5$ $x_o := \begin{cases} \dfrac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \dfrac{T1}{3} & \text{otherwise} \end{cases}$ $x_o = 0\,m$

$EFF_BASE := \dfrac{T1}{L}$

$FSS := \dfrac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.67$ <...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 2

Initial point of inflection

$PI = 5.35 \cdot m$

Initial point of inflection as % of the base

$\dfrac{PI}{L} = 175.31 \cdot \%$

$$Result := \begin{cases} \text{"ok"} & \text{if } E \le \frac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

CALCULATED CRACK LENGTH **% OF CRACKED BASE** **% OF EFFECTIVE BASE** (portion not cracked)

$Cr = 3.05 \cdot m$ $\frac{Cr}{T} = 100 \cdot \%$ $EFF_BASE = 3.28 \times 10^{-6} \cdot \%$

$q_{1min} = -298.7 \cdot kPa$

When c is taken into account

$$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \ge \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$ $fmax = 3.59 \times 10^{9} \cdot kPa$

$x_o = 0\,m$

<...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \le 0.3 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "not good"

SAFETY FACTOR AGAINST SLIDING

$FSS = 0.67$

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam

When c is taken into account

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam s

Summary of Forces and Moments

1) Vertical Forces

$Wc = 340.03 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 0 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U = 160.37 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 340.03 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 179.66 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 140.92 \cdot kN$ <... upstream hydrostatic force

$Fhd = 0 \cdot kN$ <... downstream hydrostatic force

$Fi = 77 \cdot kN$ <... Ice Force

$USHF = 0 \cdot kN$ <...horizontal force **due to hydrostatic pressure from stoplogs**

$USHF1 = 0 \cdot kN$ <...horizontal force **due to ice pressure from stoplogs**

$DSHF = 23.51 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 194.41 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 581.6 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 862.06 \cdot kN \cdot m$ <... overturning moment

$Mr = -280.47 \cdot kN \cdot m$ <... resultant moment

Load Case 3. Unusual Loading IDF ($D+H_{IDF}+S+U_{IDF}$)

IDF_Level := 231.03 <... IDF level

Tailwater_Level_IDF := 230.44 <... tailwater level at IDF

SF := 1.5 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0

- Drag Force

<... Horizontal Force acting on the Downstream Direction. For instance, this force can be used to input effect of soil pressure on the upstream side of Structure

EL_US_Hor_F := 226 <...Elevation at which the Upstream Horizontal Force is applied to the Structure

$DS_Hor_Force := \frac{-Silt_Pressure}{unM} = -20.68$ Silt pressure

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{silt}$ <...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{slab}}{unM}$ <... Weight of Deck Slab

$Dist_to_F1 := 0$ <... distance from most upstream point of structure to point of application of vertical force

$h1top := IDF_Level - CREST_ELE = 0.71$ <... height of the tailwater

$W_{topwater} := \frac{B}{unL} \cdot h1top \cdot \frac{L}{unL} \cdot \frac{\gamma w}{\left(\frac{kN}{m^3}\right)} = 21.24$ <... Weight of tailwater on dam

$L_{topwater} := \frac{\left(\frac{L}{unL}\right)}{2} = 1.53$ <... Moment arm of Weight of tailwater to toe

$\tan\theta := 0$ <... Downstream face slope

$h1 := Tailwater_Level_IDF - DS_L_ELE = 5.76$ <... height of the tailwater

$W_{tailwater} := \frac{1}{2} \cdot \frac{B}{unL} \cdot (h1)^2 \cdot \tan\theta \cdot \frac{\gamma w}{\left(\frac{kN}{m^3}\right)} = 0$ <... Weight of tailwater on dam

$L_{tailwater} := \frac{1}{3} \cdot h1 \cdot \tan\theta = 0$ <... Moment arm of Weight of tailwater to toe

$Vert_Force_2 := W_{topwater} + W_{tailwater} = 21.24$ <... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := \frac{L}{unL} - L_{topwater} = 1.53$ <... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL_{IDF} := IDF_Level \cdot (unL)$

$TWL_{IDF} := Tailwater_Level_IDF \cdot (unL)$

$F_ANC := Anchor_Force \cdot (unM)$ $Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$

$EL_USF := EL_US_Hor_F \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$

$EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$

$TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu_{IDF} := (WL_{IDF} - F_EL) \cdot \gamma w$

$F_EL = 224.68\,m$ <... upstream hydrostatic pressure

$Phd_{IDF} := (TWL_{IDF} - F_EL_D) \cdot \gamma w$

$F_EL_D = 224.68\,m$ <... downstream hydrostatic pressure

$Fhu_{IDF} := Phu_{IDF} \cdot \frac{WL_{IDF} - F_EL}{2} \cdot B$

$Fhu_{IDF} = 197.78 \cdot kN$ <... upstream hydrostatic force

$Fhd_{IDF} := Phd_{IDF} \cdot \frac{TWL_{IDF} - F_EL_D}{2} \cdot B$

$Fhd_{IDF} = 162.74 \cdot kN$ <... downstream hydrostatic force

$Pu_{IDF} := Phu_{IDF} - Phd_{IDF}$ <... Uplift pressure at upstream end

$U_{uIDF} := \frac{(Pu_{IDF} \cdot Ueff) \cdot L \cdot B}{2}$

$U_{uIDF} = 8.83 \cdot kN$ <... Uplift force due to upstream head water

$U_{dIDF} := Phd_{IDF} \cdot L \cdot B$

$U_{dIDF} = 172.34 \cdot kN$ <... Uplift force due to downstream tailwater

$U_{IDF} := U_{uIDF} + U_{dIDF}$ <... Total Uplift force

$F_{horIDF} := (Fhu_{IDF} - Fhd_{IDF}) + (USHF - DSHF)$ <... Resultant of Horizontal Forces

$F_{verIDF} := Wc + Fa + V1 + V2 - U_{IDF}$ <... Resultant of Vertical Forces

$Ms_{IDF} := Wc \cdot (L - Xg) + Fhd_{IDF} \cdot \left(\frac{TWL_{IDF} - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots + V1 \cdot Dist_V1 + V2 \cdot Dist_V2 = 926.45 \cdot kN \cdot m$

<... Stabilizing Moment

$Mo_{1IDF} := Fhu_{IDF} \cdot \left[\frac{WL_{IDF} - F_EL}{3} + (F_EL_D - F_EL)\right] + USHF \cdot (EL_USF - F_EL_D) + [-DSHF \cdot (EL_DSF - F_EL_D)] \ldots = 439.66 \cdot kN \cdot m$

<... Overturning Moment (uplift not included)

$Mo_{2IDF} := U_{uIDF} \cdot \left(\frac{2 \cdot L}{3}\right) + U_{dIDF} \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_{TIDF} := Mo_{1IDF} + Mo_{2IDF}$ <... Total Overturning Moment

$Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$ <... Resultant Moment

$x_{oIDF} := \frac{Mr_{IDF}}{F_{verIDF}}$ $x_{oIDF} = 1.14\,m$ <...Resultant location. Distance from the toe.

$E_{IDF} := \frac{L}{2} - x_{oIDF}$ $E_{IDF} = 0.38\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E_{IDF}}{L}\right) = 103.33 \cdot kPa$ $q_{1min} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E_{IDF}}{L}\right) = 14.77 \cdot kPa$

(To consider cohesion and tensile

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \quad = 0\,m \\ 0 \cdot m & \text{otherwise} \end{cases}$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \quad = 3.05\,m \\ (L - PI) & \text{otherwise} \end{cases}$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \quad = 3.05\,m^2 \\ 0 & \text{otherwise} \end{cases}$ $L_1 := L_c + L_t = 3.05\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms_{IDF} := Ms_{IDF} + M_{ten}$ $F_{verIDF} := F_{verIDF} + N_{ten} = 180.11 \cdot kN$ $Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$

$x_{oIDF} := \frac{Mr_{IDF}}{F_{verIDF}}$ $x_{oIDF} = 1.14\,m$

$E_{IDF} := \frac{L}{2} - x_{oIDF}$ $E_{IDF} = 0.38\,m$

$q_{1max} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E_{IDF}}{L}\right) = 103.33 \cdot kPa$ $q_{1min} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E_{IDF}}{L}\right) = 14.77 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{verIDF}}{3 \cdot x_{oIDF} \cdot B} & \text{otherwise} \end{cases} = 103.33 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 14.77 \cdot kPa$

<...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot (x_{oIDF}) & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 0\,m$

<...Point of inflection of stresses at foundation from heel

$FSS := \frac{(F_{verIDF} \cdot \cos(\alpha) + F_{horIDF} \cdot \sin(\alpha)) \cdot \tan(\phi) + c \cdot A_c}{F_{horIDF} \cdot \cos(\alpha) - F_{verIDF} \cdot \sin(\alpha)} = 2.35$

<...Factor of Safety Against Sliding.

STRESSES AT FOUNDATION

fmin = 14.8 · kPa

fmax = 103.3 · kPa

initial point of inflection

$PI = 0 \cdot m$

initial point of inflection as % of the base

$\frac{PI}{L} = 0 \cdot \%$

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

Static Method of Analysis.-*Assumptions associated with a static, cracked-section analysis are:*
1 Stress distribution along a horizontal section,without uplift, varies linearly between upstream and downstream faces.
2 Once a crack occurs, uplift pressure equivalent to reservoir pressure above the crack exists throughout the entire crack depth. Uplift is then assumed to vary linearly from crack tip to tailwater pressure at downstream face.
3 Crack penetrates to point of zero stress. This assumes no tensile strength at crack tip, which means the entire untracked length is entirely in compression.

Refer to "Design of small dam" P 331-332

$\Sigma M_{IDF} := Ms_{IDF} - Mo_{1IDF}$ $\Sigma W_{IDF} := Wc + Fa + V1 + V2$ $A3 := Phu_{IDF} \cdot B$

$T := L$

$$T1 := \begin{cases} \dfrac{3 \cdot \left(\Sigma M_{IDF} - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W_{IDF} - A3 \cdot T} & \text{if } \dfrac{3 \cdot \left(\Sigma M_{IDF} - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W_{IDF} - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases}$$

<...Un cracked section depth

$T1 = 3.05\,m$

$$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$$

<...Full base cracked

$$B5 := \begin{cases} \dfrac{2 \cdot \dfrac{\Sigma W_{IDF} - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases}$$

<...Stress at downstream face

$Cr := T - T1$ $\quad Cr = 0 \cdot m$ <...Cracked section depth

$T1 := T - Cr = 3.05\,m$ $\quad A_c := B \cdot T1 = 3.05\,m^2$

$$U_{IDF} := \left(Pu_{IDF} \cdot Ueff\right) \cdot Cr \cdot B + \left(Pu_{IDF} \cdot Ueff\right) \cdot \left[\frac{(T1) \cdot B}{2}\right] + U_{dIDF}$$

$F_{ver_UIDF} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces without uplift

<... Resultant of Vertical Forces

$F_{verIDF} := Wc + Fa + V1 + V2 - U_{IDF}$

$$Mo_{3IDF} := \left[\left(Pu_{IDF} \cdot Ueff\right) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2}\right) + \left(Pu_{IDF} \cdot Ueff\right) \cdot \left[\frac{(T1) \cdot B}{2}\right] \cdot \frac{2}{3} \cdot (T1)\right] + U_{dIDF} \cdot \frac{L}{2}$$

$Mo_{TIDF} := Mo_{1IDF} + Mo_{3IDF}$ $\quad Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$

$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$ $\quad fmax := B5$ $\quad EFF_BASE := \dfrac{T1}{L}$

$$x_{oIDF} := \begin{cases} \dfrac{Mr_{IDF}}{F_{verIDF}} & \text{if } Cr = 0 \\ \dfrac{T1}{3} & \text{otherwise} \end{cases}$$

$x_{oIDF} = 1.14\,m$

$$FSS := \frac{\left(F_{verIDF} \cdot \cos(\alpha) + F_{horIDF} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{horIDF} \cdot \cos(\alpha) - F_{verIDF} \cdot \sin(\alpha)} = 2.35$$

<...Factor of Safety Against Sliding.

RESULTS for Loading Case 3

Initial point of inflection

$PI = 0 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 0 \cdot \%$

CALCULATED CRACK LENGTH

$Cr = 0 \cdot m$

% OF CRACKED BASE

$\frac{Cr}{T} = 0 \cdot \%$

% OF EFFECTIVE BASE (portion not cracked)

$EFF_BASE = 100 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 75 \cdot \% \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant required to be is within base.

Result = "ok"

$q_{1min} = 14.8 \cdot kPa$

When c is taken into account

$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \geq \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "ok"

STRESSES AT FOUNDATION

fmin = 14.8 · kPa

fmax = 103.3 · kPa

$f := \max(fmin, fmax)$

$x_{oIDF} = 1.144\ m$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.5 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.5 x f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST OVERTURNING and SLIDING

FSS = 2.35

$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \geq 1.3 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$

Safety_against_sliding = "OK"

When c is taken into account

Safety_against_sliding := "OK" if FSS ≥ SF
"to increase dam size" otherwise

Safety_against_sliding = "OK"

Summary of Forces and Moments

1) Vertical Forces

$Wc = 340.03 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 0 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 21.24 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U_{IDF} = 181.17 \cdot kN$ <... uplift force acting upwards

$F_{ver_UIDF} = 361.28 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{verIDF} = 180.11 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu_{IDF} = 197.78 \cdot kN$ <... upstream hydrostatic force

$Fhd_{IDF} = 162.74 \cdot kN$ <... downstream hydrostatic force

$USHF = 0 \cdot kN$ <...Drag Force

$DSHF = -20.68 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{horIDF} = 55.72 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms_{IDF} = 926.45 \cdot kN \cdot m$ <... stabilizing moment

$Mo_{TIDF} = 720.43 \cdot kN \cdot m$ <... overturning moment

$Mr_{IDF} = 206.02 \cdot kN \cdot m$ <... resultant moment

Load Case 4. Extreme Loading Earthquake (D+H+S+Q+U_Q)

Horizontal & Vertical Components of the earthquake Intensity and Coefficient of Hydrodynamic Pressure Distribution

$\lambda_{hor} := 0.05$ <... **horizontal** component of earthquake intensity = ratio of earthquake acceleration to acceleration due to gravity

$\lambda_{vert} := \frac{2}{3} \cdot \lambda_{hor}$ <... **vertical** component of earthquake intensity

<... from owner's requirement 21.2.5.1

$C_m := 0.74$ <... from fig 8-6, page 325 of "Design of Small Dams", for a structure with **vertical face**

US_Water_Level := 230.34 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

$f_t := 0$ **To be assumed for EQ case due to loss of cohesive bond** <... tensile strength at interface of concrete /rock

SF := 1.3 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **acceleration,** i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

$$US_Hor_Force := \frac{1}{2} \cdot \gamma w \cdot (US_Water_Level - ROLLWAY_TOP_ELE)^2 \cdot Sp_{gate} \cdot \frac{(unL)^3}{kN}$$

US_Hor_Force = 0 <...Hydrostatic on stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

<...Elevation at which the Upstream Horizontal Force is applied to the Structure

EL_US_Hor_F = 226.57

$$DS_Hor_Force := \frac{-soil_pressure}{unM} = 23.51$$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{ed}$ <...Elevation at which the Downstream Horizontal Force is applied to the Structure

$W_{tower} := 0$ <...Weight of tower

$H_{tower} := 0$ <...Height of tower

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **acceleration,** i.e., metric units, **Vertical Forces** will be in **kN**

$$Vert_Force_1 := \frac{W_{slab}}{unM}$$ <... Weight of Deck Slab

Dist_to_F1 := 0 <... distance from most upstream point of structure to point of application of vertical force

Vert_Force_2 := 0 <... Vertical Force acting on the Downward Direction. It is a stabilizing force

Dist_to_F2 := 0 <... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$ $F_EL := US_L_ELE \cdot (unL)$ $F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$ $F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

$F_ANC := Anchor_Force \cdot (unM)$ $Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$ $EL_USF := EL_US_Hor_F \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$ $EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces and Uplift

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 157.13 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d$ <... Total Uplift force

Computation of Hydrodynamic Force Due to Earthquake

$h := US_Water_Level \cdot (unL) - F_EL$ $h = 5.66\,m$ <...total depth of the reservoir at section being analyses

$y := h$ <....vertical distance from reservoir surface to the elevation in question

$C := \frac{C_m}{2} \cdot \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right) + \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right)\right]^{0.5}\right]$ <...Coefficient of Hydrodynamic Pressure Distribution

$Pe := C \cdot \lambda_{hor} \cdot \gamma w \cdot h \cdot Structure_Width \cdot (unL)$ $Pe = 2.05 \cdot \frac{kN}{m}$ <...hydrodynamic pressure

$Fe1 := 0.726 \cdot Pe \cdot y$ $Fe1 = 8.4 \cdot kN$ <...hydrodynamic force at pier

$Me := 0.299 \cdot Pe \cdot y^2$ $Me = 20 \cdot kN \cdot m$ <...moment at the base of the structure due to hydrodynamic force

$P_{appl} := \begin{cases} F_EL + \frac{Me}{Fe1} & \text{if } Fe1 > 0kN \\ 0 \cdot m & \text{otherwise} \end{cases}$ $P_{appl} = 227.011 \cdot m$ <...elevation of point of application of hydrodynamic force

$Fe2 := 0.726 \cdot Pe \cdot y \cdot \frac{Sp_{gate}}{(Structure_Width)}$ $Fe2 = 0 \cdot kN$ <...hydrodynamic force at gate/ stoplogs

$Fe := Fe1 + Fe2$ $Fe = 8.44 \cdot kN$ <...Total hydrodynamic force

$Fec1_h := Wc \cdot \lambda_{hor}$ $Fec1_h = 17 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec_v := Wc \cdot \lambda_{vert}$ $Fec_v = 11.33 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec2_h := W_{gate} \cdot \lambda_{hor}$ $Fec2_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Gate/stologs due to earthquake

$Fec_h := Fec1_h + Fec2_h$ $Fec_h = 17 \cdot kN$ <... Equivalent Horizontal Inertia Force on pier and gate due to earthquake

$Fec3_h := W_{slab} \cdot \lambda_{hor}$ $Fec3_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$Fec3_v := W_{slab} \cdot \lambda_{vert}$ $Fec3_v = 0 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

$Fec4_h := W_{tower} \cdot \lambda_{hor}$ $Fec4_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Tower due to earthquake

$Fec4_v := W_{tower} \cdot \lambda_{vert}$ $Fec4_v = 0 \cdot kN$ <... Equivalent Vertical Inertia Force on Tower due to earthquake

$F_ANC := Anchor_Force \cdot (unM)$ $Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$F_{hor} := (Fhu - Fhd) + Fe + Fec_h + Fec3_h + Fec4_h + (USHF - DSHF) = 159.07 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - \left(U + Fec_v + Fec3_v + Fec4_v\right) = 244.02 \cdot kN$ <... Resultant of Vertical Forces

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 581.6 \cdot kN \cdot m$ <... Stabilizing Moment

$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$

$$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fe \cdot \left(P_{appl} - F_EL_D\right) \ldots$$
$$+ Fec_h \cdot (Yg - F_EL_D) + Fec_v \cdot (L - Xg) + USHF \cdot (EL_USF - F_EL_D) \ldots$$
$$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$$

<... Overturning Moment (uplift and EQ due to slab/tower inertial force not included)

$Mo_1 = 354.26 \cdot kN \cdot m$

$$Mo_{11} := Fec3_h \cdot (CREST_ELE \cdot unL - F_EL_D) + Fec3_v \cdot (L - Dist_to_F1 \cdot unL) \ldots$$
$$+ Fec4_h \cdot \left(CREST_ELE \cdot unL - F_EL_D + \frac{H_{tower}}{2}\right) + Fec4_v \cdot (L - Dist_to_F2 \cdot unL)$$

<... Overturning Moment due to slab and tower inertial force

$Mo_{11} = 0 \cdot kN \cdot m$

$$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$$

$Mo_2 = 172 \cdot kN \cdot m$

<... Overturning Moment due to Uplift

$$Mo_T := \left(Mo_1 + Mo_{11}\right) + Mo_2$$

<... Total Overturning Moment

$$Mr := Ms - Mo_T$$

<... Resultant Moment

$$x_o := \frac{Mr}{F_{ver}}$$

$x_o = 0.23\,m$

<...Resultant location. Distance from the toe.

$$E := \frac{L}{2} - x_o$$

$E = 1.3\,m$

<...Eccentricity of Resultant

$$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 284.45 \cdot kPa \qquad q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -124.44 \cdot kPa$$

(To consider cohesion and tensile strength)

$$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$$

$L_t = 0\,m$

<...Length of base under tention

$$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$$

$PI = 2.37\,m$

$$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \\ (L - PI) & \text{otherwise} \end{cases} = 0.68\,m$$

<...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 0.68\,m^2$

$L_1 := L_c + L_t = 0.68\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t \right) \right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 244.02 \cdot kN$ $Mr := Ms - Mo_T = 55.16 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 0.23\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.3\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L} \right) = 284.45 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L} \right) = -124.44 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 719.64 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 2.37\,m$ <...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$ <...Stress at Foundation

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha) \right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.11$ <...Factor of Safety Against Sliding.

RESULTS for Loading case 4

Initial point of inflection

$PI = 2.37 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 77.76 \cdot \%$

$EFF_BASE := 1 - \left(\frac{PI}{L}\right)$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 22 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \cdot \% \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant required to be is within base.

Result = "ok"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 719.6 \cdot kPa$

$f := \max(fmin, fmax)$

$x_o = 0.23\,m$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.9 \cdot \min\left(f'_c, f'_{rock}\right) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.9 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

Not Required. The Earthquake load case is used to establish post-earthquake condition of the dam

Summary of Forces and Moments

1) Vertical Forces

$Wc = 340.03 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 0 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$Fec_v = 11.33 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec3_v = 0 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

$U = 84.68 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 340.03 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 244.02 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 157.13 \cdot kN$ <... upstream hydrostatic force

$USHF = 0 \cdot kN$ <...hydrostatic force on gate/stoplogs

$Fhd = 0 \cdot kN$ <... downstream hydrostatic force

$Fe = 8.44 \cdot kN$ <...hydrodynamic force on pier + gate/stoplogs

$Fec_h = 17 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec3_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$DSHF = 23.51 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 159.07 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 581.6 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 526.43 \cdot kN \cdot m$ <... overturning moment

$Mr = 55.16 \cdot kN \cdot m$ <... resultant moment

Load Case 5. Post-Earthquake (D+H+S+U_{PQ})

Computation of crack length based on earthquake loading case

$\Sigma M := Ms - Mo_1 + Mo_{11} = 227.34 \cdot kN \cdot m$

$\Sigma W := (Wc + Fa + V1 + V2) - \left(Fec_v + Fec3_v + Fec4_v\right) = 328.7 \cdot kN$

$A3 := Phu \cdot B$

$$T1 := \begin{cases} \dfrac{3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} & \text{if } \dfrac{3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases}$$

<..Uncracked section depth

$$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases} \qquad T1 = 0\,m$$

<...Full base cracked when T1=0

$A_c := T1 \cdot B = 0\,m^2$

$Cr := \max(T - T1, PI) \qquad Cr = 3.05 \cdot m$

<...Cracked section depth

$T1 := T - Cr = 0\,m$

$$B5 := \begin{cases} \dfrac{2 \cdot \dfrac{\Sigma W_{IDF} - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases}$$

<...Stress at downstream face

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] + U_d = 169.35 \cdot kN$

$F_{ver_U} := Wc + Fa + V1 + V2$

<... Downwards Vertical Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$

<... Resultant of Vertical Forces

$F_{hor} := (Fhu - Fhd) + Fi + (USHF - DSHF)$

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fi \cdot (EL_ICE - F_EL_D) \ldots = 272.56 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) - DSHF \cdot (EL_DSF - F_EL_D)$

$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2}\right) + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] \cdot \frac{2}{3} \cdot (T1)\right] + U_d \cdot \frac{L}{2}$

<... Overturning Moment due to increased uplift force

$Mo_T := Mo_1 + Mo_3 = 531 \cdot kN \cdot m$ $\quad Mr := Ms - Mo_T$

$x_o := \begin{cases} \frac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \frac{T1}{3} & \text{otherwise} \end{cases}$ $\quad x_o = 0\,m$

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.93$

<...Factor of Safety Against Sliding.

$EFF_BASE := \frac{T - Cr}{L}$ $\quad fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$ $\quad fmax := B5$

RESULTS for Loading case 5

CALCULATED CRACK LENGTH

$Cr = 3.05 \cdot m$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 3.3 \times 10^{-6} \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant within the base .

Result = "ok"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 3.8 \times 10^{9} \cdot kPa$

$f := \max(fmin, fmax)$

$x_o = 0\,m$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.5 \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength = 0.5 x f'_c (and f'_{rock})

Normal_stresses = "not good"

SAFETY FACTOR AGAINST OVERTURNING and SLIDING

FSS = 0.93

Safety_against_sliding := "OK" if FSS ≥ 1.1
"to increase dam size" otherwise

Safety_against_sliding = "to increase dam size

Summary of Forces and Moments

1) Vertical Forces

Wc = 340.03 · kN <... weight of concrete in structure

Fa = 0 · kN <... anchor Force

V1 = 0 · kN <... Vertical Force 1 acting on the Downward Direction.

V2 = 0 · kN <... Vertical Force 2 acting on the Downward Direction.

U = 169.35 · kN <... uplift force acting upwards

F_{ver_U} = 340.03 · kN <... resultant of the vertical forces without uplift

F_{ver} = 170.68 · kN <... resultant of the vertical forces with uplift

2) Horizontal Forces

Fhu = 157.13 · kN <... upstream hydrostatic force

USHF = 0 · kN <...hydrostatic force on gate/stoplogs

Fhd = 0 · kN <... downstream hydrostatic force

DSHF = 23.51 · kN <...horizontal force acting on the Upstream Direction

F_{hor} = 133.63 · kN <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

Ms = 581.6 · kN · m <... stabilizing moment

Mo_T = 530.82 · kN · m <... overturning moment

Mr = 50.78 · kN · m <... resultant moment

KGS GROUP

DESIGN CALCULATIONS COVER SHEET

Project No. :	12-0006-028	**Project Name :**	Talbot River Dam		
File No. :		**Discipline :**	Civil Engineering		
Calculation Title :	**Pier A**				
Calculation No. :	CIV-003	**Prepared by :**	Y.F.	**Date :**	Mar 27, 2013
No. of Sheets :	41	**Checked by :**	H.H	**Date :**	Oct. 16, 2013
Supersedes Calc. No. :		**Approved by :**	H. H.	**Date :**	Oct. 16, 2013

Calculation Description :

The pier has been reviewed against 2007 CDA Dam Safety Guidelines

Related Design Concept :

Stability analysis for the structures is carried out by using "Gravity Method"
Five loading cases are utilized in the analyses based 2007 CDA Guidelines

Reference Codes and Standards :

1.Design of Small Dams, Third Edition, U.S. Government Printing Office, Washington, D.C. 1999.
2. 2007 CDA Dam Safety Guidelines
3. 2009 Parks Canada Directive for Dam Safety Program of Dams and Water-Retaining Structures

ENGINEER'S SEAL

Rev. #	Rev. Description	Rev. Author	Date Revised	Checked by	Approved by	Approved Date

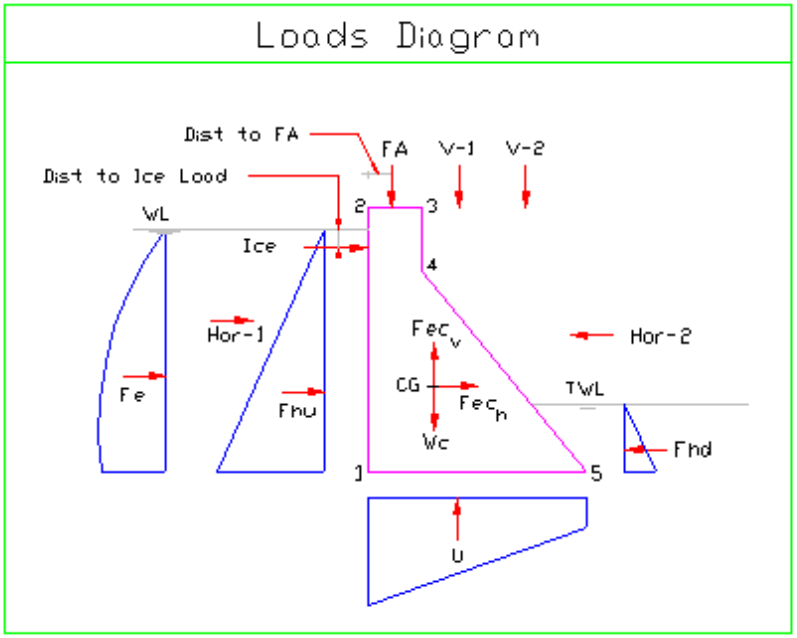
PROGRAM DESCRIPTION SECTION

This Mathcad file can calculate factors of safety against sliding and perform cracked section analysis for any shaped structure. Loading combinations conforms to CDA Dam Safety Guidelines 2007
Features

- metric or imperial units
- ice, anchor, hydrostatic and gravity loads
- crack analysis
- calculates percentage of effective base
- bearing pressures
- volume and position of center of gravity of structure (including voids)
- summary of forces and moments (including resultant and eccentricity)
- additional point loads (e.g.. backfill, vertical hydrostatic, etc.)
- hydrodynamic forces by formulas of Small Dam Design
- equivalent horizontal and vertical inertia forces on structure during earthquake

Assumptions and Limitations

- 2-dimensional
- horizontal base only
- vertical hydrostatic and backfill loads must be input as point loads
- lowest point on structure is level of base
- all vertical point loads are referenced to most upstream point of structure
- Evaluation of cracking during analyses are for unusual and extreme loading cases only.



INPUT SECTION

Unit System

1 - metric
2 - imperial

Select unit system

units := 1 <... select unit system

unL := if(units = 1, 1 · m, 1 · ft)

unM := if(units = 1, 1 · kN, 1 · lb)

Geometry of Structure defined by perimeter coordinates

There is no need to "close" the perimeter ,i.e., so that last point equal to first point

$$X := \begin{pmatrix} 0.000 \\ 0.000 \\ 5.79 \\ 5.79 \\ 7.01 \\ 7.01 \\ 0.00 \end{pmatrix} \cdot unL \qquad Y := \begin{pmatrix} 224.68 \\ 231.70 \\ 231.70 \\ 230.33 \\ 228.25 \\ 224.68 \\ 224.68 \end{pmatrix} \cdot unL$$

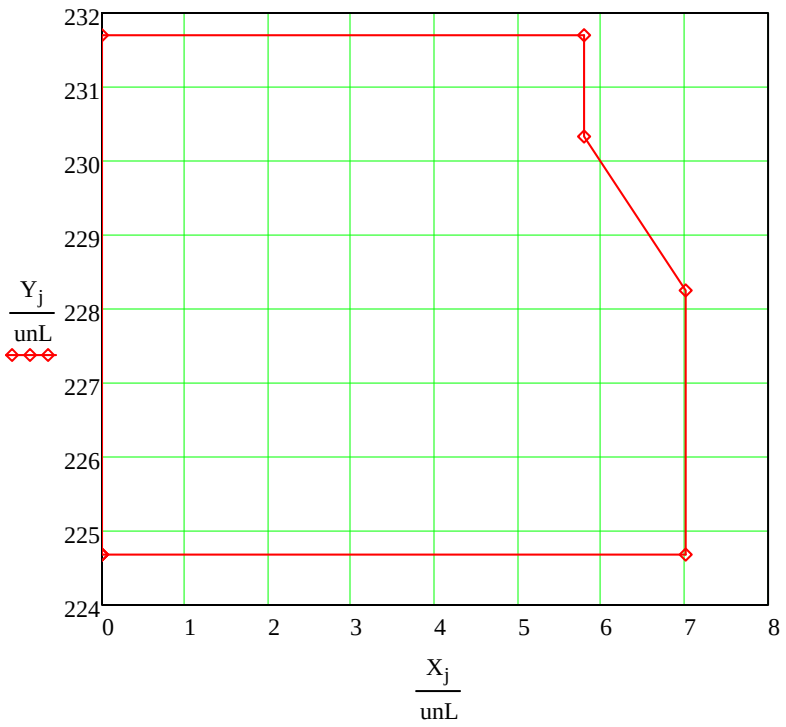
<... input X & Y coordinates

$$maxY := \frac{\max(Y)}{unL} + 5 \qquad minY := \frac{\min(Y)}{unL} - 5 \qquad i := 1 .. length(X)$$

$$maxX := \frac{\max(X)}{unL} + 5 \qquad minX := \frac{\min(X)}{unL} - 5 \qquad j := 1 .. length(X) + 1$$

$X_{length(X)+1} := X_1$ $Y_{length(Y)+1} := Y_1$

Graphical Representation of Structure defined by X & Y Coordinates



Densities of Water and Concrete, Width of Structure, of Structure defined by perimeter coordinates

$\gamma w := \text{if}\left(\text{units} = 1, 9.81 \cdot \frac{kN}{m^3}, 62.5 \cdot \frac{lb}{ft^3}\right)$ <... water density

$\gamma_{silt} := \text{if}\left(\text{units} = 1, 7.7 \cdot \frac{kN}{m^3}, 49 \cdot \frac{lb}{ft^3}\right)$ <... Silt density

$\gamma c := \text{if}\left(\text{units} = 1, 23.5 \cdot \frac{kN}{m^3}, 145 \cdot \frac{lb}{ft^3}\right)$ <... concrete density

$\gamma_{timber} := \text{if}\left(\text{units} = 1, 10 \cdot \frac{kN}{m^3}, 62.4 \cdot \frac{lb}{ft^3}\right)$ <... timber density (for Stoplogs)

$\gamma_{steel} := \text{if}\left(\text{units} = 1, 77 \cdot \frac{kN}{m^3}, 490 \cdot \frac{lb}{ft^3}\right)$ <... Steel density

$UNI_{floor} := \text{if}\left(\text{units} = 1, 0.5 \cdot \frac{kN}{m^2}, 10.4 \cdot \frac{lb}{ft^2}\right)$ <... Steel deck unit area weight

$f'_c := \text{if}(\text{units} = 1, 17 \cdot MPa, 2030 \cdot psi)$ <... concrete compressive strength

$f'_{rock} := \text{if}(\text{units} = 1, 30 \cdot MPa, 4350 \cdot psi)$ <... mass rock compressive strength

$Structure_Width := 2.438$ <... width of structure

$\phi := 36 \cdot deg$ <... angle of internal friction for foundation material

$c := 0 \cdot f'_c$ $c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

Computation of Area and Coordinates of Center of Gravity of Structure

$deltax_i := X_{i+1} - X_i$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot \left(deltay_i \cdot xplusx_i\right)$

$Yginc_i := \frac{deltax_i}{8} \cdot \left[\left(yplusy_i\right)^2 + \frac{\left(deltay_i\right)^2}{3}\right]$ $Xginc_i := \frac{deltay_i}{8} \cdot \left[\left(xplusx_i\right)^2 + \frac{\left(deltax_i\right)^2}{3}\right]$

$Area := \sum_i Areainc_i$ $Area := if\left[\frac{Area}{unL^2} > 0, Area, Area \cdot (-1)\right]$ <... Area of the Structure (defined by X & Y)

$Xg := \sum_i Xginc_i$ $Xg := \frac{if\left[\frac{Xg}{unL^3} > 0, Xg, Xg \cdot (-1)\right]}{Area}$ <... X coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$Yg := \sum_i Yginc_i$ $Yg := \frac{if\left[\frac{Yg}{unL^3} > 0, Yg, Yg \cdot (-1)\right]}{Area}$ <... Y coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$B := Structure_Width \cdot (unL)$ $L := \max(X) - \min(X) = 7.01\,m$

Computation of Concrete Volume and Weight in Structure

$Vol_conc := Area \cdot B$ $Vol_conc = 113 \cdot m^3$ <... volume of concrete in structure

$Wc := Area \cdot B \cdot \gamma c$ $Wc = 2650.9 \cdot kN$ <... weight of concrete in structure

Load Case 1. Usual Loading Summer Case (D+H+S+U)

US_Water_Level := 230.44 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

CREST_ELE := 231.70 <...Crest Elevation

US_L_ELE := 224.68 <...Lowest Dam Elevation at upstream

DS_L_ELE := 224.68 <...Lowest Dam Elevation at downstream

ROLLWAY_TOP_ELE := 227.28 <...Rollways Top Elevation

$$\alpha := \operatorname{atan}\left(\frac{DS_L_ELE - US_L_ELE}{\frac{L}{unL}}\right) = 0 \cdot deg$$ <...Angle of inclined rock foundation

SF := 2.0 <... Minimum Safety Factor

Deck Slab and Gate weight

$Sp_{gate} := 7.62$ <... Clear opening for gate/stoplogs

$N_s := 10$ <... Number of Stoplogs

Lsize := 0.331 <... Size of stoplog

$$W_{gate} := \gamma_{timber} \cdot \left[Sp_{gate} + 2(Lsize)\right] \cdot unL \cdot (Lsize \cdot unL)^2 \cdot N_s$$

$W_{gate} = 90.74 \cdot kN$ <... Weight of gate/stoplogs

$S_{wid} := 4.88$ <... Slab width

$S_{thk} := 0.19$ <... Slab thickness

$$\gamma_{deck} := 5 \cdot \frac{kN}{m^3}$$

$$W_{slab} := \gamma_{deck} \cdot Sp_{gate} \cdot S_{wid} \cdot S_{thk} \cdot (unL)^3$$

$W_{slab} = 35.3 \cdot kN$ <... Weight of slab

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

$$US_Hor_Force := \frac{1}{2} \cdot \gamma w \cdot (US_Water_Level - ROLLWAY_TOP_ELE)^2 \cdot Sp_{gate} \cdot \frac{(unL)^3}{kN}$$

US_Hor_Force = 373.22 <...Hydrostatic force on Gate / Stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

EL_US_Hor_F = 228.33 <...Elevation of the Hydrostatic force on Gate / Stoplogs

ϕ' := 25 · deg <... angle of internal friction for Silt at rest condition

Ko := 1 − sin(ϕ') = 0.58

SILT_ELE := 226.1 <... Silt top elevation

$$Silt_Pressure := \frac{1}{2} \cdot \gamma_{silt} \cdot Ko \cdot (SILT_ELE \cdot unL - DS_L_ELE \cdot unL)^2 \cdot B = 10.93 \cdot kN$$

$$E_{silt} := DS_L_ELE + \frac{1}{3}(SILT_ELE - DS_L_ELE) = 225.15$$

$$DS_Hor_Force := \frac{-Silt_Pressure}{unM}$$ Silt pressure <... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

EL_DS_Hor_F := E_{silt} <...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{slab}}{unM}$ <... Weight of Deck slab

$Dist_to_F1 := 0$ <... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := 0$ <... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := 0$ <... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$ $F_EL := US_L_ELE \cdot (unL)$ $F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$ $F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

$F_ANC := Anchor_Force \cdot (unM)$ $Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$ $EL_USF := EL_US_Hor_F \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$ $EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 397 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 482.85 \cdot kN$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF - DSHF) = 780.9 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ ... = 10041.66 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fi \cdot (EL_ICE - F_EL_D) \ ... = 2130.44 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) - DSHF \cdot (EL_DSF - F_EL_D)$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2} = 2256.52 \cdot kN \cdot m$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ $x_o = 2.57\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ $E = 0.94\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 232.51 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = 25.34 \cdot kPa$

$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$

Calculation_to_be = "continued"

(To ensure un cracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases} \qquad L_t = 0\,m$$

<...Length of base under tention

$$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} \qquad PI = 0\,m$$

$$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases} = 7.01\,m$$

<...Length of base under compression

$$A_t := L_t \cdot B \qquad A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases} \qquad A_c = 17.09\,m^2$$

$$L_1 := L_c + L_t = 7.01\,m$$

$$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$$

<...vertical force due to tension

$$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$$

<...stable moment due to tension

$$Ms := Ms + M_{ten} \qquad F_{ver} := F_{ver} + N_{ten} = 2203.42 \cdot kN \qquad Mr := Ms - Mo_T = 5654.69 \cdot kN \cdot m$$

$$x_o := \frac{Mr}{F_{ver}} = 2.57\,m$$

<...Resultant location. Distance from the toe.

$$E := \frac{L}{2} - x_o = 0.94\,m$$

<...Eccentricity of Resultant

$$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 232.51 \cdot kPa \qquad q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = 25.34 \cdot kPa$$

$$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 232.51 \cdot kPa \qquad fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 25.34 \cdot kPa$$

<...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 0\,m$ <...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \dfrac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$ <...Stress at Foundation

$FSS := \dfrac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 2.05$ <...Factor of Safety Against Sliding.

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

$\Sigma M := Ms - Mo_1$

$\Sigma W := Wc + Fa + V1 + V2$

$A3 := Phu \cdot B$ $T := L$

$T1 := \begin{cases} 3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right) & \text{if } \dfrac{3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ \Sigma W - A3 \cdot T & \\ T & \text{otherwise} \end{cases}$ $T1 = 7.01\,m$ <...Uncracked section depth

$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$ $T1 = 7.01\,m$ <...Full base cracked

$B5 := \begin{cases} 2 \cdot \dfrac{\Sigma W - A3 \cdot T}{T1} + A3 & \\ \dfrac{}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases}$ $B5 = 232.51 \cdot kPa$ <...Stress at downstream face

$Cr := T - T1$ $Cr = 0 \cdot m$ <...Cracked section depth

$T1 := T - Cr = 7.01\,m$ $A_c := B \cdot T1 = 17.09\,m^2$

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] + U_d = 482.85 \cdot kN$

$F_{ver} := Wc + Fa + V1 + V2 - U = 2203.42 \cdot kN$

$F_{ver} := F_{ver} + N_{ten} = 2203.42 \cdot kN$

$$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2} \right) + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2} \right] \cdot \frac{2}{3} \cdot (T1) \right] + U_d \cdot \frac{L}{2} = 2257 \cdot kN \cdot m$$

$Mo_T := Mo_1 + Mo_3 = 4387 \cdot kN \cdot m$ $\quad$ $Mr := Ms - Mo_T$

$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$ $\quad$ $fmax := B5$ $\quad$ $x_o := \begin{cases} \frac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \frac{T1}{3} & \text{otherwise} \end{cases}$ $\quad$ $x_o = 2.57\,m$

$EFF_BASE := \frac{T1}{L}$

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 2.05$ <...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 1

Initial point of inflection

$PI = 0 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 0 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } E \leq \frac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "ok"

CALCULATED CRACK LENGTH

$Cr = 0 \cdot m$

% OF CRACKED BASE

$\frac{Cr}{T} = 0 \cdot \%$

% OF EFFECTIVE BASE (portion not cracked)

$EFF_BASE = 100 \cdot \%$

$q_{1min} = 25.3 \cdot kPa$

When c is taken into account

$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \geq \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "ok"

STRESSES AT FOUNDATION

fmin = 25.3 · kPa fmax = 232.51 · kPa

x_o = 2.57 m <...Resultant location. Distance from the toe.

f := max(fmin, fmax)

Normal_stresses := "OK" if $f \le 0.3 \cdot \min(f'_c, f'_{rock})$; "not good" otherwise <... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

FSS = 2.05

Safety_against_sliding := "OK" if FSS ≥ 1.5; "to increase dam size" otherwise

Safety_against_sliding = "OK"

When c is taken into account

Safety_against_sliding := "OK" if FSS ≥ SF; "to increase dam size" otherwise

Safety_against_sliding = "OK"

Summary of Forces and Moments

1) Vertical Forces

Wc = 2650.95 · kN <... weight of concrete in structure

Fa = 0 · kN <... anchor Force

V1 = 35.33 · kN <... Vertical Force 1 acting on the Downward Direction.

V2 = 0 · kN <... Vertical Force 2 acting on the Downward Direction.

U = 482.85 · kN <... uplift force acting upwards

F_{ver_U} = 2686.27 · kN <... resultant of the vertical forces without uplift

F_{ver} = 2203.42 · kN <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 396.75 \cdot kN$ <... upstream hydrostatic force

$Fhd = 0 \cdot kN$ <... downstream hydrostatic force

$Fi = 0 \cdot kN$ <... Ice Force

$USHF = 373.22 \cdot kN$ <...horizontal force acting on the Downstream Direction

$DSHF = -10.93 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 780.9 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 10041.66 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 4386.97 \cdot kN \cdot m$ <... overturning moment

$Mr = 5654.69 \cdot kN \cdot m$ <... resultant moment

Load Case 2. Usual Loading Winter Case (D+H+S+U+I)

US_Water_Level := 230.14 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

SF := 2.0 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 77 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0.3 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

$$US_Hor_Force := \frac{1}{2} \cdot \gamma w \cdot (US_Water_Level - ROLLWAY_TOP_ELE)^2 \cdot Sp_{gate} \cdot \frac{(unL)^3}{kN}$$

US_Hor_Force = 305.72 <...Hydrostatic force on Gate / Stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

EL_US_Hor_F = 228.23 <...Elevation of the Hydrostatic force on Gate / Stoplogs

US_Hor_Force1 := Ice_Load · Sp_{gate} US_Hor_Force1 = 586.74 <... Ice Load on gate / stoplogs

EL_US_Hor_F1 := US_Water_Level − Dist_WL_to_Ice_Load

EL_US_Hor_F1 = 229.84 <...Elevation of the ice load

$DS_Hor_Force := \frac{-Silt_Pressure}{unM}$ <... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

Silt pressure

$EL_DS_Hor_F := E_{silt}$ <...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{slab}}{unM}$ <... Weight of Deck Slab

Dist_to_F1 := 0 <... distance from most upstream point of structure to point of application of vertical force

Vert_Force_2 := 0 <... Vertical Force acting on the Downward Direction. It is a stabilizing force

Dist_to_F2 := 0 <... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

WL := US_Water_Level · (unL)

F_EL := US_L_ELE · (unL)

F_EL_D := DS_L_ELE · (unL)

EL_ICE := WL − Dist_WL_to_Ice_Load · (unL)

$F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

F_ANC := Anchor_Force · (unM)

Anc_Dist := L − Dist_to_Anc_Force · (unL)

V1 := Vert_Force_1 · (unM)

Dist_V1 := L − Dist_to_F1 · (unL)

V2 := Vert_Force_2 · (unM)

Dist_V2 := L − Dist_to_F2 · (unL)

USHF := US_Hor_Force · (unM)

EL_USF := EL_US_Hor_F · (unL)

USHF1 := US_Hor_Force1 · (unM)

EL_USF1 := EL_US_Hor_F1 · (unL)

DSHF := DS_Hor_Force · (unM)

EL_DSF := EL_DS_Hor_F · (unL)

Ueff := Uplift · %

TWL := Tailwater_Level · (unL)

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 356.5 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Net uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Net uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF + USHF1 - DSHF)$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 10041.66 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fi \cdot (EL_ICE - F_EL_D) \ldots = 5736.58 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) + USHF1 \cdot (EL_USF1 - F_EL_D) \ldots$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 413.11 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -152.32 \cdot kPa$

$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$ Calculation_to_be = "stoped"

(To ensure uncracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 4.09\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases} = 2.92\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 7.11\,m^2$

$L_1 := L_c + L_t = 2.92\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 2228.57 \cdot kN$ $Mr := Ms - Mo_T = 2166.09 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 0.97\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 2.53\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 413.11 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -152.32 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 626.98 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 4.09\,m$ <...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } fmin \leq 0 \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 4.09\,m$ <...Point of inflection of stresses at foundation from heel

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.12$ <...Factor of Safety Against Sliding.

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

$\Sigma M := Ms - Mo_1$

$\Sigma W := Wc + Fa + V1 + V2$

$A3 := Phu \cdot B$ $T := L$

$T1 := \begin{cases} 3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right) & \text{if } \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ \Sigma W - A3 \cdot T & \\ T & \text{otherwise} \end{cases}$ $T1 = 1.86\,m$ <...Uncracked section depth

$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$ $T1 = 1.86\,m$ <...Full base cracked

$B5 := \begin{cases} \dfrac{2 \cdot \dfrac{\Sigma W - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases}$ $B5 = 835.56 \cdot kPa$ <...Stress at downstream face

$Cr := T - T1$ $Cr = 5.15 \cdot m$ <...Cracked section depth

$T1 := T - Cr = 1.86\,m$ $A_c := B \cdot T1 = 4.53\,m^2$

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] + U_d = 794.11 \cdot kN$

$F_{ver} := Wc + Fa + V1 + V2 - U = 1892.16 \cdot kN$

$F_{ver} := F_{ver} + N_{ten} = 1892.16 \cdot kN$

$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \dfrac{Cr}{2}\right) + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] \cdot \dfrac{2}{3} \cdot (T1)\right] + U_d \cdot \dfrac{L}{2} = 3133 \cdot kN \cdot m$

$Mo_T := Mo_1 + Mo_3 = 8870 \cdot kN \cdot m$ $Mr := Ms - Mo_T$

$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$ $fmax := B5$ $x_o := \begin{cases} \dfrac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \dfrac{T1}{3} & \text{otherwise} \end{cases}$ $x_o = 0.62\,m$

$EFF_BASE := \dfrac{T1}{L}$

$FSS := \dfrac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.95$ <...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 2

Initial point of inflection

$PI = 4.09 \cdot m$

Initial point of inflection as % of the base

$\dfrac{PI}{L} = 58.4 \cdot \%$

$$Result := \begin{cases} \text{"ok"} & \text{if } E \le \frac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

CALCULATED CRACK LENGTH | **% OF CRACKED BASE** | **% OF EFFECTIVE BASE** (portion not cracked)

$Cr = 5.152 \cdot m$ | $\frac{Cr}{T} = 73.5 \cdot \%$ | $EFF_BASE = 26.5 \cdot \%$

$q_{1min} = -152.3 \cdot kPa$

When c is taken into account

$$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \ge \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$ $fmax = 835.56 \cdot kPa$

$x_o = 0.62\ m$

<...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \le 0.3 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

$FSS = 0.95$

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam

When c is taken into account

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam s

Summary of Forces and Moments

1) Vertical Forces

$Wc = 2650.95 \cdot kN$	<... weight of concrete in structure
$Fa = 0 \cdot kN$	<... anchor Force
$V1 = 35.33 \cdot kN$	<... Vertical Force 1 acting on the Downward Direction.
$V2 = 0 \cdot kN$	<... Vertical Force 2 acting on the Downward Direction.
$U = 794.11 \cdot kN$	<... uplift force acting upwards
$F_{ver_U} = 2686.27 \cdot kN$	<... resultant of the vertical forces without uplift
$F_{ver} = 1892.16 \cdot kN$	<... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 356.5 \cdot kN$	<... upstream hydrostatic force
$Fhd = 0 \cdot kN$	<... downstream hydrostatic force
$Fi = 187.73 \cdot kN$	<... Ice Force
$USHF = 305.72 \cdot kN$	<...horizontal force **due to hydrostatic pressure from stoplogs**
$USHF1 = 586.74 \cdot kN$	<...horizontal force **due to ice pressure from stoplogs**
$DSHF = -10.93 \cdot kN$	<...horizontal force acting on the Upstream Direction
$F_{hor} = 1447.61 \cdot kN$	<... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 10041.66 \cdot kN \cdot m$	<... stabilizing moment
$Mo_T = 8869.96 \cdot kN \cdot m$	<... overturning moment
$Mr = 1171.7 \cdot kN \cdot m$	<... resultant moment

Load Case 3. Unusual Loading IDF ($D+H_{IDF}+S+U_{IDF}$)

IDF_Level := 231.03 <... IDF level

Tailwater_Level_IDF := 230.44 <... tailwater level at IDF

SF := 1.5 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 121

- Drag Force

<... Horizontal Force acting on the Downstream Direction. For instance, this force can be used to input effect of soil pressure on the upstream side of Structure

EL_US_Hor_F := 228 <...Elevation at which the Upstream Horizontal Force is applied to the Structure

$DS_Hor_Force := \frac{-Silt_Pressure}{unM}$ Silt pressure

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

EL_DS_Hor_F := E_{sil} <...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

Vert_Force_1 := $\frac{W_{slab}}{unM}$ <... Weight of Deck Slab

Dist_to_F1 := 0 <... distance from most upstream point of structure to point of application of vertical force

Vert_Force_2 := 0 <... Vertical Force acting on the Downward Direction. It is a stabilizing force

Dist_to_F2 := 0 <... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL_{IDF} := IDF_Level \cdot (unL)$

$TWL_{IDF} := Tailwater_Level_IDF \cdot (unL)$

F_ANC := Anchor_Force · (unM)

Anc_Dist := L − Dist_to_Anc_Force · (unL)

V1 := Vert_Force_1 · (unM)

Dist_V1 := L − Dist_to_F1 · (unL)

V2 := Vert_Force_2 · (unM)

Dist_V2 := L − Dist_to_F2 · (unL)

USHF := US_Hor_Force · (unM)

EL_USF := EL_US_Hor_F · (unL)

DSHF := DS_Hor_Force · (unM)

EL_DSF := EL_DS_Hor_F · (unL)

Ueff := Uplift · %

TWL := Tailwater_Level · (unL)

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu_{IDF} := (WL_{IDF} - F_EL) \cdot \gamma w$

F_EL = 224.68 m <... upstream hydrostatic pressure

$Phd_{IDF} := (TWL_{IDF} - F_EL_D) \cdot \gamma w$

F_EL_D = 224.68 m <... downstream hydrostatic pressure

$Fhu_{IDF} := Phu_{IDF} \cdot \frac{WL_{IDF} - F_EL}{2} \cdot B$ $Fhu_{IDF} = 482.19 \cdot kN$ <... upstream hydrostatic force

$Fhd_{IDF} := Phd_{IDF} \cdot \frac{TWL_{IDF} - F_EL_D}{2} \cdot B$ $Fhd_{IDF} = 396.75 \cdot kN$ <... downstream hydrostatic force

$Pu_{IDF} := Phu_{IDF} - Phd_{IDF}$ <... Uplift pressure at upstream end

$U_{uIDF} := \frac{(Pu_{IDF} \cdot Ueff) \cdot L \cdot B}{2}$ $U_{uIDF} = 49.46 \cdot kN$ <... Uplift force due to upstream head water

$U_{dIDF} := Phd_{IDF} \cdot L \cdot B$ $U_{dIDF} = 965.7 \cdot kN$ <... Uplift force due to downstream tailwater

$U_{IDF} := U_{uIDF} + U_{dIDF}$ <... Total Uplift force

$F_{horIDF} := (Fhu_{IDF} - Fhd_{IDF}) + (USHF - DSHF)$ <... Resultant of Horizontal Forces

$F_{verIDF} := Wc + Fa + V1 + V2 - U_{IDF}$ <... Resultant of Vertical Forces

$Ms_{IDF} := Wc \cdot (L - Xg) + Fhd_{IDF} \cdot \left(\frac{TWL_{IDF} - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ ... = 10803.42 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_{1IDF} := Fhu_{IDF} \cdot \left[\frac{WL_{IDF} - F_EL}{3} + (F_EL_D - F_EL)\right] + USHF \cdot (EL_USF - F_EL_D) \ ... = 1427.53 \cdot kN \cdot m$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$ <... Overturning Moment (uplift not included)

$Mo_{2IDF} := U_{uIDF} \cdot \left(\frac{2 \cdot L}{3}\right) + U_{dIDF} \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_{TIDF} := Mo_{1IDF} + Mo_{2IDF}$ <... Total Overturning Moment

$Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$ <... Resultant Moment

$x_{oIDF} := \frac{Mr_{IDF}}{F_{verIDF}}$ $x_{oIDF} = 3.45\,m$ <...Resultant location. Distance from the toe.

$E_{IDF} := \frac{L}{2} - x_{oIDF}$ $E_{IDF} = 0.06\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E_{IDF}}{L}\right) = 102.65 \cdot kPa$ $q_{1min} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E_{IDF}}{L}\right) = 92.91 \cdot kPa$

(To consider cohesion and tensile

$$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases} \qquad L_t = 0\,m$$

<...Length of base under tention

$$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 0\,m$$

$$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases} = 7.01\,m$$

<...Length of base under compression

$$A_t := L_t \cdot B \qquad A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases} = 17.09\,m^2 \qquad L_1 := L_c + L_t = 7.01\,m$$

$$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$$

<...vertical force due to tension

$$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$$

<...stable moment due to tension

$$Ms_{IDF} := Ms_{IDF} + M_{ten} \qquad F_{verIDF} := F_{verIDF} + N_{ten} = 1671.11 \cdot kN \qquad Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$$

$$x_{oIDF} := \frac{Mr_{IDF}}{F_{verIDF}} \qquad x_{oIDF} = 3.45\,m$$

$$E_{IDF} := \frac{L}{2} - x_{oIDF} \qquad E_{IDF} = 0.06\,m$$

$$q_{1max} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E_{IDF}}{L}\right) = 102.65 \cdot kPa \qquad q_{1min} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E_{IDF}}{L}\right) = 92.91 \cdot kPa$$

$$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{verIDF}}{3 \cdot x_{oIDF} \cdot B} & \text{otherwise} \end{cases} = 102.65 \cdot kPa \qquad fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 92.91 \cdot kPa$$

<...Stress at Foundation

$$PI := \begin{cases} L - 3 \cdot (x_{oIDF}) & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 0\,m$$

<...Point of inflection of stresses at foundation from heel

$$FSS := \frac{\left(F_{verIDF} \cdot \cos(\alpha) + F_{horIDF} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{horIDF} \cdot \cos(\alpha) - F_{verIDF} \cdot \sin(\alpha)} = 5.59$$ <...Factor of Safety Against Sliding.

STRESSES AT FOUNDATION

$fmin = 92.9 \cdot kPa$ $fmax = 102.7 \cdot kPa$

initial point of inflection

$PI = 0 \cdot m$

initial point of inflection as % of the base

$\frac{PI}{L} = 0 \cdot \%$

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

Static Method of Analysis.-*Assumptions associated with a static, cracked-section analysis are:*
1 Stress distribution along a horizontal section,without uplift, varies linearly between upstream and downstream faces.
2 Once a crack occurs, uplift pressure equivalent to reservoir pressure above the crack exists throughout the entire crack depth. Uplift is then assumed to vary linearly from crack tip to tailwater pressure at downstream face.
3 Crack penetrates to point of zero stress. This assumes no tensile strength at crack tip, which means the entire untracked length is entirely in compression.

Refer to "Design of small dam" P 331-332

$\Sigma M_{IDF} := Ms_{IDF} - Mo_{1IDF}$ $\Sigma W_{IDF} := Wc + Fa + V1 + V2$ $A3 := Phu_{IDF} \cdot B$

$T := L$

$$T1 := \begin{cases} \frac{3 \cdot \left(\Sigma M_{IDF} - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W_{IDF} - A3 \cdot T} & \text{if } \frac{3 \cdot \left(\Sigma M_{IDF} - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W_{IDF} - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases}$$ <...Un cracked section depth

$T1 = 7.01\,m$

$$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$$ <...Full base cracked

$$B5 := \begin{cases} \frac{2 \cdot \frac{\Sigma W_{IDF} - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases}$$ <...Stress at downstream face

$Cr := T - T1$ $Cr = 0 \cdot m$ <...Cracked section depth

$T1 := T - Cr = 7.01\,m$ $A_n := B \cdot T1 = 17.09\,m^2$

$$U_{IDF} := \left(Pu_{IDF} \cdot Ueff\right) \cdot Cr \cdot B + \left(Pu_{IDF} \cdot Ueff\right) \cdot \left[\frac{(T1) \cdot B}{2}\right] + U_{dIDF}$$

$F_{ver_UIDF} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces without uplift

$F_{verIDF} := Wc + Fa + V1 + V2 - U_{IDF}$ <... Resultant of Vertical Forces

$$Mo_{3IDF} := \left[\left(Pu_{IDF} \cdot Ueff \right) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2} \right) + \left(Pu_{IDF} \cdot Ueff \right) \cdot \left[\frac{(T1) \cdot B}{2} \right] \cdot \frac{2}{3} \cdot (T1) \right] + U_{dIDF} \cdot \frac{L}{2}$$

$Mo_{TIDF} := Mo_{1IDF} + Mo_{3IDF}$ $Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$

$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$ $fmax := B5$ $EFF_BASE := \frac{T1}{L}$

$$x_{oIDF} := \begin{cases} \frac{Mr_{IDF}}{F_{verIDF}} & \text{if } Cr = 0 \\ \frac{T1}{3} & \text{otherwise} \end{cases}$$

$x_{oIDF} = 3.45\,m$

$$FSS := \frac{\left(F_{verIDF} \cdot \cos(\alpha) + F_{horIDF} \cdot \sin(\alpha) \right) \cdot \tan(\phi) + c \cdot A_c}{F_{horIDF} \cdot \cos(\alpha) - F_{verIDF} \cdot \sin(\alpha)} = 5.59$$

<...Factor of Safety Against Sliding.

RESULTS for Loading Case 3

Initial point of inflection

$PI = 0 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 0 \cdot \%$

CALCULATED CRACK LENGTH

$Cr = 0 \cdot m$

% OF CRACKED BASE

$\frac{Cr}{T} = 0 \cdot \%$

% OF EFFECTIVE BASE (portion not cracked)

$EFF_BASE = 100 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 75 \cdot \% \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$ Result = "ok"

<...Resultant required to be is within base.

$q_{1min} = 92.9 \cdot kPa$

When c is taken into account

$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \geq \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "ok"

STRESSES AT FOUNDATION

$fmin = 92.9 \cdot kPa$ $fmax = 102.7 \cdot kPa$

$f := \max(fmin, fmax)$ $x_{oIDF} = 3.447\ m$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \le 0.5 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.5 x f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST OVERTURNING and SLIDING

$FSS = 5.59$

$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge 1.3 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$

Safety_against_sliding = "OK"

When c is taken into account

$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$

Safety_against_sliding = "OK"

Summary of Forces and Moments

1) Vertical Forces

$Wc = 2650.95 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 35.33 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U_{IDF} = 1015.16 \cdot kN$ <... uplift force acting upwards

$F_{ver_UIDF} = 2686.27 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{verIDF} = 1671.11 \cdot kN$ <... resultant of the vertical forces with

uplift

2) Horizontal Forces

$Fhu_{IDF} = 482.19 \cdot kN$ <... upstream hydrostatic force

$Fhd_{IDF} = 396.75 \cdot kN$ <... downstream hydrostatic force

$USHF = 121 \cdot kN$ <...Drag Force

$DSHF = -10.93 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{horIDF} = 217.37 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms_{IDF} = 10803.42 \cdot kN \cdot m$ <... stabilizing moment

$Mo_{TIDF} = 5043.46 \cdot kN \cdot m$ <... overturning moment

$Mr_{IDF} = 5759.96 \cdot kN \cdot m$ <... resultant moment

Load Case 4. Extreme Loading Earthquake (D+H+S+Q+U_Q)

Horizontal & Vertical Components of the earthquake Intensity and Coefficient of Hydrodynamic Pressure Distribution

$\lambda_{hor} := 0.05$ <... **horizontal** component of earthquake intensity = ratio of earthquake acceleration to acceleration due to gravity

$\lambda_{vert} := \frac{2}{3} \cdot \lambda_{hor}$ <... **vertical** component of earthquake intensity

<... from owner's requirement 21.2.5.1

$C_m := 0.74$ <... from fig 8-6, page 325 of "Design of Small Dams", for a structure with **vertical face**

US_Water_Level := 230.44 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

$f_t := 0$ **To be assumed for EQ case due to loss of cohesive bond** <... tensile strength at interface of concrete /rock

SF := 1.3 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **acceleration,** i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

$$US_Hor_Force := \frac{1}{2} \cdot \gamma w \cdot (US_Water_Level - ROLLWAY_TOP_ELE)^2 \cdot Sp_{gate} \cdot \frac{(unL)^3}{kN}$$

US_Hor_Force = 373.22 <...Hydrostatic on stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

<...Elevation at which the Upstream Horizontal Force is applied to the Structure

EL_US_Hor_F = 228.33

$$DS_Hor_Force := \frac{-Silt_Pressure}{unM}$$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

Silt pressure

$EL_DS_Hor_F := E_{silt}$ <...Elevation at which the Downstream Horizontal Force is applied to the Structure

$W_{tower} := 0$ <...Weight of tower

$H_{tower} := 0$ <...Height of tower

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **acceleration,** i.e., metric units, **Vertical Forces** will be in **kN**

$$Vert_Force_1 := \frac{W_{slab}}{unM}$$

<... Weight of Deck Slab

$Dist_to_F1 := 0$ <... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := 0$ <... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := 0$ <... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$ $F_EL := US_L_ELE \cdot (unL)$ $F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$ $F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

$F_ANC := Anchor_Force \cdot (unM)$ $Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$ $EL_USF := EL_US_Hor_F \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$ $EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces and Uplift

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 396.75 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d$ <... Total Uplift force

Computation of Hydrodynamic Force Due to Earthquake

$h := US_Water_Level \cdot (unL) - F_EL$ $h = 5.76\,m$ <...total depth of the reservoir at section being analyses

$y := h$ <....vertical distance from reservoir surface to the elevation in question

$C := \frac{C_m}{2} \cdot \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right) + \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right)\right]^{0.5}\right]$ <...Coefficient of Hydrodynamic Pressure Distribution

$Pe := C \cdot \lambda_{hor} \cdot \gamma w \cdot h \cdot Structure_Width \cdot (unL)$ $Pe = 5.1 \cdot \frac{kN}{m}$ <...hydrodynamic pressure

$Fe1 := 0.726 \cdot Pe \cdot y$ $Fe1 = 21.3 \cdot kN$ <...hydrodynamic force at pier

$Me := 0.299 \cdot Pe \cdot y^2$ $Me = 51 \cdot kN \cdot m$ <...moment at the base of the structure due to hydrodynamic force

$P_{appl} := \begin{cases} F_EL + \frac{Me}{Fe1} & \text{if } Fe1 > 0kN \\ 0 \cdot m & \text{otherwise} \end{cases}$ $P_{appl} = 227.052 \cdot m$ <...elevation of point of application of hydrodynamic force

$Fe2 := 0.726 \cdot Pe \cdot y \cdot \frac{Sp_{gate}}{(Structure_Width)}$ $Fe2 = 66.62 \cdot kN$ <...hydrodynamic force at gate/ stoplogs

$Fe := Fe1 + Fe2$ $Fe = 87.94 \cdot kN$ <...Total hydrodynamic force

$Fec1_h := Wc \cdot \lambda_{hor}$ $Fec1_h = 132.55 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec_v := Wc \cdot \lambda_{vert}$ $Fec_v = 88.36 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec2_h := W_{gate} \cdot \lambda_{hor}$ $Fec2_h = 4.54 \cdot kN$ <... Equivalent Horizontal Inertia Force on Gate/stologs due to earthquake

$Fec_h := Fec1_h + Fec2_h$ $Fec_h = 137.08 \cdot kN$ <... Equivalent Horizontal Inertia Force on pier and gate due to earthquake

$Fec3_h := W_{slab} \cdot \lambda_{hor}$ $Fec3_h = 1.77 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$Fec3_v := W_{slab} \cdot \lambda_{vert}$ $Fec3_v = 1.18 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

$Fec4_h := W_{tower} \cdot \lambda_{hor}$ $Fec4_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Tower due to earthquake

$Fec4_v := W_{tower} \cdot \lambda_{vert}$ $Fec4_v = 0 \cdot kN$ <... Equivalent Vertical Inertia Force on Tower due to earthquake

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$F_{hor} := (Fhu - Fhd) + Fe + Fec_h + Fec3_h + Fec4_h + (USHF - DSHF) = 1007.69 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - \left(U + Fec_v + Fec3_v + Fec4_v\right) = 2113.88 \cdot kN$ <... Resultant of Vertical Forces

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 10041.66 \cdot kN \cdot m$ <... Stabilizing Moment

$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$

$$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fe \cdot \left(P_{appl} - F_EL_D\right) \ldots$$
$$+ Fec_h \cdot (Yg - F_EL_D) + Fec_v \cdot (L - Xg) + USHF \cdot (EL_USF - F_EL_D) \ldots$$
$$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$$

<... Overturning Moment (uplift and EQ due to slab/tower inertial force not included)

$Mo_1 = 3127.25 \cdot kN \cdot m$

$$Mo_{11} := Fec3_h \cdot (CREST_ELE \cdot unL - F_EL_D) + Fec3_v \cdot (L - Dist_to_F1 \cdot unL) \ldots$$
$$+ Fec4_h \cdot \left(CREST_ELE \cdot unL - F_EL_D + \frac{H_{tower}}{2}\right) + Fec4_v \cdot (L - Dist_to_F2 \cdot unL)$$

<... Overturning Moment due to slab and tower inertial force

$Mo_{11} = 20.65 \cdot kN \cdot m$

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ $\quad Mo_2 = 2257 \cdot kN \cdot m$ <... Overturning Moment due to Uplift

$Mo_T := \left(Mo_1 + Mo_{11}\right) + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ $\quad x_o = 2.19\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ $\quad E = 1.31\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 262.51 \cdot kPa$ $\qquad q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -15.13 \cdot kPa$

(To consider cohesion and tensile strength)

$$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$$

$L_t = 0\,m$ <...Length of base under tention

$$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$$

$PI = 0.43\,m$

$$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \\ (L - PI) & \text{otherwise} \end{cases} = 6.58\,m$$

<...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 16.04\,m^2$

$L_1 := L_c + L_t = 6.58\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t \right) \right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 2113.88 \cdot kN$ $Mr := Ms - Mo_T = 4637.23 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 2.19\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.31\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L} \right) = 262.51 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L} \right) = -15.13 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 263.5 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 0.43\,m$ <...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$ <...Stress at Foundation

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha) \right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.52$ <...Factor of Safety Against Sliding.

RESULTS for Loading case 4

Initial point of inflection

$PI = 0.43 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 6.12 \cdot \%$

$EFF_BASE := 1 - \left(\frac{PI}{L}\right)$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 94 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \cdot \% \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant required to be is within base.

Result = "ok"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 263.5 \cdot kPa$

$f := \max(fmin, fmax)$

$x_o = 2.19\,m$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.9 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.9 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

Not Required. The Earthquake load case is used to establish post-earthquake condition of the dam

Summary of Forces and Moments

1) Vertical Forces

$Wc = 2650.95 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 35.33 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$Fec_v = 88.36 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec3_v = 1.18 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

$U = 482.85 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 2686.27 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 2113.88 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 396.75 \cdot kN$ <... upstream hydrostatic force

$USHF = 373.22 \cdot kN$ <...hydrostatic force on gate/stoplogs

$Fhd = 0 \cdot kN$ <... downstream hydrostatic force

$Fe = 87.94 \cdot kN$ <...hydrodynamic force on pier + gate/stoplogs

$Fec_h = 137.08 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec3_h = 1.77 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$DSHF = -10.93 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 1007.69 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 10041.66 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 5404.43 \cdot kN \cdot m$ <... overturning moment

$Mr = 4637.23 \cdot kN \cdot m$ <... resultant moment

Load Case 5. Post-Earthquake (D+H+S+U_{PQ})

Computation of crack length based on earthquake loading case

$\Sigma M := Ms - Mo_1 + Mo_{11} = 6935.06 \cdot kN \cdot m$

$\Sigma W := (Wc + Fa + V1 + V2) - \left(Fec_v + Fec3_v + Fec4_v\right) = 2596.73 \cdot kN$

$A3 := Phu \cdot B$

$$T1 := \begin{cases} \dfrac{3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} & \text{if } \dfrac{3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases}$$

<..Uncracked section depth

$$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$$

$T1 = 6.53\,m$

<...Full base cracked when T1=0

$A_c := T1 \cdot B = 15.92\,m^2$

$Cr := \max(T - T1, PI)$ $\quad Cr = 0.48 \cdot m$ <...Cracked section depth

$T1 := T - Cr = 6.53\,m$

$$B5 := \begin{cases} 2 \cdot \dfrac{\Sigma W_{IDF} - A3 \cdot T}{T1} + A3 \\ \hline B \end{cases} \quad \text{if } T1 < T$$
$$fmax \quad \text{otherwise}$$

<...Stress at downstream face

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] + U_d = 515.9 \cdot kN$

$F_{ver_U} := Wc + Fa + V1 + V2$ <... Downwards Vertical Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver} := F_{ver} + N_{ten} = 2170.37 \cdot kN$

$F_{hor} := (Fhu - Fhd) + Fi + (USHF - DSHF)$

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fi \cdot (EL_ICE - F_EL_D) \ldots = 2130.44 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) - DSHF \cdot (EL_DSF - F_EL_D)$

$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2}\right) + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] \cdot \frac{2}{3} \cdot (T1)\right] + U_d \cdot \frac{L}{2}$

<... Overturning Moment due to increased uplift force

$Mo_T := Mo_1 + Mo_3 = 4536 \cdot kN \cdot m$

$Mr := Ms - Mo_T = 5505.51 \cdot kN \cdot m$

$x_o := \begin{cases} \frac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \frac{T1}{3} & \text{otherwise} \end{cases}$ $\quad x_o = 2.18\,m$

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 2.02$

<...Factor of Safety Against Sliding.

$EFF_BASE := \frac{T - Cr}{L}$

$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$ $\quad fmax := B5$

RESULTS for Loading case 5

CALCULATED CRACK LENGTH

$Cr = 0.48 \cdot m$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 93.2 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant within the base .

Result = "ok"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 272.7 \cdot kPa$

$f := \max(fmin, fmax)$

$x_o = 2.18\,m$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.5 \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength = 0.5 x f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST OVERTURNING and SLIDING

FSS = 2.02

Safety_against_sliding := "OK" if FSS ≥ 1.1
"to increase dam size" otherwise

Safety_against_sliding = "OK"

Summary of Forces and Moments

1) Vertical Forces

Wc = 2650.95 · kN <... weight of concrete in structure

Fa = 0 · kN <... anchor Force

V1 = 35.33 · kN <... Vertical Force 1 acting on the Downward Direction.

V2 = 0 · kN <... Vertical Force 2 acting on the Downward Direction.

U = 515.9 · kN <... uplift force acting upwards

F_{ver_U} = 2686.27 · kN <... resultant of the vertical forces without uplift

F_{ver} = 2170.37 · kN <... resultant of the vertical forces with uplift

2) Horizontal Forces

Fhu = 396.75 · kN <... upstream hydrostatic force

USHF = 373.22 · kN <...hydrostatic force on gate/stoplogs

Fhd = 0 · kN <... downstream hydrostatic force

DSHF = –10.93 · kN <...horizontal force acting on the Upstream Direction

F_{hor} = 780.9 · kN <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

Ms = 10041.66 · kN · m <... stabilizing moment

Mo_T = 4536.15 · kN · m <... overturning moment

Mr = 5505.51 · kN · m <... resultant moment

KGS GROUP

DESIGN CALCULATIONS COVER SHEET

Project No. :	12-0006-028	**Project Name :**	PCA Talbot River Dam		
File No. :		**Discipline :**	Civil Engineering		
Calculation Title :	**Pier with Rollway**				
Calculation No. :	CIV-004	**Prepared by :**	Y.F.	**Date :**	Mar 28, 2013
No. of Sheets :	47	**Checked by :**	H.H	**Date :**	Oct. 18, 2013
Supersedes Calc. No. :		**Approved by :**	H. H.	**Date :**	Oct. 18, 2013

Calculation Description :

The pier has been reviewed against 2007 CDA Dam Safety Guidelines

Related Design Concept :

Stability analysis for the structures is carried out by using "Gravity Method"
Five loading cases are utilized in the analyses based 2007 CDA Guidelines

Reference Codes and Standards :

1.Design of Small Dams, Third Edition, U.S. Government Printing Office, Washington, D.C. 1999.
2. 2007 CDA Dam Safety Guidelines
3. 2009 Parks Canada Directive for Dam Safety Program of Dams and Water-Retaining Structures

ENGINEER'S SEAL

Rev. #	Rev. Description	Rev. Author	Date Revised	Checked by	Approved by	Approved Date

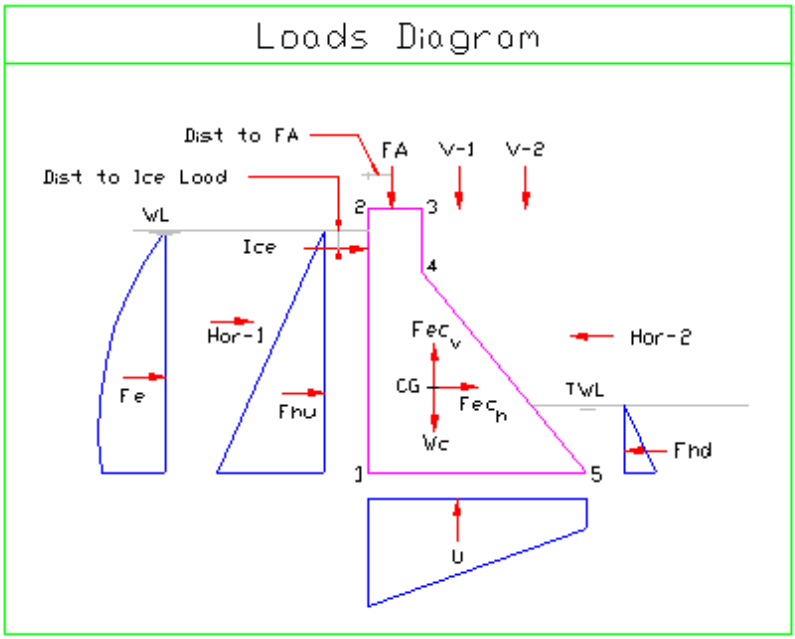
PROGRAM DESCRIPTION SECTION

This Mathcad file can calculate factors of safety against sliding and perform cracked section analysis for any shaped structure. Loading combinations conforms to CDA Dam Safety Guidelines 2007
Features

- metric or imperial units
- ice, anchor, hydrostatic and gravity loads
- crack analysis
- calculates percentage of effective base
- bearing pressures
- volume and position of center of gravity of structure (including voids)
- summary of forces and moments (including resultant and eccentricity)
- additional point loads (e.g.. backfill, vertical hydrostatic, etc.)
- hydrodynamic forces by formulas of Small Dam Design
- equivalent horizontal and vertical inertia forces on structure during earthquake

Assumptions and Limitations

- 2-dimensional
- horizontal base only
- vertical hydrostatic and backfill loads must be input as point loads
- lowest point on structure is level of base
- all vertical point loads are referenced to most upstream point of structure
- Evaluation of cracking during analyses are for unusual and extreme loading cases only.



INPUT SECTION

Unit System

1 - metric
2 - imperial

Select unit system

units := 1 <... select unit system

unL := if(units = 1, 1 · m, 1 · ft)

unM := if(units = 1, 1 · kN, 1 · lb)

Geometry of Structure defined by perimeter coordinates

There is no need to "close" the perimeter ,i.e., so that last point equal to first point

$$X := \begin{pmatrix} 0.000 \\ 0.000 \\ 1.07 \\ 1.07 \\ 2.32 \\ 3.4 \\ 3.4 \end{pmatrix} \cdot unL \qquad Y := \begin{pmatrix} 224.68 \\ 227.28 \\ 227.28 \\ 226.87 \\ 226.87 \\ 225.14 \\ 224.68 \end{pmatrix} \cdot unL$$

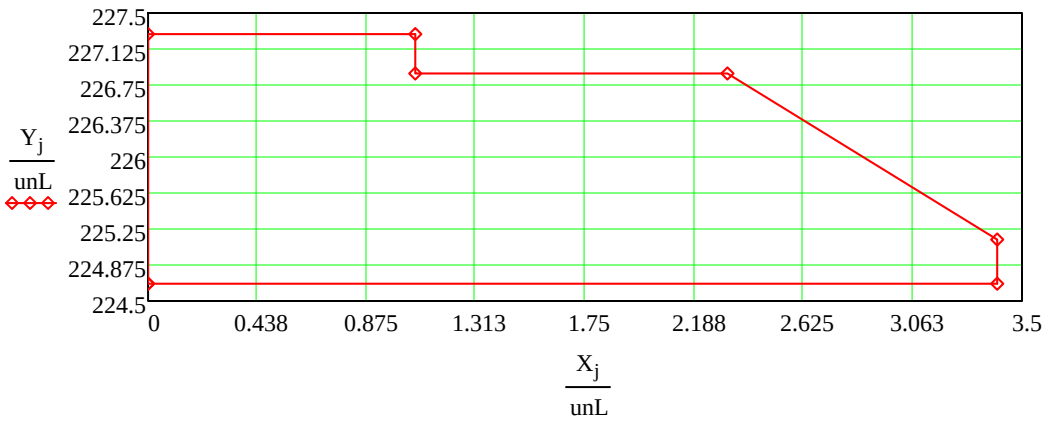
<... input X & Y coordinates

$$maxY := \frac{max(Y)}{unL} + 5 \qquad minY := \frac{min(Y)}{unL} - 5 \qquad i := 1 .. length(X)$$

$$maxX := \frac{max(X)}{unL} + 5 \qquad minX := \frac{min(X)}{unL} - 5 \qquad j := 1 .. length(X) + 1$$

$$X_{length(X)+1} := X_1 \qquad Y_{length(Y)+1} := Y_1$$

Graphical Representation of Structure defined by X & Y Coordinates



Densities of Water and Concrete, Width of Structure, of Structure defined by perimeter coordinates

$\gamma w := \text{if}\left(\text{units} = 1, 9.81 \cdot \frac{kN}{m^3}, 62.5 \cdot \frac{lb}{ft^3}\right)$ <... water density

$\gamma_{silt} := \text{if}\left(\text{units} = 1, 7.7 \cdot \frac{kN}{m^3}, 49 \cdot \frac{lb}{ft^3}\right)$ <... Silt density

$\gamma c := \text{if}\left(\text{units} = 1, 23.5 \cdot \frac{kN}{m^3}, 145 \cdot \frac{lb}{ft^3}\right)$ <... concrete density

$\gamma_{timber} := \text{if}\left(\text{units} = 1, 10 \cdot \frac{kN}{m^3}, 62.4 \cdot \frac{lb}{ft^3}\right)$ <... timber density (for Stoplogs)

$\gamma_{steel} := \text{if}\left(\text{units} = 1, 77 \cdot \frac{kN}{m^3}, 490 \cdot \frac{lb}{ft^3}\right)$ <... Steel density

$UNI_{floor} := \text{if}\left(\text{units} = 1, 0.5 \cdot \frac{kN}{m^2}, 10.4 \cdot \frac{lb}{ft^2}\right)$ <... Steel deck unit area weight

$f'_c := \text{if}(\text{units} = 1, 17 \cdot MPa, 2030 \cdot psi)$ <... concrete compressive strength

$f'_{rock} := \text{if}(\text{units} = 1, 30 \cdot MPa, 4350 \cdot psi)$ <... mass rock compressive strength

$Structure_Width := 1.0$ <... width of structure

$\phi := 36 \cdot deg$ <... angle of internal friction for foundation material

$c := 0 \cdot f'_c$ $c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

Computation of Area and Coordinates of Center of Gravity of Structure

$deltax_i := X_{i+1} - X_i$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot \left(deltay_i \cdot xplusx_i\right)$

$Yginc_i := \frac{deltax_i}{8} \cdot \left[\left(yplusy_i\right)^2 + \frac{\left(deltay_i\right)^2}{3}\right]$ $Xginc_i := \frac{deltay_i}{8} \cdot \left[\left(xplusx_i\right)^2 + \frac{\left(deltax_i\right)^2}{3}\right]$

$Area := \sum_i Areainc_i$ $Area := if\left[\frac{Area}{unL^2} > 0, Area, Area \cdot (-1)\right]$ <... Area of the Structure (defined by X & Y)

$Xg := \sum_i Xginc_i$ $Xg := \frac{if\left[\frac{Xg}{unL^3} > 0, Xg, Xg \cdot (-1)\right]}{Area}$ <... X coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$Yg := \sum_i Yginc_i$ $Yg := \frac{if\left[\frac{Yg}{unL^3} > 0, Yg, Yg \cdot (-1)\right]}{Area}$ <... Y coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$B := Structure_Width \cdot (unL) = 1\,m$ $L := \max(X) - \min(X) = 3.4\,m$

Computation of Concrete Volume and Weight in Structure

$Vol_conc := Area \cdot B$ $Vol_conc = 7 \cdot m^3$ <... volume of concrete in structure

$Wc := Area \cdot B \cdot \gamma c$ $Wc = 163.3 \cdot kN$ <... weight of concrete in structure

Load Case 1. Usual Loading Summer Case (D+H+S+U)

US_Water_Level := 230.44 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

ROLLWAY_TOP_ELE := 227.28 <...Rollways Top Elevation

$$CREST_ELE := ROLLWAY_TOP_ELE + \frac{US_Water_Level - ROLLWAY_TOP_ELE}{2} = 228.86$$

<...Mid-height Elevation of water above rollway

US_L_ELE := 224.68 <...Lowest Dam Elevation at upstream

DS_L_ELE := 224.68 <...Lowest Dam Elevation at downstream

$$\alpha := \operatorname{atan}\left(\frac{DS_L_ELE - US_L_ELE}{\frac{L}{unL}}\right) = 0 \cdot deg$$

<...Angle of inclined rock foundation

SF := 2.0 <... Minimum Safety Factor

Stoplog_weight

$Sp_{gate} := 0$ <... Clear opening for gate/stoplogs

$N_s := 9$ <... Number of Stoplogs

Lsize := 0.356 <... Size of stoplog

$$W_{gate} := \gamma_{timber} \cdot (Sp_{gate}) \cdot (Lsize \cdot unL)^2 \cdot N_s$$

$W_{gate} = 0 \cdot kN$ <... Weight of gate/stoplogs

$l_{water} := 1.07 + 0.36$ <...Water on the rollway in front of stoplogs plus stoplogs

$$V_{water} := (US_Water_Level - ROLLWAY_TOP_ELE) \cdot (l_{water}) \cdot (Structure_Width) = 4.52$$

$W_{water} := V_{water} \cdot \gamma w \cdot (unL)^3 = 44.33 \cdot kN$ <... weight of water above rollway

$$X_{water} := \frac{(l_{water} + Lsize)}{2} + \left(\frac{L}{unL} - l_{water} - Lsize\right) = 2.51$$

<... distance of center of gravity to toe

$W_{slab} := W_{water} = 44.33 \cdot kN$ <... weight of water above rollway

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Horizontal Forces Acting on the stoplog to piers - to be removed

$$US_Hor_Force := -\frac{1}{2}\cdot\gamma w\cdot(US_Water_Level - ROLLWAY_TOP_ELE)^2\cdot\frac{B}{unL}\cdot\frac{(unL)^3}{kN}$$

US_Hor_Force = −48.98 <...Hydrostatic force on Gate / Stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

EL_US_Hor_F = 228.33 <...Elevation of the Hydrostatic force on Gate / Stoplogs

$\phi' := 25\cdot deg$ <... angle of internal friction for Silt at rest condition

$Ko := 1 - \sin(\phi') = 0.58$

SILT_ELE := 225.1 <... Silt top elevation

$$Silt_Pressure := \frac{1}{2}\cdot\gamma_{silt}\cdot Ko\cdot(SILT_ELE\cdot unL - DS_L_ELE\cdot unL)^2\cdot B = 0.39\cdot kN$$

$$E_{silt} := DS_L_ELE + \frac{1}{3}(SILT_ELE - DS_L_ELE) = 224.82$$

$DS_Hor_Force := \frac{-Silt_Pressure}{unM}$ Silt pressure

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{silt}$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{slab}}{unM}$

<... Weight of Deck slab

$Dist_to_F1 := \frac{L}{unL} - X_{water} = 0.89$

<... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := 0$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := 0$

<... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$ $F_EL := US_L_ELE \cdot (unL)$ $F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$ $F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

$F_ANC := Anchor_Force \cdot (unM)$ $Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$ $EL_USF := EL_US_Hor_F \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$ $EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$

<... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$

<... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 163 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 96.06 \cdot kN$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF - DSHF) = 114.15 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces <u>without uplift</u>

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ ... = 430.23 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fi \cdot (EL_ICE - F_EL_D) \ ... = 133.57 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) - DSHF \cdot (EL_DSF - F_EL_D)$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ $x_o = 0.71\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ $E = 0.99\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 90.34 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -24.68 \cdot kPa$

Calculation_to_be := "continued" if $q_{1min} > \frac{f_t}{SF}$; "stoped" otherwise

Calculation_to_be = "stoped"

(To ensure un cracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$L_t :=$ 0 if $q_{1min} \geq 0 \vee q_{1min} < f_t$; $\left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right]$ otherwise

$L_t = 0\,m$ <...Length of base under tention

$PI :=$ $L - 3 \cdot x_o$ if $q_{1min} < f_t$; $0 \cdot m$ otherwise

$PI = 1.28\,m$

$L_c :=$ $L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)}$ if $(q_{1min} \geq f_t) \wedge (q_{1min} < 0)$ $= 2.12\,m$ <...Length of base under compression

; $(L - PI)$ otherwise

$A_t := L_t \cdot B$ $A_c :=$ $L_c \cdot B$ if $L_c \geq 0$; 0 otherwise $A_c = 2.12\,m^2$

$L_1 := L_c + L_t = 2.12\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 111.61 \cdot kN$ $Mr := Ms - Mo_T = 78.93 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 0.71\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 0.99\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 90.34 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -24.68 \cdot kPa$

$$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \dfrac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 105.21 \cdot kPa \qquad fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$$

<...Stress at Foundation

$$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 1.28\,m$$

<...Point of inflection of stresses at foundation from heel

$$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \dfrac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} \qquad fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$$

<...Stress at Foundation

$$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.71$$

<...Factor of Safety Against Sliding.

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

$$\Sigma M := Ms - Mo_1$$

$$\Sigma W := Wc + Fa + V1 + V2$$

$$A3 := Phu \cdot B \qquad T := L$$

PI := 0

If calcs show that combined pier / rollway would not have cracked base or the crack not extend beyond very u/s face of rollway, to remove any cracks from the rollway calculations.

$$T1 := \begin{cases} \dfrac{3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} & \text{if } \dfrac{3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases} \qquad T1 = 3.4\,m$$

<...Uncracked section depth

$$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases} \qquad T1 = 3.4\,m$$

<...Full base cracked

$$B5 := \begin{cases} \dfrac{2 \cdot \dfrac{\Sigma W - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases} \qquad B5 = 105.21 \cdot kPa$$

<...Stress at downstream face

$$Cr := \max(T - T1, PI) = 0\,m$$

<...Cracked section depth

$$T1 := T - Cr = 3.4\,m$$

$$A_c := B \cdot T1 = 3.4\,m^2$$

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] + U_d = 96.06 \cdot kN$

$F_{ver} := Wc + Fa + V1 + V2 - U = 111.61 \cdot kN$

$F_{ver} := F_{ver} + N_{ten} = 111.61 \cdot kN$

$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2}\right) + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] \cdot \frac{2}{3} \cdot (T1)\right] + U_d \cdot \frac{L}{2} = 218 \cdot kN \cdot m$

$Mo_T := Mo_1 + Mo_3 = 351 \cdot kN \cdot m$ $\qquad$ $Mr := Ms - Mo_T$

$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$ $\qquad$ $fmax := B5$

$x_o := \begin{cases} \frac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \frac{T1}{3} & \text{otherwise} \end{cases}$ $\qquad$ $x_o = 0.71\,m$

$EFF_BASE := \frac{T1}{L}$

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.71$ <...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 1

Initial point of inflection

$PI = 0 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 0 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } E \leq \frac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

CALCULATED CRACK LENGTH

$Cr = 0 \cdot m$

% OF CRACKED BASE

$\frac{Cr}{T} = 0 \cdot \%$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 100 \cdot \%$

$q_{1min} = -24.7 \cdot kPa$

When c is taken into account

$$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \geq \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 105.21 \cdot kPa$

$x_o = 0.71\,m$

<...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.3 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

$FSS = 0.71$

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \geq 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam

When c is taken into account

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \geq SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam s

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1) Vertical Forces

$Wc = 163.34 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 44.33 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U = 96.06 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 207.67 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 111.61 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 162.74 \cdot kN$ <... upstream hydrostatic force

$Fhd = 0 \cdot kN$ <... downstream hydrostatic force

$Fi = 0 \cdot kN$ <... Ice Force

$USHF = -48.98 \cdot kN$ <...horizontal force acting on the Downstream Direction

$DSHF = -0.39 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 114.15 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 430.23 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 351.31 \cdot kN \cdot m$ <... overturning moment

$Mr = 78.93 \cdot kN \cdot m$ <... resultant moment

Load Case 2. Usual Loading Winter Case (D+H+S+U+I)

US_Water_Level := 230.14 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

SF := 2.0 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0.3 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

$$US_Hor_Force := -\frac{1}{2}\cdot \gamma w \cdot (US_Water_Level - ROLLWAY_TOP_ELE)^2 \cdot \frac{B}{unL}\cdot \frac{(unL)^3}{kN}$$

US_Hor_Force = −40.12 <...Hydrostatic force on Gate / Stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

EL_US_Hor_F = 228.23 <...Elevation of the Hydrostatic force on Gate / Stoplogs

$US_Hor_Force1 := Ice_Load \cdot Sp_{gate}$ US_Hor_Force1 = 0 <... Ice Load on gate / stoplogs

EL_US_Hor_F1 := US_Water_Level − Dist_WL_to_Ice_Load

EL_US_Hor_F1 = 229.84 <...Elevation of the ice load

$$DS_Hor_Force := \frac{-Silt_Pressure}{unM}$$

Silt pressure

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{silt}$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$$Vert_Force_1 := \frac{W_{slab}}{unM}$$

<... weight of water above rollway

$Dist_to_F1 := 0$

<... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := 0$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := 0$

<... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$

$F_EL := US_L_ELE \cdot (unL)$

$F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$

$F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$

$EL_USF := EL_US_Hor_F \cdot (unL)$

$USHF1 := US_Hor_Force1 \cdot (unM)$

$EL_USF1 := EL_US_Hor_F1 \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$

$EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$

$TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 146.23 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Net uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Net uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF + USHF1 - DSHF)$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ ... = 469.82 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fi \cdot (EL_ICE - F_EL_D) \ ... = 123.62 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) + USHF1 \cdot (EL_USF1 - F_EL_D) \ ...$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 64.63 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = 3.97 \cdot kPa$

$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$

Calculation_to_be = "continued"

(To ensure uncracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 0\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases} = 3.4\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 3.4\,m^2$

$L_1 := L_c + L_t = 3.4\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 116.61 \cdot kN$ $Mr := Ms - Mo_T = 139.8 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 1.2\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 0.5\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 64.63 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = 3.97 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 64.63 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 3.97 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 0\,m$ <...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } fmin \leq 0 \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 0\,m$ <...Point of inflection of stresses at foundation from heel

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.8$ <...Factor of Safety Against Sliding.

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

$\Sigma M := Ms - Mo_1$

$\Sigma W := Wc + Fa + V1 + V2$

$A3 := Phu \cdot B$ $T := L$

$T1 := \begin{cases} 3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right) & \text{if } \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ \Sigma W - A3 \cdot T & \\ T & \text{otherwise} \end{cases}$ $T1 = 3.4\,m$ <...Uncracked section depth

$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$ $T1 = 3.4\,m$ <...Full base cracked

$B5 := \begin{cases} \dfrac{2 \cdot \dfrac{\Sigma W - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases}$ $B5 = 64.63 \cdot kPa$ <...Stress at downstream face

$Cr := \max(T - T1, PI) = 0\,m$ <...Cracked section depth

$T1 := T - Cr = 3.4\,m$

$A_c := B \cdot T1 = 3.4\,m^2$

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] + U_d = 91.06 \cdot kN$

$F_{ver} := Wc + Fa + V1 + V2 - U = 116.61 \cdot kN$

$F_{ver} := F_{ver} + N_{ten} = 116.61 \cdot kN$

$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \dfrac{Cr}{2}\right) + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] \cdot \dfrac{2}{3} \cdot (T1)\right] + U_d \cdot \dfrac{L}{2} = 206 \cdot kN \cdot m$

$Mo_T := Mo_1 + Mo_3 = 330 \cdot kN \cdot m$ $Mr := Ms - Mo_T$

$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$ $fmax := B5$ $x_o := \begin{cases} \dfrac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \dfrac{T1}{3} & \text{otherwise} \end{cases}$ $x_o = 1.2\,m$

$EFF_BASE := \dfrac{T1}{L}$

$FSS := \dfrac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.8$ <...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 2

Initial point of inflection

Initial point of inflection as % of the base

$PI = 0 \cdot m$ $\quad \frac{PI}{L} = 0 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } E \le \frac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "ok"

CALCULATED CRACK LENGTH **% OF CRACKED BASE** **% OF EFFECTIVE BASE** (portion not cracked)

$Cr = 0 \cdot m$ $\quad \frac{Cr}{T} = 0 \cdot \%$ $\quad EFF_BASE = 100 \cdot \%$

$q_{1min} = 4 \cdot kPa$

When c is taken into account

$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \ge \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "ok"

STRESSES AT FOUNDATION

$fmin = 4 \cdot kPa$ $\quad fmax = 64.63 \cdot kPa$

$x_o = 1.2\,m$

<...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \le 0.3 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

$FSS = 0.8$

$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$

Safety_against_sliding = "to increase dam

When c is taken into account

Safety_against_sliding := "OK" if FSS ≥ SF
"to increase dam size" otherwise

Safety_against_sliding = "to increase dam s

Summary of Forces and Moments

1) Vertical Forces

Wc = 163.34 · kN <... weight of concrete in structure

Fa = 0 · kN <... anchor Force

V1 = 44.33 · kN <... Vertical Force 1 acting on the Downward Direction.

V2 = 0 · kN <... Vertical Force 2 acting on the Downward Direction.

U = 91.06 · kN <... uplift force acting upwards

F_{ver_U} = 207.67 · kN <... resultant of the vertical forces without uplift

F_{ver} = 116.61 · kN <... resultant of the vertical forces with uplift

2) Horizontal Forces

Fhu = 146.23 · kN <... upstream hydrostatic force

Fhd = 0 · kN <... downstream hydrostatic force

Fi = 0 · kN <... Ice Force

USHF = –40.12 · kN <...horizontal force **due to hydrostatic pressure from stoplogs**

USHF1 = 0 · kN <...horizontal force **due to ice pressure from stoplogs**

DSHF = –0.39 · kN <...horizontal force acting on the Upstream Direction

F_{hor} = 106.5 · kN <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

Ms = 469.82 · kN · m <... stabilizing moment

Mo_T = 330.02 · kN · m <... overturning moment

Mr = 139.8 · kN · m <... resultant moment

Load Case 3. Unusual Loading IDF ($D+H_{IDF}+S+U_{IDF}$)

IDF_Level := 231.03 <... IDF level

Tailwater_Level_IDF := 230.44 <... tailwater level at IDF

SF := 1.5 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 40 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0

- Drag Force

<... Horizontal Force acting on the Downstream Direction. For instance, this force can be used to input effect of soil pressure on the upstream side of Structure

EL_US_Hor_F := 0 <...Elevation at which the Upstream Horizontal Force is applied to the Structure

$DS_Hor_Force := \frac{-Silt_Pressure}{unM}$ Silt pressure

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{silt}$ <...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$L_{rollway} := \frac{L}{unL} = 3.4$

$h1 := IDF_Level - ROLLWAY_TOP_ELE = 3.75$ <... height of the tailwater

$W_{topwater} := L_{rollway} \cdot \frac{B}{unL} \cdot (h1) \cdot \frac{\gamma w}{\left(\frac{kN}{m^3}\right)} = 125.08$ <... Weight of water above rollway, conservatively

$L_{topwater} := \frac{1}{2} \cdot L_{rollway} = 1.7$ <... Moment arm of Weight of tailwater to toe

$Vert_Force_1 := W_{topwater}$ <... Weight of water above rollway

$Dist_to_F1 := \frac{L}{unL} - L_{topwater} = 1.7$ <... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := 0$ <... Weight of tailwater on dam

$Dist_to_F2 := 0$ <... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL_{IDF} := IDF_Level \cdot (unL)$

$TWL_{IDF} := Tailwater_Level_IDF \cdot (unL)$

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$

$EL_USF := EL_US_Hor_F \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$

$EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu_{IDF} := \left(WL_{IDF} - F_EL\right) \cdot \gamma w$ $F_EL = 224.68\,m$ <... upstream hydrostatic pressure

$Phd_{IDF} := \left(TWL_{IDF} - F_EL_D\right) \cdot \gamma w$ $F_EL_D = 224.68\,m$ <... downstream hydrostatic pressure

$Fhu_{IDF} := Phu_{IDF} \cdot \frac{WL_{IDF} - F_EL}{2} \cdot B$ $Fhu_{IDF} = 197.78 \cdot kN$ <... upstream hydrostatic force

$Fhd_{IDF} := Phd_{IDF} \cdot \frac{TWL_{IDF} - F_EL_D}{2} \cdot B$ $Fhd_{IDF} = 162.74 \cdot kN$ <... downstream hydrostatic force

$Pu_{IDF} := Phu_{IDF} - Phd_{IDF}$ <... Uplift pressure at upstream end

$U_{uIDF} := \frac{\left(Pu_{IDF} \cdot Ueff\right) \cdot L \cdot B}{2}$ $U_{uIDF} = 3.94 \cdot kN$ <... Uplift force due to upstream head water

$U_{dIDF} := Phd_{IDF} \cdot L \cdot B$ $U_{dIDF} = 192.12 \cdot kN$ <... Uplift force due to downstream tailwater

$U_{IDF} := U_{uIDF} + U_{dIDF}$ <... Total Uplift force

$F_{horIDF} := \left(Fhu_{IDF} - Fhd_{IDF}\right) + (USHF - DSHF)$ <... Resultant of Horizontal Forces

$F_{verIDF} := Wc + Fa + V1 + V2 - U_{IDF}$ <... Resultant of Vertical Forces

$Ms_{IDF} := Wc \cdot (L - Xg) + Fhd_{IDF} \cdot \left(\frac{TWL_{IDF} - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 844.19 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_{1IDF} := Fhu_{IDF} \cdot \left[\frac{WL_{IDF} - F_EL}{3} + (F_EL_D - F_EL)\right] + USHF \cdot (EL_USF - F_EL_D) \ldots = 418.69 \cdot kN \cdot m$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$ <... Overturning Moment (uplift not included)

$Mo_{2IDF} := U_{uIDF} \cdot \left(\frac{2 \cdot L}{3}\right) + U_{dIDF} \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_{TIDF} := Mo_{1IDF} + Mo_{2IDF}$ <... Total Overturning Moment

$Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$ <... Resultant Moment

$x_{oIDF} := \frac{Mr_{IDF}}{F_{verIDF}}$ $x_{oIDF} = 0.97\,m$ <...Resultant location. Distance from the toe.

$E_{IDF} := \frac{L}{2} - x_{oIDF}$ $E_{IDF} = 0.73\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E_{IDF}}{L}\right) = 61.96 \cdot kPa$ $q_{1min} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E_{IDF}}{L}\right) = -7.63 \cdot kPa$

(To consider cohesion and tensile

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \quad = -0.2\,m \\ 0 \cdot m & \text{otherwise} \end{cases}$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \quad = 3.6\,m \\ (L - PI) & \text{otherwise} \end{cases}$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \quad = 3.6\,m^2 \\ 0 & \text{otherwise} \end{cases}$ $L_1 := L_c + L_t = 3.6\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms_{IDF} := Ms_{IDF} + M_{ten}$ $F_{verIDF} := F_{verIDF} + N_{ten} = 92.36 \cdot kN$ $Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$

$x_{oIDF} := \frac{Mr_{IDF}}{F_{verIDF}}$ $x_{oIDF} = 0.97\,m$

$E_{IDF} := \frac{L}{2} - x_{oIDF}$ $E_{IDF} = 0.73\,m$

$q_{1max} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E_{IDF}}{L}\right) = 61.96 \cdot kPa$ $q_{1min} := \frac{F_{verIDF}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E_{IDF}}{L}\right) = -7.63 \cdot kPa$

$fmax := \begin{cases} q_{1max} \text{ if } q_{1min} \geq f_t = 63.21 \cdot kPa \\ \frac{2 \cdot F_{verIDF}}{3 \cdot x_{oIDF} \cdot B} \text{ otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} \text{ if } q_{1min} \geq f_t = 0 \cdot kPa \\ 0 \text{ otherwise} \end{cases}$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot (x_{oIDF}) \text{ if } q_{1min} < f_t = 0.48\,m \\ 0 \cdot m \text{ otherwise} \end{cases}$ <...Point of inflection of stresses at foundation from heel

$FSS := \frac{(F_{verIDF} \cdot \cos(\alpha) + F_{horIDF} \cdot \sin(\alpha)) \cdot \tan(\phi) + c \cdot A_c}{F_{horIDF} \cdot \cos(\alpha) - F_{verIDF} \cdot \sin(\alpha)} = 1.89$ <...Factor of Safety Against Sliding.

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$ $fmax = 63.2 \cdot kPa$

initial point of inflection

$PI = 0.48 \cdot m$

initial point of inflection as % of the base

$\frac{PI}{L} = 14.05 \cdot \%$

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

Static Method of Analysis.-*Assumptions associated with a static, cracked-section analysis are:*
1 Stress distribution along a horizontal section,without uplift, varies linearly between upstream and downstream faces.
2 Once a crack occurs, uplift pressure equivalent to reservoir pressure above the crack exists throughout the entire crack depth. Uplift is then assumed to vary linearly from crack tip to tailwater pressure at downstream face.
3 Crack penetrates to point of zero stress. This assumes no tensile strength at crack tip, which means the entire untracked length is entirely in compression.

Refer to "Design of small dam" P 331-332

$\Sigma M_{IDF} := Ms_{IDF} - Mo_{1IDF}$ $\Sigma W_{IDF} := Wc + Fa + V1 + V2$ $A3 := Phu_{IDF} \cdot B$

$T := L$

$T1 := \begin{cases} \frac{3 \cdot \left(\Sigma M_{IDF} - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W_{IDF} - A3 \cdot T} \text{ if } \frac{3 \cdot \left(\Sigma M_{IDF} - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W_{IDF} - A3 \cdot T} < T \wedge PI > 0 \\ T \text{ otherwise} \end{cases}$ <...Un cracked section depth

$T1 = 2.56\,m$

$T1 := \begin{cases} 0.0000001 \cdot (unL) \text{ if } T1 < 0 \\ T1 \text{ otherwise} \end{cases}$ <...Full base cracked

$$B5 := \begin{cases} 2 \cdot \dfrac{\Sigma W_{IDF} - A3 \cdot T}{T1} + A3 \\ \hline B \end{cases} \quad \text{if } T1 < T \qquad \text{fmax otherwise}$$

<...Stress at downstream face

$$Cr := \max(T - T1, PI) = 0.84\,m$$

<...Cracked section depth

$$T1 := T - Cr = 2.56\,m$$

$$A_c := B \cdot T1 = 2.56\,m^2$$

$$U_{IDF} := \left(Pu_{IDF} \cdot Ueff\right) \cdot Cr \cdot B + \left(Pu_{IDF} \cdot Ueff\right) \cdot \left[\frac{(T1) \cdot B}{2}\right] + U_{dIDF}$$

$$F_{ver_UIDF} := Wc + Fa + V1 + V2$$

<... resultant of the vertical forces without uplift

$$F_{verIDF} := Wc + Fa + V1 + V2 - U_{IDF}$$

<... Resultant of Vertical Forces

$$Mo_{3IDF} := \left[\left(Pu_{IDF} \cdot Ueff\right) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2}\right) + \left(Pu_{IDF} \cdot Ueff\right) \cdot \left[\frac{(T1) \cdot B}{2}\right] \cdot \frac{2}{3} \cdot (T1)\right] + U_{dIDF} \cdot \frac{L}{2}$$

$$Mo_{TIDF} := Mo_{1IDF} + Mo_{3IDF} \qquad Mr_{IDF} := Ms_{IDF} - Mo_{TIDF}$$

$$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases} \qquad fmax := B5 \qquad EFF_BASE := \frac{T1}{L}$$

$$x_{oIDF} := \begin{cases} \dfrac{Mr_{IDF}}{F_{verIDF}} & \text{if } Cr = 0 \\ \dfrac{T1}{3} & \text{otherwise} \end{cases} \qquad x_{oIDF} = 0.85\,m$$

$$FSS := \frac{\left(F_{verIDF} \cdot \cos(\alpha) + F_{horIDF} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{horIDF} \cdot \cos(\alpha) - F_{verIDF} \cdot \sin(\alpha)} = 1.87$$

<...Factor of Safety Against Sliding.

RESULTS for Loading Case 3

Initial point of inflection

$$PI = 0.48 \cdot m$$

Initial point of inflection as % of the base

$$\frac{PI}{L} = 14.05 \cdot \%$$

CALCULATED CRACK LENGTH | **% OF CRACKED BASE** | **% OF EFFECTIVE BASE** (portion not cracked)

$Cr = 0.838 \cdot m$ | $\frac{Cr}{T} = 24.64 \cdot \%$ | $EFF_BASE = 75.36 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 75 \cdot \% \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

Result = "ok"

<...Resultant required to be is within base.

$q_{1min} = -7.6 \cdot kPa$

When c is taken into account

$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \geq \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 122.1 \cdot kPa$

$f := \max(fmin, fmax)$

$x_{oIDF} = 0.854\ m$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.5 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.5 x f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST OVERTURNING and SLIDING

$FSS = 1.87$

$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \geq 1.3 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$

Safety_against_sliding = "OK"

When c is taken into account

$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \geq SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$

Safety_against_sliding = "OK"

Summary of Forces and Moments

1) Vertical Forces

$Wc = 163.34 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 125.08 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U_{IDF} = 197.02 \cdot kN$ <... uplift force acting upwards

$F_{ver_UIDF} = 288.41 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{verIDF} = 91.39 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu_{IDF} = 197.78 \cdot kN$ <... upstream hydrostatic force

$Fhd_{IDF} = 162.74 \cdot kN$ <... downstream hydrostatic force

$USHF = 0 \cdot kN$ <...Drag Force

$DSHF = -0.39 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{horIDF} = 35.44 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms_{IDF} = 844.19 \cdot kN \cdot m$ <... stabilizing moment

$Mo_{TIDF} = 756.14 \cdot kN \cdot m$ <... overturning moment

$Mr_{IDF} = 88.04 \cdot kN \cdot m$ <... resultant moment

Load Case 4. Extreme Loading Earthquake (D+H+S+Q+U_Q)

Horizontal & Vertical Components of the earthquake Intensity and Coefficient of Hydrodynamic Pressure Distribution

$\lambda_{hor} := 0.05$ <... **horizontal** component of earthquake intensity = ratio of earthquake acceleration to acceleration due to gravity

$\lambda_{vert} := \frac{2}{3} \cdot \lambda_{hor}$ <... **vertical** component of earthquake intensity

<... from owner's requirement 21.2.5.1

$C_m := 0.74$ <... from fig 8-6, page 325 of "Design of Small Dams", for a structure with **vertical face**

US_Water_Level := 230.44 <... max. headwater level

Tailwater_Level := 224.68 <... tailwater level

$f_t := 0$ **To be assumed for EQ case due to loss of cohesive bond** <... tensile strength at interface of concrete /rock

SF := 1.3 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **acceleration,** i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Horizontal Forces Acting on the stoplog to piers - to be removed

$$US_Hor_Force := -\frac{1}{2} \cdot \gamma w \cdot (US_Water_Level - ROLLWAY_TOP_ELE)^2 \cdot \frac{B}{unL} \cdot \frac{(unL)^3}{kN}$$

US_Hor_Force = −48.98 <...Hydrostatic on stoplogs

$$EL_US_Hor_F := ROLLWAY_TOP_ELE + \frac{(US_Water_Level - ROLLWAY_TOP_ELE)}{3}$$

<...Elevation at which the Upstream Horizontal Force is applied to the Structure

EL_US_Hor_F = 228.33

$$DS_Hor_Force := \frac{-Silt_Pressure}{unM}$$

Silt pressure

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{silt}$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

$W_{tower} := 0$

<...Weight of tower

$H_{tower} := 0$

<...Height of tower

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **acceleration,** i.e., metric units, **Vertical Forces** will be in **kN**

$$Vert_Force_1 := \frac{W_{slab}}{unM} = 44.33$$

<... weight of water above rollway

$$Dist_to_F1 := \frac{L}{unL} - X_{water} = 0.89$$

<... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := 0$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := 0$

<... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$

$F_EL := US_L_ELE \cdot (unL)$

$F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$

$$F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$$

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$ $EL_USF := EL_US_Hor_F \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$ $EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces and Uplift

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 162.74 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d$ <... Total Uplift force

Computation of Hydrodynamic Force Due to Earthquake

$h := US_Water_Level \cdot (unL) - F_EL$ $h = 5.76\,m$ <...total depth of the reservoir at section being analyses

$y := h$ <....vertical distance from reservoir surface to the elevation in question

$C := \frac{C_m}{2} \cdot \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right) + \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right)\right]^{0.5}\right]$ <...Coefficient of Hydrodynamic Pressure Distribution

$Pe := C \cdot \lambda_{hor} \cdot \gamma w \cdot h \cdot Structure_Width \cdot (unL)$ $Pe = 2.09 \cdot \frac{kN}{m}$ <...hydrodynamic pressure

$Fe1 := 0.726 \cdot Pe \cdot y$ $Fe1 = 8.7 \cdot kN$ <...hydrodynamic force total

$Me := 0.299 \cdot Pe \cdot y^2$ $Me = 21 \cdot kN \cdot m$ <...moment at the base of the structure due to hydrodynamic force

$P_{appl} := \begin{cases} F_EL + \frac{Me}{Fe1} & \text{if } Fe1 > 0kN \\ 0 \cdot m & \text{otherwise} \end{cases}$ $P_{appl} = 227.052 \cdot m$ <...elevation of point of application of hydrodynamic force

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Hydrodynamic forces transfered to piers - to be removed from rollway

$y1 := (US_Water_Level - ROLLWAY_TOP_ELE) \cdot unL = 3.16\,m$ <....vertical distance from reservoir surface to the elevation in question

$C := \frac{C_m}{2} \cdot \left[\frac{y1}{h} \cdot \left(2 - \frac{y1}{h}\right) + \left[\frac{y1}{h} \cdot \left(2 - \frac{y1}{h}\right)\right]^{0.5}\right] = 0.62$ <...Coefficient of Hydrodynamic Pressure Distribution

$Pe1 := C \cdot \lambda_{hor} \cdot \gamma w \cdot y1 \cdot Structure_Width \cdot (unL)$ $Pe1 = 0.97 \cdot \frac{kN}{m}$ <...hydrodynamic pressure

$Fe11 := 0.726 \cdot Pe1 \cdot y1$ $Fe11 = 2.2 \cdot kN$ <...hydrodynamic force total

$Me1 := 0.299 \cdot Pe1 \cdot y1^2$ $Me1 = 3 \cdot kN \cdot m$ <...moment at the base of the structure due to hydrodynamic force

$P_{appl1} := \begin{cases} (ROLLWAY_TOP_ELE) \cdot unL + \frac{Me1}{Fe1} & \text{if } Fe1 > 0kN \\ 0 \cdot m & \text{otherwise} \end{cases}$ $P_{appl1} = 227.611 \cdot m$ <...elevation of point of application of hydrodynamic force

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$Fe2 := 0.726 \cdot Pe \cdot y \cdot \frac{Sp_{gate}}{(Structure_Width)}$ $Fe2 = 0 \cdot kN$ <...hydrodynamic force at gate/ stoplogs

$Fe := Fe1 + Fe2$ $Fe = 8.74 \cdot kN$ <...Total hydrodynamic force

$Fec1_h := Wc \cdot \lambda_{hor}$ $Fec1_h = 8.17 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec_v := Wc \cdot \lambda_{vert}$ $Fec_v = 5.44 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec2_h := W_{gate} \cdot \lambda_{hor}$ $Fec2_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Gate/stologs due to earthquake

$Fec_h := Fec1_h + Fec2_h$ $Fec_h = 8.17 \cdot kN$ <... Equivalent Horizontal Inertia Force on pier and gate due to earthquake

$Fec3_h := W_{slab} \cdot \lambda_{hor}$ $Fec3_h = 2.22 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$Fec3_v := W_{slab} \cdot \lambda_{vert}$ $Fec3_v = 1.48 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

$Fec4_h := W_{tower} \cdot \lambda_{hor}$ $Fec4_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Tower due to earthquake

$Fec4_v := W_{tower} \cdot \lambda_{vert}$ $Fec4_v = 0 \cdot kN$ <... Equivalent Vertical Inertia Force on Tower due to earthquake

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$F_{hor} := (Fhu - Fhd) + (Fe - Fe11) + Fec_h + Fec3_h + Fec4_h + (USHF - DSHF) = 131.05 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - \left(U + Fec_v + Fec3_v + Fec4_v\right) = 104.68 \cdot kN$ <... Resultant of Vertical Forces

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ ... = 430.23 \cdot kN \cdot m$ <... Stabilizing Moment
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL)\right] + Fe \cdot \left(P_{appl} - F_EL_D\right) - Fe11 \cdot \left(P_{appl1} - F_EL_D\right) ...$
$+ Fec_h \cdot (Yg - F_EL_D) + Fec_v \cdot (L - Xg) + USHF \cdot (EL_USF - F_EL_D) ...$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$

$Mo_1 = 167.48 \cdot kN \cdot m$ <... Overturning Moment (uplift and EQ due to slab/tower inertial force not included)

$Mo_{11} := Fec3_h \cdot (CREST_ELE \cdot unL - F_EL_D) + Fec3_v \cdot (L - Dist_to_F1 \cdot unL) ...$
$+ Fec4_h \cdot \left(CREST_ELE \cdot unL - F_EL_D + \frac{H_{tower}}{2}\right) + Fec4_v \cdot (L - Dist_to_F2 \cdot unL)$

<... Overturning Moment due to slab and tower inertial force

$Mo_{11} = 12.97 \cdot kN \cdot m$

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ $Mo_2 = 218 \cdot kN \cdot m$ <... Overturning Moment due to Uplift

$Mo_T := \left(Mo_1 + Mo_{11}\right) + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ $x_o = 0.31\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ $E = 1.39\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 106.52 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -44.94 \cdot kPa$

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 2.48\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \\ (L - PI) & \text{otherwise} \end{cases} = 0.92\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 0.92\,m^2$

$L_1 := L_c + L_t = 0.92\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 104.68 \cdot kN$ $Mr := Ms - Mo_T = 32.05 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 0.31\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.39\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 106.52 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -44.94 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 227.95 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$

<...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 2.48\,m$

<...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$

<...Stress at Foundation

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.58$

<...Factor of Safety Against Sliding.

RESULTS for Loading case 4

Initial point of inflection

$PI = 2.48 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 72.99 \cdot \%$

$EFF_BASE := 1 - \left(\frac{PI}{L}\right)$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 27 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \cdot \% \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant required to be is within base.

Result = "ok"

STRESSES AT FOUNDATION

fmin = 0 · kPa

fmax = 227.9 · kPa

$f := \max(fmin, fmax)$ $x_o = 0.31\,m$

$Normal_stresses := \begin{cases} "OK" & \text{if } f \le 0.9 \cdot \min(f'_c, f'_{rock}) \\ "not\ good" & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.9 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

Not Required. The Earthquake load case is used to establish post-earthquake condition of the dam

Summary of Forces and Moments

1) Vertical Forces

$Wc = 163.34 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 44.33 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$Fec_v = 5.44 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec3_v = 1.48 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

$U = 96.06 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 207.67 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 104.68 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 162.74 \cdot kN$ <... upstream hydrostatic force

$USHF = -48.98 \cdot kN$ <...hydrostatic force on gate/stoplogs

$Fhd = 0 \cdot kN$ <... downstream hydrostatic force

$Fe = 8.74 \cdot kN$ <...hydrodynamic force on pier + gate/stoplogs

$Fec_h = 8.17 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec3_h = 2.22 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$DSHF = -0.39 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 131.05 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 430.23 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 398.18 \cdot kN \cdot m$ <... overturning moment

$Mr = 32.05 \cdot kN \cdot m$ <... resultant moment

Load Case 5. Post-Earthquake (D+H+S+U_{PQ})

Computation of crack length based on earthquake loading case

$\Sigma M := Ms - Mo_1 + Mo_{11} = 275.72 \cdot kN \cdot m$

$\Sigma W := (Wc + Fa + V1 + V2) - (Fec_v + Fec3_v + Fec4_v) = 200.74 \cdot kN$

$A3 := Phu \cdot B$

$PI := 0$

If calcs show that combined pier / rollway would not have cracked base or the crack not extend beyond very u/s face of rollway, to remove any cracks from the rollway calculations.

$$T1 := \begin{cases} \dfrac{3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} & \text{if } \dfrac{3 \cdot \left(\Sigma M - A3 \cdot \dfrac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases}$$

<..Uncracked section depth

$$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$$

$T1 = 3.4\,m$

<...Full base cracked when T1=0

$A_c := T1 \cdot B = 3.4\,m^2$

$Cr := \max(T - T1, PI)$ $\quad Cr = 0 \cdot m$ <...Cracked section depth

$T1 := T - Cr = 3.4\,m$

$$B5 := \begin{cases} 2 \cdot \dfrac{\Sigma W_{IDF} - A3 \cdot T}{T1} + A3 \\ \hline B \end{cases} \quad \text{if } T1 < T$$
$$fmax \quad \text{otherwise}$$

<...Stress at downstream face

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\dfrac{(T1) \cdot B}{2}\right] + U_d = 96.06 \cdot kN$

$F_{ver_U} := Wc + Fa + V1 + V2$ <... Downwards Vertical Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{hor} := (Fhu - Fhd) + Fi + (USHF - DSHF)$

$$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2} \right) + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2} \right] \cdot \frac{2}{3} \cdot (T1) \right] + U_d \cdot \frac{L}{2}$$

<... Overturning Moment due to increased uplift force

$$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL_D - F_EL) \right] + Fi \cdot (EL_ICE - F_EL_D) \ldots = 133.57 \cdot kN \cdot m$$
$$+ USHF \cdot (EL_USF - F_EL_D) - DSHF \cdot (EL_DSF - F_EL_D)$$

$$Mo_T := Mo_1 + Mo_3 = 351 \cdot kN \cdot m \qquad Mr := Ms - Mo_T$$

$$x_o := \begin{cases} \frac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \frac{T1}{3} & \text{otherwise} \end{cases} \qquad x_o = 0.71\,m$$

$$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.71$$

<...Factor of Safety Against Sliding.

$$EFF_BASE := \frac{T - Cr}{L} \qquad fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases} \qquad fmax := B5$$

RESULTS for Loading case 5

CALCULATED CRACK LENGTH

$Cr = 0 \cdot m$

% OF CRACKED BASE

$\frac{Cr}{T} = 0 \cdot \%$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 100 \cdot \%$

$$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant within the base .

Result = "ok"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 227.9 \cdot kPa$

$f := \max(fmin, fmax)$

$x_o = 0.71\,m$

<... Allowable compressive

$$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \le 0.5 \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$$

strength = 0.5 x f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST OVERTURNING and SLIDING

FSS = 0.71

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge 1.1 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam size

Summary of Forces and Moments

1) Vertical Forces

Wc = 163.34 · kN <... weight of concrete in structure

Fa = 0 · kN <... anchor Force

V1 = 44.33 · kN <... Vertical Force 1 acting on the Downward Direction.

V2 = 0 · kN <... Vertical Force 2 acting on the Downward Direction.

U = 96.06 · kN <... uplift force acting upwards

F_{ver_U} = 207.67 · kN <... resultant of the vertical forces without uplift

F_{ver} = 111.61 · kN <... resultant of the vertical forces with uplift

2) Horizontal Forces

Fhu = 162.74 · kN <... upstream hydrostatic force

USHF = –48.98 · kN <...hydrostatic force on gate/stoplogs

Fhd = 0 · kN <... downstream hydrostatic force

DSHF = –0.39 · kN <...horizontal force acting on the Upstream Direction

F_{hor} = 114.15 · kN <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

Ms = 430.23 · kN · m <... stabilizing moment

Mo_T = 351.31 · kN · m <... overturning moment

$Mr = 78.93 \cdot kN \cdot m$ <... resultant moment

Reference:P:\Projects\2012\12-0006-028\design\struct\Calculations and design\Calculations for Draft Reports\Stability review Dam @ Lock

Results of Rollway working together with Pier C

Note: If calcs show that combined pier / rollway would not have cracked base or the crack not extend beyond very u/s face of rollway, to remove any cracks from the rollway calculations.

$S_p := 7.62 \cdot m$ <...Applicable Span of Rollway

$L_{ave} := 7.01 \cdot m$ using pier base <...Average length of pier and rollway

Load Case 1

$A_{c1b} = 3.4\,m^2$ $F_{ver1b} = 111.61 \cdot kN$ $F_{hor1b} = 114.15 \cdot kN$ $M_{r1b} = 78.93 \cdot kN \cdot m$

$F_{ver1} := F_{ver1a} + F_{ver1b} \cdot \frac{S_p}{B} = 2552.25 \cdot kN$ $F_{hor1} := F_{hor1a} + F_{hor1b} \cdot \frac{S_p}{B} = 1137.68 \cdot kN$

$M_{r1} := M_{r1a} + M_{r1b} \cdot \frac{S_p}{B} = 5901.39 \cdot kN \cdot m$ $A_{c1} := A_{c1a} = 17.09\,m^2$

$FSS1 := \frac{\left(F_{ver1} \cdot \cos(\alpha) + F_{hor1} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_{c1}}{F_{hor1} \cdot \cos(\alpha) - F_{ver1} \cdot \sin(\alpha)} = 1.63$ $x_o := \frac{M_{r1}}{F_{ver1}} = 2.31\,m$

$L_{cr} := \begin{cases} 0 & \text{if } 3x_o \geq L_{ave} \\ \left(L_{ave} - 3x_o\right) & \text{otherwise} \end{cases}$ $L_{cr} = 0.07\,m$ <...Crack length at base

Load Case 2

$A_{c2b} = 3.4\,m^2$ $F_{ver2b} = 116.61 \cdot kN$ $F_{hor2b} = 106.5 \cdot kN$ $M_{r2b} = 139.8 \cdot kN \cdot m$

$F_{ver2} := F_{ver2a} + F_{ver2b} \cdot \frac{S_p}{B} = 2256.63 \cdot kN$ $F_{hor2} := F_{hor2a} + F_{hor2b} \cdot \frac{S_p}{B} = 1780.47 \cdot kN$

$M_{r2} := M_{r2a} + M_{r2b} \cdot \frac{S_p}{B} = 1608.66 \cdot kN \cdot m$ $A_{c2} := A_{c2a} = 4.53\,m^2$

$FSS2 := \frac{\left(F_{ver2} \cdot \cos(\alpha) + F_{hor2} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_{c2}}{F_{hor2} \cdot \cos(\alpha) - F_{ver2} \cdot \sin(\alpha)} = 0.92$ $x_o := \frac{M_{r2}}{F_{ver2}} = 0.71\,m$

$L_{cr} := \begin{cases} 0 & \text{if } 3x_o \geq L_{ave} \\ \left(L_{ave} - 3x_o\right) & \text{otherwise} \end{cases}$ $L_{cr} = 4.87\,m$ <...Crack length at base

Load Case 3

$A_{c3b} = 2.56\,m^2$ $F_{ver3b} = 91.39 \cdot kN$ $F_{hor3b} = 35.44 \cdot kN$ $M_{r3b} = 88.04 \cdot kN \cdot m$

$F_{ver3} := F_{ver3a} + F_{ver3b} \cdot \frac{S_p}{B} = 1956.75 \cdot kN$ $F_{hor3} := F_{hor3a} + F_{hor3b} \cdot \frac{S_p}{B} = 328.13 \cdot kN$

$M_{r3} := M_{r3a} + M_{r3b} \cdot \frac{S_p}{B} = 6035.14 \cdot kN \cdot m$ $A_{c3} := A_{c3a} = 17.09\,m^2$

$FSS3 := \frac{\left(F_{ver3} \cdot \cos(\alpha) + F_{hor3} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_{c3}}{F_{hor3} \cdot \cos(\alpha) - F_{ver3} \cdot \sin(\alpha)} = 4.33$ $x_o := \frac{M_{r3}}{F_{ver3}} = 3.08\,m$

$L_{cr} := \begin{cases} 0 & \text{if } 3x_o \geq L_{ave} \\ \left(L_{ave} - 3x_o\right) & \text{otherwise} \end{cases}$ $L_{cr} = 0\,m$ <...Crack length at base

Load Case 4

$A_{c4b} = 0.92\,m^2$ $F_{ver4b} = 104.68 \cdot kN$ $F_{hor4b} = 131.05 \cdot kN$ $M_{r4b} = 32.05 \cdot kN \cdot m$

$F_{ver4} := F_{ver4a} + F_{ver4b} \cdot \frac{S_p}{B} = 2441.07 \cdot kN$ $F_{hor4} := F_{hor4a} + F_{hor4b} \cdot \frac{S_p}{B} = 1417.3 \cdot kN$

$M_{r4} := M_{r4a} + M_{r4b} \cdot \frac{S_p}{B} = 4737.41 \cdot kN \cdot m$ $A_{c4} := A_{c4a} = 16.04\,m^2$

$$FSS4 := \frac{\left(F_{ver4} \cdot \cos(\alpha) + F_{hor4} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_{c4}}{F_{hor4} \cdot \cos(\alpha) - F_{ver4} \cdot \sin(\alpha)} = 1.25 \qquad x_o := \frac{M_{r4}}{F_{ver4}} = 1.94\,m$$

$$L_{cr} := \begin{cases} 0 & \text{if } 3x_o \geq L_{ave} \\ \left(L_{ave} - 3x_o\right) & \text{otherwise} \end{cases} \qquad L_{cr} = 1.19\,m$$

<...Crack length at base

Load Case 5

$A_{c5b} = 3.4\,m^2$ $F_{ver5b} = 111.61 \cdot kN$ $F_{hor5b} = 114.15 \cdot kN$ $M_{r5b} = 78.93 \cdot kN \cdot m$

$$F_{ver5} := F_{ver5a} + F_{ver5b} \cdot \frac{S_p}{B} = 2519.2 \cdot kN \qquad F_{hor5} := F_{hor5a} + F_{hor5b} \cdot \frac{S_p}{B} = 1137.68 \cdot kN$$

$$M_{r5} := M_{r5a} + M_{r5b} \cdot \frac{S_p}{B} = 5752.21 \cdot kN \cdot m \qquad A_{c5} := A_{c5a} = 15.92\,m^2$$

$$FSS5 := \frac{\left(F_{ver5} \cdot \cos(\alpha) + F_{hor5} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_{c5}}{F_{hor5} \cdot \cos(\alpha) - F_{ver5} \cdot \sin(\alpha)} = 1.61 \qquad x_o := \frac{M_{r5}}{F_{ver5}} = 2.28\,m$$

$$L_{cr} := \begin{cases} 0 & \text{if } 3x_o \geq L_{ave} \\ \left(L_{ave} - 3x_o\right) & \text{otherwise} \end{cases} \qquad L_{cr} = 0.16\,m$$

<...Crack length at base

DESIGN CALCULATIONS COVER SHEET

Project No. :	12-0006-028	**Project Name :**	Lock # 39		
File No. :		**Discipline :**	Civil Engineering		
Calculation Title :	**Lock# 39- Chamber Monolith Stability**				
Calculation No. :	CIV-005	**Prepared by :**	Y.F.	**Date :**	Feb 21, 2014
No. of Sheets :	33	**Checked by :**	H.H	**Date :**	March. 4, 2014
Supersedes Calc. No. :		**Approved by :**	H. H.	**Date :**	March 4, 2014

Calculation Description :

The wall has been reviewed against 2007 CDA Dam Safety Guidelines

Related Design Concept :

Stability analysis for the structures is carried out by using "Gravity Method"
Five loading cases are utilized in the analyses based 2007 CDA Guidelines

Reference Codes and Standards :

1.Design of Small Dams, Third Edition, U.S. Government Printing Office, Washington, D.C. 1999.
2. 2007 CDA Dam Safety Guidelines
3. 2009 Parks Canada Directive for Dam Safety Program of Dams and Water-Retaining Structures

ENGINEER'S SEAL

Rev. #	Rev. Description	Rev. Author	Date Revised	Checked by	Approved by	Approved Date

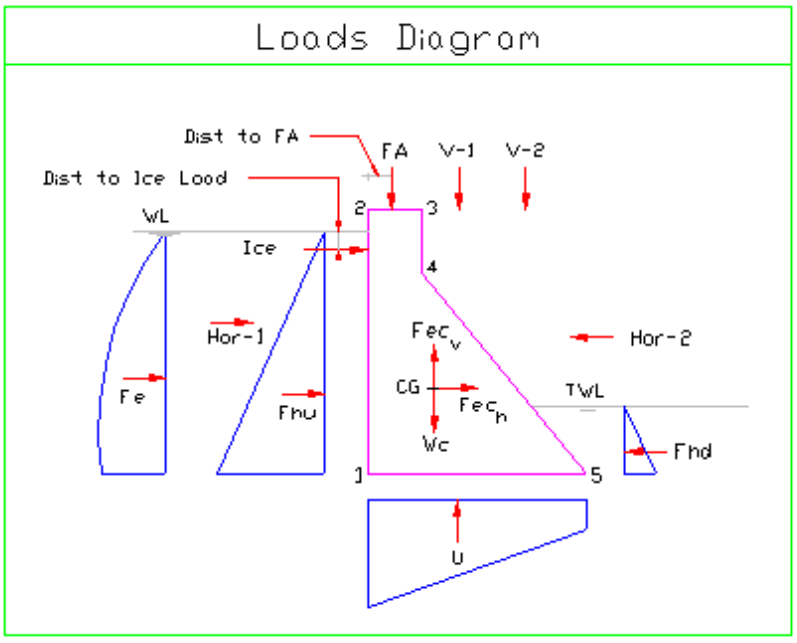
PROGRAM DESCRIPTION SECTION

This Mathcad file can calculate factors of safety against sliding and perform cracked section analysis for any shaped structure. Loading combinations conforms to CDA Dam Safety Guidelines 2007
Features

- metric or imperial units
- ice, anchor, hydrostatic and gravity loads
- crack analysis
- calculates percentage of effective base
- bearing pressures
- volume and position of center of gravity of structure (including voids)
- summary of forces and moments (including resultant and eccentricity)
- additional point loads (e.g.. backfill, vertical hydrostatic, etc.)
- hydrodynamic forces by formulas of Small Dam Design
- equivalent horizontal and vertical inertia forces on structure during earthquake

Assumptions and Limitations

- 2-dimensional
- horizontal base only
- vertical hydrostatic and backfill loads must be input as point loads
- lowest point on structure is level of base
- all vertical point loads are referenced to most upstream point of structure
- Evaluation of cracking during analyses are for unusual and extreme loading cases only.



INPUT SECTION

Unit System

1 - metric
2 - imperial

Select unit system

units := 1 <... select unit system

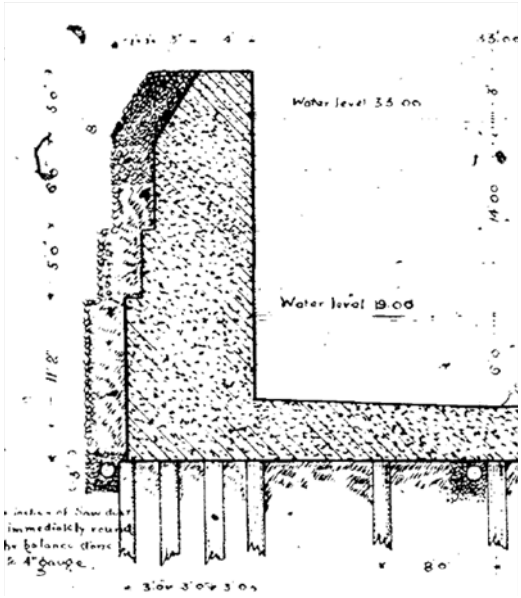
unL := if(units = 1, 1 · m, 1 · ft)

unM := if(units = 1, 1 · kN, 1 · lb)

Geometry of Structure defined by perimeter coordinates

There is no need to "close" the perimeter ,i.e., so that last point equal to first point

$$X := \begin{pmatrix} 0 \\ 0 \\ 0.3 \\ 0.3 \\ 0.61 \\ 0.61 \\ 1.52 \\ 2.74 \\ 2.74 \\ 0 \end{pmatrix} \cdot unL \qquad Y := \begin{pmatrix} 224.23 \\ 226.37 \\ 226.37 \\ 227.89 \\ 227.89 \\ 229.72 \\ 231.24 \\ 231.24 \\ 224.23 \\ 224.23 \end{pmatrix} \cdot unL$$



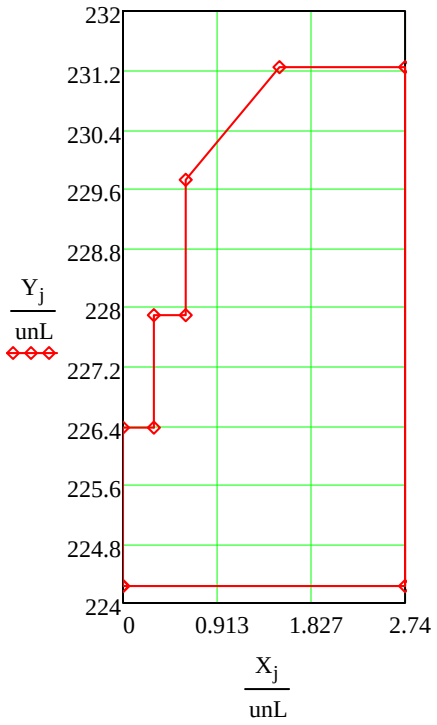
<... input X & Y coordinates

$$maxY := \frac{max(Y)}{unL} + 5 \qquad minY := \frac{min(Y)}{unL} - 5 \qquad i := 1 .. length(X)$$

$$maxX := \frac{max(X)}{unL} + 5 \qquad minX := \frac{min(X)}{unL} - 5 \qquad j := 1 .. length(X) + 1$$

$$X_{length(X)+1} := X_1 \qquad Y_{length(Y)+1} := Y_1$$

Graphical Representation of Structure defined by X & Y Coordinates



Densities of Water and Concrete, Width of Structure, of Structure defined by perimeter coordinates

$\gamma w := \text{if}\left(\text{units} = 1, 9.81 \cdot \frac{kN}{m^3}, 62.5 \cdot \frac{lb}{ft^3}\right)$ <... water density

$\gamma := \text{if}\left(\text{units} = 1, 18 \cdot \frac{kN}{m^3}, 115 \cdot \frac{lb}{ft^3}\right)$ <... backfill

$\gamma_s := \text{if}\left(\text{units} = 1, 9.1 \cdot \frac{kN}{m^3}, 49 \cdot \frac{lb}{ft^3}\right)$ <... Saturated Backfill density

$\gamma c := \text{if}\left(\text{units} = 1, 23.5 \cdot \frac{kN}{m^3}, 145 \cdot \frac{lb}{ft^3}\right)$ <... concrete density

$f'_c := \text{if}(\text{units} = 1, 17 \cdot MPa, 2030 \cdot psi)$ <... concrete compressive strength

$f'_{rock} := \text{if}(\text{units} = 1, 30 \cdot MPa, 4350 \cdot psi)$ <... mass rock compressive strength

$Structure_Width := 1.0$ <... width of structure

$\phi := 36 \cdot deg$ <... angle of internal friction for foundation material

$c := 0 \cdot f'_c$ $\quad c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $\quad f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

Computation of Area and Coordinates of Center of Gravity of Structure

$deltax_i := X_{i+1} - X_i$ $\quad deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\quad yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot \left(deltay_i \cdot xplusx_i\right)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[\left(yplusy_i\right)^2 + \frac{\left(deltay_i\right)^2}{3} \right] \qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[\left(xplusx_i\right)^2 + \frac{\left(deltax_i\right)^2}{3} \right]$$

$$Area := \sum_i Areainc_i \qquad Area := if\left[\frac{Area}{unL^2} > 0, Area, Area \cdot (-1)\right]$$

<... Area of the Structure (defined by X & Y)

$$Xg := \sum_i Xginc_i \qquad Xg := \frac{if\left[\frac{Xg}{unL^3} > 0, Xg, Xg \cdot (-1)\right]}{Area} = 1.56\,m$$

<... X coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$$Yg := \sum_i Yginc_i \qquad Yg := \frac{if\left[\frac{Yg}{unL^3} > 0, Yg, Yg \cdot (-1)\right]}{Area} = 227.39\,m$$

<... Y coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$B := Structure_Width \cdot (unL) = 1\,m$ $\quad L := \max(X) - \min(X) = 2.74\,m$ $\quad H := \max(Y) - \min(Y) = 7.01\,m$

Computation of Concrete Volume and Weight in Structure

$Vol_conc := Area \cdot B$ $\quad Vol_conc = 16.02 \cdot m^3$

<... volume of concrete in structure

$Wc := Area \cdot B \cdot \gamma c$ $\quad Wc = 376.4 \cdot kN$

<... weight of concrete in structure

Load Case 1. Backfill in wet condition (Chamber filled with upper pool)

US_Water_Level := 228.3 <... Pizo head of backfill

Tailwater_Level := 230.34 <... Water level in chamber

SF := 2.0 <... Minimum Safety Factor

$c := 0 \cdot f'_c$ $c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

US_L_ELE := 224.23 <...Lowest Dam Elevation at upstream

DS_L_ELE := 224.23 <...Lowest Dam Elevation at downstream

$$\alpha := \operatorname{atan}\left(\frac{DS_L_ELE - US_L_ELE}{\frac{L}{unL}}\right) = 0 \cdot deg$$ <...Angle of base

$$L := \frac{L}{\cos(\alpha)} = 2.74\,m$$

Weight of soil carried by concrete wall

$W_{soil} := \gamma \cdot (H \cdot L \cdot 1.0m - Vol_conc) = 57.44 \cdot kN$ <... Weight of soil

$E_{soil} := 231.24 \cdot m - \frac{1}{3} \cdot (231.24 \cdot m - 226.37 \cdot m) = 229.62\,m$ <... Elevation of soil gravity center

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0.3 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0 <...Hydrostatic force on Gate / Stoplogs

EL_US_Hor_F := 0 <...Elevation of the Hydrostatic force on Gate / Stoplogs

US_Hor_Force1 := 0 <... Ice Load on gate / stoplogs

EL_US_Hor_F1 := 0 <...Elevation of the ice load

$i := 0 \cdot deg$ <... backfill slope

$\phi' := 19.5 \cdot deg$ Assumed for gatting at-rest coef. <... angle of internal friction for backfill

$K_a := \frac{1 - \sin(\phi')}{1 + \sin(\phi')} = 0.5$ <... Active pressure

SILT_ELE := 231.24 <... top elevation of backfill

BOT_ELE := 224.23 <...bottom elevation of backfill

$H_{soil} := (SILT_ELE - BOT_ELE) \cdot unL = 7.01\,m$

$h_{sur} := 0.25 \cdot m$ consider 0.25 m surcharge on top of backfill

$q := \gamma \cdot h_{sur} = 4.5 \cdot kPa$

$H_{sur} := q \cdot H_{soil} \cdot K_a \cdot B = 15.76 \cdot kN$ Lateral force due to surcharge

$y_{sur} := \frac{H_{soil}}{2} = 3.51\,m$ Acting at

Lateral force due to backfill above water

$h'' := SILT_ELE \cdot unL - US_Water_Level \cdot unL = 2.94\,m$ backfill height above water

$h''' := (US_Water_Level - BOT_ELE) \cdot unL = 4.07\,m$ backfill submerged in water

$H_{b1} := \frac{1}{2} \cdot \gamma \cdot K_a \cdot (h'')^2 \cdot B = 38.85 \cdot kN$

$y_{b1} := h''' + \frac{h''}{3} = 5.05\,m$

Lateral force due to backfill submerged in water

$$H_{b2} := \frac{1}{2} \cdot \left[\gamma \cdot h'' + \left(\gamma \cdot h'' + \gamma_s \cdot h'''\right)\right] \cdot h''' \cdot K_a \cdot B = 145.22 \cdot kN$$

$$y_{b2} := h''' - \frac{h''' \cdot \left[2\left(\gamma \cdot h'' + \gamma_s \cdot h'''\right) + \gamma \cdot h''\right]}{3 \cdot \left(\gamma \cdot h'' + \gamma_s \cdot h''' + \gamma \cdot h''\right)} = 1.86\,m$$

Lateral force due to all backfill and surcharge

$$Hb := \left(H_{b1} + H_{b2}\right) + H_{sur} = 199.83 \cdot kN$$

$$y_a := \frac{H_{sur} \cdot y_{sur} + H_{b1} \cdot y_{b1} + H_{b2} \cdot y_{b2}}{H_{sur} + H_{b1} + H_{b2}} = 2.61\,m$$

$$E_{BF} := BOT_ELE + \frac{y_a}{unL} = 226.84$$

$$DS_Hor_Force := \frac{-Hb}{unM} = -199.83$$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$$EL_DS_Hor_F := E_{BF}$$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$$Vert_Force_1 := \frac{W_{soil}}{unM}$$

<... Weight of soil

$$Dist_to_F1 := 0.51$$

<... distance from most upstream point of structure to point of application of vertical force

$$Vert_Force_2 := 0$$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$$Dist_to_F2 := 0$$

<... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$$WL := US_Water_Level \cdot (unL) \qquad F_EL := US_L_ELE \cdot (unL) \qquad F_EL_D := DS_L_ELE \cdot (unL)$$

$$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL) \qquad F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$$

$$F_ANC := Anchor_Force \cdot (unM) \qquad Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$ $EL_USF := EL_US_Hor_F \cdot (unL)$

$USHF1 := US_Hor_Force1 \cdot (unM)$ $EL_USF1 := EL_US_Hor_F1 \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$ $EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 81.25 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 183.11 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Net uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2} = -27.42 \cdot kN$ <... Net uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B = 164.23 \cdot kN$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 136.82 \cdot kN$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF + USHF1 - DSHF)$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U = 297.01 \cdot kN$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2 = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 945.04 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$

<... Stabilizing Moment

$$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL - F_EL_D)\right] + Fi \cdot (EL_ICE - F_EL_D) \ldots = 631.66 \cdot kN \cdot m$$
$$+ USHF \cdot (EL_USF - F_EL_D) + USHF1 \cdot (EL_USF1 - F_EL_D) \ldots$$
$$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$$

<... Overturning Moment (uplift not included)

$$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2} = 174.92 \cdot kN \cdot m$$

<... Overturning Moment due to Uplift

$$Mo_T := Mo_1 + Mo_2$$

<... Total Overturning Moment

$$Mr := Ms - Mo_T$$

<... Resultant Moment

$$x_o := \frac{Mr}{F_{ver}} = 0.47\,m$$

<...Resultant location. Distance from the toe.

$$E := \frac{L}{2} - x_o$$

<...Eccentricity of Resultant

$$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 322.93 \cdot kPa \qquad q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -106.14 \cdot kPa$$

$$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$$

Calculation_to_be = "stoped"

(To ensure uncracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases}$$

$L_t = 0\,m$

<...Length of base under tention

$$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$$

$PI = 1.34\,m$

$$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases} = 1.4\,m$$

<...Length of base under compression

$$A_t := L_t \cdot B \qquad A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases} \qquad A_c = 1.4\,m^2$$

$L_1 := L_c + L_t = 1.4\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot \left| q_{1min} \right| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t \right) \right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 297.01 \cdot kN$ $Mr := Ms - Mo_T = 138.46 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 0.47\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 0.9\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L} \right) = 322.93 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L} \right) = -106.14 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 424.74 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 1.34\,m$ <...Point of inflection of stresses at foundation from heel

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha) \right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 2.2$ <...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 1

Initial point of inflection

$PI = 1.34 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 48.96 \cdot \%$

$$Result := \begin{cases} \text{"ok"} & \text{if } E \le \frac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

$q_{1min} = -106.1 \cdot kPa$

When c is taken into account

$$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \ge \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 424.74 \cdot kPa$

$x_o = 0.47\ m$

<...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \le 0.3 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

$FSS = 2.2$

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "OK"

When c is taken into account

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "OK"

Summary of Forces and Moments

1) Vertical Forces

$Wc = 376.38 \cdot kN$

<... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 57.44 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U = 136.82 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 297.01 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 81.25 \cdot kN$ <... upstream hydrostatic force

$Fhd = 183.11 \cdot kN$ <... downstream hydrostatic force

$Fi = 0 \cdot kN$ <... Ice Force

$USHF = 0 \cdot kN$ <...horizontal force **due to hydrostatic pressure from stoplogs**

$USHF1 = 0 \cdot kN$ <...horizontal force **due to ice pressure from stoplogs**

$DSHF = -199.83 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 97.97 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 945.04 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 806.58 \cdot kN \cdot m$ <... overturning moment

$Mr = 138.46 \cdot kN \cdot m$ <... resultant moment

Load Case 4. Extreme Loading Earthquake (Chamber filled with upper pool)

Horizontal & Vertical Components of the earthquake Intensity and Coefficient of Hydrodynamic Pressure Distribution

$\lambda_{hor} := 0.05$ <... **horizontal** component of earthquake intensity = ratio of earthquake acceleration to acceleration due to gravity

$\lambda_{vert} := \frac{2}{3} \cdot \lambda_{hor}$ <... **vertical** component of earthquake intensity

<... from owner's requirement 21.2.5.1

$C_m := 0.74$ <... from fig 8-6, page 325 of "Design of Small Dams", for a structure with

vertical face

SF := 1.3 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 0 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **acceleration,** i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0 <...Hydrostatic on stoplogs

EL_US_Hor_F := 0 <...Elevation at which the Upstream Horizontal Force is applied to the Structure

$\zeta := \operatorname{atan}(\lambda_{hor}) = 2.86 \cdot deg$

$K_a := \frac{1 - \sin(\phi')}{1 + \sin(\phi')} = 0.5$ <...Active Pressure

K_{ae} = Coefficient for active earth pressure and seismic thrust

$$K_{ae} := \frac{\left[\left(\cos(i-\zeta) - \sqrt{\cos(i+\zeta)^2 - \cos(\phi')^2}\right)^2 + (\sin(i+\zeta) - \sin(i-\zeta))^2\right]^{\frac{1}{2}}}{\cos(\zeta)\cdot\left(\cos(i+\zeta) + \sqrt{\cos(i+\zeta)^2 - \cos(\phi')^2}\right)}$$

$K_{ae} = 0.509$ Using Mononobe-Rankine K_{ae}

$DS_Hor_Force := \frac{-Hb}{unM} \cdot \left(\frac{K_{ae}}{K_a}\right) = -203.83$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{BF}$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **acceleration,** i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{soil}}{unM}$

<... Weight of soil

$Dist_to_F1 := 0.51$

<... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := 0$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := 0$

<... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$

$F_EL := US_L_ELE \cdot (unL)$

$F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$

$F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$

$EL_USF := EL_US_Hor_F \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$

$EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$

$TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces and Uplift

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 81.25 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 183.11 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 164.23 \cdot kN$ <... Total Uplift force

Computation of Hydrodynamic Force Due to Earthquake

$h := US_Water_Level \cdot (unL) - F_EL$ $h = 4.07\,m$ <...total depth of the reservoir at section being analyses

$y := h$ <....vertical distance from reservoir surface to the elevation in question

$C := \frac{C_m}{2} \cdot \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right) + \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right)\right]^{0.5}\right]$ <...Coefficient of Hydrodynamic Pressure Distribution

$Pe := C \cdot \lambda_{hor} \cdot \gamma w \cdot h \cdot Structure_Width \cdot (unL)$ $Pe = 1.48 \cdot \frac{kN}{m}$ <...hydrodynamic pressure

$Fe1 := 0.726 \cdot Pe \cdot y \cdot \left(\frac{0}{100}\right)$ $Fe1 = 0 \cdot kN$ <...hydrodynamic force at pier

void ratio 0% assumed

$Me := 0.299 \cdot Pe \cdot y^2 \cdot \left(\frac{0}{100}\right)$ $Me = 0 \cdot kN \cdot m$ <...moment at the base of the structure due to hydrodynamic force

$P_{appl} := \begin{cases} F_EL + \frac{Me}{Fe1} & \text{if } Fe1 > 0kN \\ 0 \cdot m & \text{otherwise} \end{cases}$ $P_{appl} = 0 \cdot m$ <...elevation of point of application of hydrodynamic force

$Sp_{gate} := 0$ $W_{gate} := 0$

$Fe2 := 0.726 \cdot Pe \cdot y \cdot \frac{Sp_{gate}}{(Structure_Width)}$ $Fe2 = 0 \cdot kN$ <...hydrodynamic force at gate/ stoplogs

$Fe := Fe1 + Fe2$ $Fe = 0 \cdot kN$ <...Total hydrodynamic force

$Fec1_h := Wc \cdot \lambda_{hor}$ $Fec1_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec_v := Wc \cdot \lambda_{vert}$ $Fec_v = 12.55 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec2_h := W_{gate} \cdot \lambda_{hor}$ $Fec2_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Gate/stologs due to earthquake

$Fec_h := Fec1_h + Fec2_h$ $Fec_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on pier and gate due to earthquake

$Fec3_h := W_{soil} \cdot \lambda_{hor}$ $Fec3_h = 2.87 \cdot kN$ <... Equivalent Horizontal Inertia Force on Soil due to earthquake

$Fec3_v := W_{soil} \cdot \lambda_{vert}$ $Fec3_v = 1.91 \cdot kN$ <... Equivalent Vertical Inertia Force on Soil due to earthquake

$F_ANC := Anchor_Force \cdot (unM)$ $Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$F_{hor} := (Fhu - Fhd) + Fe + Fec_h + Fec3_h + (USHF - DSHF) = 123.66 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - (U + Fec_v + Fec3_v) = 255.13 \cdot kN$ <... Resultant of Vertical Forces

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 945.04 \cdot kN \cdot m$ <... Stabilizing Moment
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL - F_EL_D)\right] + Fe \cdot (P_{appl} - F_EL_D) \ldots$
$+ Fec_h \cdot (Yg - F_EL_D) + Fec_v \cdot (L - Xg) + USHF \cdot (EL_USF - F_EL_D) \ldots$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$
$Mo_1 = 716.34 \cdot kN \cdot m$

<... Overturning Moment (uplift and EQ due to slab/tower inertial force not included)

$Mo_{11} := Fec3_h \cdot (E_{soil} - F_EL_D) + Fec3_v \cdot (L - Dist_to_F1 \cdot unL)$ <... Overturning Moment due to slab and tower inertial force

$Mo_{11} = 19.74 \cdot kN \cdot m$

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ $Mo_2 = 225 \cdot kN \cdot m$ <... Overturning Moment due to Uplift

$Mo_T := (Mo_1 + Mo_{11}) + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ $x_o = -0.06\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ $E = 1.43\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 385.27 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -199.05 \cdot kPa$

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 2.93\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases}$ $= -0.19\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 0\,m^2$

$L_1 := L_c + L_t = -0.19\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 255.13 \cdot kN$ $Mr := Ms - Mo_T = -16.04 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = -0.06\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.43\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 385.27 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -199.05 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = -2704.6 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 2.93\,m$ <...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$ <...Stress at Foundation

$SF_{ot} := \frac{Ms}{Mo_T} = 0.98$

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.5$ <...Factor of Safety Against Sliding.

RESULTS for Loading case 4

Initial point of inflection

$PI = 2.93 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 106.89 \cdot \%$

$EFF_BASE := 1 - \left(\frac{PI}{L}\right)$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = -7 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \cdot \% \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$ <...Resultant required to be is within base.

Result = "to increase base size"

STRESSES AT FOUNDATION

fmin = 0 · kPa

fmax = −2704.6 · kPa

$f := \max(fmin, fmax)$

$x_o = -0.06\,m$

$Normal_stresses := \begin{cases} "OK" & \text{if } f \le 0.9 \cdot \min(f'_c, f'_{rock}) \\ "not\ good" & otherwise \end{cases}$

<... Allowable compressive strength =0.9 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

Not Required. The Earthquake load case is used to establish post-earthquake condition of the dam

Summary of Forces and Moments

1) Vertical Forces

Wc = 376.38 · kN <... weight of concrete in structure

Fa = 0 · kN <... anchor Force

V1 = 57.44 · kN <... Vertical Force 1 acting on the Downward Direction.

V2 = 0 · kN <... Vertical Force 2 acting on the Downward Direction.

$Fec_v = 12.55 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec3_v = 1.91 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

U = 164.23 · kN <... uplift force acting upwards

$F_{ver_U} = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 255.13 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

Fhu = 81.25 · kN <... upstream hydrostatic force

USHF = 0 · kN <...hydrostatic force on gate/stoplogs

Fhd = 183.11 · kN <... downstream hydrostatic force

Fe = 0 · kN <...hydrodynamic force on pier + gate/stoplogs

$Fec_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec3_h = 2.87 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$DSHF = -203.83 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 123.66 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 945.04 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 961.08 \cdot kN \cdot m$ <... overturning moment

$Mr = -16.04 \cdot kN \cdot m$ <... resultant moment

Load Case 1.1 Backfill in wet condition (Chamber filled with lower pool)

US_Water_Level := 228.3 <... Pizo head of backfill

Tailwater_Level := 226.04 <... Water level in chamber

SF := 2.0 <... Minimum Safety Factor

$c := 0 \cdot f'_c$ $c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0.3 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0 <...Hydrostatic force on Gate / Stoplogs

EL_US_Hor_F := 0 <...Elevation of the Hydrostatic force on Gate / Stoplogs

US_Hor_Force1 := 0 <... Ice Load on gate / stoplogs

EL_US_Hor_F1 := 0 <...Elevation of the ice load

DS_Hor_Force := $\frac{-Hb}{unM} = -199.83$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

EL_DS_Hor_F := E_{BF}

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

Vert_Force_1 := $\frac{W_{soil}}{unM}$

<... Weight of soil

Dist_to_F1 := 0.51

<... distance from most upstream point of structure to point of application of vertical force

Vert_Force_2 := 0

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

Dist_to_F2 := 0

<... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

WL := US_Water_Level · (unL)

F_EL := US_L_ELE · (unL)

F_EL_D := DS_L_ELE · (unL)

EL_ICE := WL − Dist_WL_to_Ice_Load · (unL)

F_ICE := Ice_Load · $\left(\frac{unM}{unL}\right)$

F_ANC := Anchor_Force · (unM)

Anc_Dist := L − Dist_to_Anc_Force · (unL)

V1 := Vert_Force_1 · (unM)

Dist_V1 := L − Dist_to_F1 · (unL)

V2 := Vert_Force_2 · (unM)

Dist_V2 := L − Dist_to_F2 · (unL)

USHF := US_Hor_Force · (unM)

EL_USF := EL_US_Hor_F · (unL)

USHF1 := US_Hor_Force1 · (unM)

EL_USF1 := EL_US_Hor_F1 · (unL)

DSHF := DS_Hor_Force · (unM)

EL_DSF := EL_DS_Hor_F · (unL)

Ueff := Uplift · %

TWL := Tailwater_Level · (unL)

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 81.25 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 16.07 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd = 22.17 \cdot kPa$ <... Net uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2} = 30.37 \cdot kN$ <... Net uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B = 48.65 \cdot kN$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 79.03 \cdot kN$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF + USHF1 - DSHF)$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U = 354.8 \cdot kN$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2 = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 581.79 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL - F_EL_D)\right] + Fi \cdot (EL_ICE - F_EL_D) \ldots = 631.66 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) + USHF1 \cdot (EL_USF1 - F_EL_D) \ldots$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2} = 122.14 \cdot kN \cdot m$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}} = -0.48\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 655.42 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -396.44 \cdot kPa$

$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$

Calculation_to_be = "stoped"

(To ensure uncracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 4.19\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases}$ $= -1.45\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 0\,m^2$

$L_1 := L_c + L_t = -1.45\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 354.8 \cdot kN$ $Mr := Ms - Mo_T = -172.01 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = -0.48\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.85\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 655.42 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -396.44 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = -487.89 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$

<...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 4.19\,m$ <...Point of inflection of stresses at foundation from heel

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.97$ <...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 1

Initial point of inflection

$PI = 4.19 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 153.08 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } E \leq \frac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

$q_{1min} = -396.4 \cdot kPa$

When c is taken into account

$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \geq \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

fmin = 0 · kPa

fmax = −487.89 · kPa

$x_o = -0.48\,m$ <...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \le 0.3 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

FSS = 0.97

$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$

Safety_against_sliding = "to increase dam

When c is taken into account

$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$

Safety_against_sliding = "to increase dam s

Summary of Forces and Moments

1) Vertical Forces

$Wc = 376.38 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 57.44 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U = 79.03 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 354.8 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 81.25 \cdot kN$ <... upstream hydrostatic force

$Fhd = 16.07 \cdot kN$ <... downstream hydrostatic force

$Fi = 0 \cdot kN$ <... Ice Force

$USHF = 0 \cdot kN$ <...horizontal force **due to hydrostatic pressure from stoplogs**

$USHF1 = 0 \cdot kN$ <...horizontal force **due to ice pressure from stoplogs**

$DSHF = -199.83 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 265.01 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 581.79 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 753.8 \cdot kN \cdot m$ <... overturning moment

$Mr = -172.01 \cdot kN \cdot m$ <... resultant moment

Load Case 4.1 Extreme Loading Earthquake (Chamber filled with lower pool)

Horizontal & Vertical Components of the earthquake Intensity and Coefficient of Hydrodynamic Pressure Distribution

$\lambda_{hor} := 0.05$ <... **horizontal** component of earthquake intensity = ratio of earthquake acceleration to acceleration due to gravity

$\lambda_{vert} := \frac{2}{3} \cdot \lambda_{hor}$ <... **vertical** component of earthquake intensity

<... from owner's requirement 21.2.5.1

$C_m := 0.74$ <... from fig 8-6, page 325 of "Design of Small Dams", for a structure with **vertical face**

$SF := 1.3$ <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

$Uplift := 0$ <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

$Ice_Load := 0$ <... ice load acting per unit width of the structure

$Dist_WL_to_Ice_Load := 0$ <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **acceleration,** i.e., metric units, Anchor Force will be in

kN

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0 <...Hydrostatic on stoplogs

EL_US_Hor_F := 0 <...Elevation at which the Upstream Horizontal Force is applied to the Structure

$\zeta := \operatorname{atan}(\lambda_{hor}) = 2.86 \cdot deg$

$K_a := \frac{1 - \sin(\phi')}{1 + \sin(\phi')} = 0.5$ <...Active Pressure

K_{ae} = Coefficient for active earth pressure and seismic thrust

$$K_{ae} := \frac{\left[\left(\cos(i - \zeta) - \sqrt{\cos(i + \zeta)^2 - \cos(\phi')^2}\right)^2 + (\sin(i + \zeta) - \sin(i - \zeta))^2\right]^{\frac{1}{2}}}{\cos(\zeta) \cdot \left(\cos(i + \zeta) + \sqrt{\cos(i + \zeta)^2 - \cos(\phi')^2}\right)}$$

$K_{ae} = 0.509$ Using Mononobe-Rankine K_{ae}

$DS_Hor_Force := \frac{-Hb}{unM} \cdot \left(\frac{K_{ae}}{K_a}\right) = -203.83$ <... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

EL_DS_Hor_F := $E_{BF} = 226.84$ <...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **acceleration,** i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{soil}}{unM}$ <... Weight of soil

Dist_to_F1 := 0.51 <... distance from most upstream point of structure to point of application of vertical

force

Vert_Force_2 := 0 <... Vertical Force acting on the Downward Direction. It is a stabilizing force

Dist_to_F2 := 0 <... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

WL := US_Water_Level · (unL)

F_EL := US_L_ELE · (unL)

F_EL_D := DS_L_ELE · (unL)

EL_ICE := WL − Dist_WL_to_Ice_Load · (unL)

$$F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$$

F_ANC := Anchor_Force · (unM)

Anc_Dist := L − Dist_to_Anc_Force · (unL)

V1 := Vert_Force_1 · (unM)

Dist_V1 := L − Dist_to_F1 · (unL)

V2 := Vert_Force_2 · (unM)

Dist_V2 := L − Dist_to_F2 · (unL)

USHF := US_Hor_Force · (unM)

EL_USF := EL_US_Hor_F · (unL)

DSHF := DS_Hor_Force · (unM)

EL_DSF := EL_DS_Hor_F · (unL)

Ueff := Uplift · %

TWL := Tailwater_Level · (unL)

Computation of Intensity of Forces and Uplift

Phu := (WL − F_EL) · γw <... upstream hydrostatic pressure

Phd := (TWL − F_EL_D) · γw <... downstream hydrostatic pressure

$$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 81.25 \cdot kN$$ <... upstream hydrostatic force

$$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 16.07 \cdot kN$$ <... downstream hydrostatic force

Fi := F_ICE · B <... Ice Force

Fa := F_ANC <... Anchor Force

Pu := Phu − Phd <... Uplift pressure at upstream end

$$U_w := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 48.65 \cdot kN$ <... Total Uplift force

Computation of Hydrodynamic Force Due to Earthquake

$h := US_Water_Level \cdot (unL) - F_EL$ $h = 4.07\,m$ <...total depth of the reservoir at section being analyses

$y := h$ <....vertical distance from reservoir surface to the elevation in question

$$C := \frac{C_m}{2} \cdot \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right) + \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right)\right]^{0.5}\right]$$ <...Coefficient of Hydrodynamic Pressure Distribution

$Pe := C \cdot \lambda_{hor} \cdot \gamma w \cdot h \cdot Structure_Width \cdot (unL)$ $Pe = 1.48 \cdot \frac{kN}{m}$ <...hydrodynamic pressure

$Fe1 := 0.726 \cdot Pe \cdot y \cdot \left(\frac{0}{100}\right)$ $Fe1 = 0 \cdot kN$ <...hydrodynamic force at pier

void ratio 0% assumed

$Me := 0.299 \cdot Pe \cdot y^2 \cdot \left(\frac{0}{100}\right)$ $Me = 0 \cdot kN \cdot m$ <...moment at the base of the structure due to hydrodynamic force

$$P_{appl} := \begin{cases} F_EL + \frac{Me}{Fe1} & \text{if } Fe1 > 0kN \\ 0 \cdot m & \text{otherwise} \end{cases}$$ $P_{appl} = 0 \cdot m$ <...elevation of point of application of hydrodynamic force

$Sp_{gate} := 0$ $W_{gate} := 0$

$Fe2 := 0.726 \cdot Pe \cdot y \cdot \frac{Sp_{gate}}{(Structure_Width)}$ $Fe2 = 0 \cdot kN$ <...hydrodynamic force at gate/ stoplogs

$Fe := Fe1 + Fe2$ $Fe = 0 \cdot kN$ <...Total hydrodynamic force

$Fec1_h := Wc \cdot \lambda_{hor}$ $Fec1_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec_v := Wc \cdot \lambda_{vert}$ $Fec_v = 12.55 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec2_h := W_{gate} \cdot \lambda_{hor}$ $Fec2_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Gate/stologs due to earthquake

$Fec_h := Fec1_h + Fec2_h$ $Fec_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on pier and gate due to earthquake

$Fec3_h := W_{soil} \cdot \lambda_{hor}$ $Fec3_h = 2.87 \cdot kN$ <... Equivalent Horizontal Inertia Force on Soil due to earthquake

$Fec3_v := W_{soil} \cdot \lambda_{vert}$ $Fec3_v = 1.91 \cdot kN$ <... Equivalent Vertical Inertia Force on Soil due to earthquake

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$F_{hor} := (Fhu - Fhd) + Fe + Fec_h + Fec3_h + (USHF - DSHF) = 290.7 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - \left(U + Fec_v + Fec3_v\right) = 370.71 \cdot kN$ <... Resultant of Vertical Forces

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 581.79 \cdot kN \cdot m$ <... Stabilizing Moment
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL - F_EL_D)\right] + Fe \cdot \left(P_{appl} - F_EL_D\right) \ldots$
$+ Fec_h \cdot (Yg - F_EL_D) + Fec_v \cdot (L - Xg) + USHF \cdot (EL_USF - F_EL_D) \ldots$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$

<... Overturning Moment (uplift and EQ due to slab/tower inertial force not included)

$Mo_1 = 716.34 \cdot kN \cdot m$

$Mo_{11} := Fec3_h \cdot \left(E_{soil} - F_EL_D\right) + Fec3_v \cdot (L - Dist_to_F1 \cdot unL)$ <... Overturning Moment due to slab and tower inertial force

$Mo_{11} = 19.74 \cdot kN \cdot m$

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ $Mo_2 = 67 \cdot kN \cdot m$ <... Overturning Moment due to Uplift

$Mo_T := \left(Mo_1 + Mo_{11}\right) + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ $x_o = -0.6\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ $E = 1.97\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 717.76 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -447.17 \cdot kPa$

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 4.53\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases}$ $= -1.79\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 0\,m^2$

$L_1 := L_c + L_t = -1.79\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 370.71 \cdot kN$ $Mr := Ms - Mo_T = -220.94 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = -0.6\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.97\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 717.76 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -447.17 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $= -414.66 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases}$ $= 0 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 4.53\,m$

<...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \dfrac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$

$fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$

<...Stress at Foundation

$SF_{ot} := \dfrac{Ms}{Mo_T} = 0.72$

$FSS := \dfrac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.93$

<...Factor of Safety Against Sliding.

RESULTS for Loading case 4.1

Initial point of inflection

$PI = 4.53 \cdot m$

Initial point of inflection as % of the base

$\dfrac{PI}{L} = 165.26 \cdot \%$

$EFF_BASE := 1 - \left(\dfrac{PI}{L}\right)$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = -65 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \cdot \% \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant required to be is within base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = -414.7 \cdot kPa$

$f := \max(fmin, fmax)$

$x_o = -0.6\,m$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.9 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.9 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

Not Required. The Earthquake load case is used to establish post-earthquake condition of the dam

Summary of Forces and Moments

1) Vertical Forces

$Wc = 376.38 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 57.44 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$Fec_v = 12.55 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec3_v = 1.91 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

$U = 48.65 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 370.71 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 81.25 \cdot kN$ <... upstream hydrostatic force

$USHF = 0 \cdot kN$ <...hydrostatic force on gate/stoplogs

$Fhd = 16.07 \cdot kN$ <... downstream hydrostatic force

$Fe = 0 \cdot kN$ <...hydrodynamic force on pier + gate/stoplogs

$Fec_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec3_h = 2.87 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$DSHF = -203.83 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 290.7 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 581.79 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 802.74 \cdot kN \cdot m$ <... overturning moment

$Mr = -220.94 \cdot kN \cdot m$ <... resultant moment

Load Case 1a. Backfill in wet condition (Chamber to be dewatered)

US_Water_Level := 228.3 <... Pizo head of backfill

Tailwater_Level := 224.23 <... Water level in chamber

SF := 2.0 <... Minimum Safety Factor

$c := 0 \cdot f'_c$ $c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0.3 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0 <...Hydrostatic force on Gate / Stoplogs

EL_US_Hor_F := 0 <...Elevation of the Hydrostatic force on Gate / Stoplogs

US_Hor_Force1 := 0 <... Ice Load on gate / stoplogs

EL_US_Hor_F1 := 0 <...Elevation of the ice load

$DS_Hor_Force := \frac{-Hb}{unM} = -199.83$ <... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

EL_DS_Hor_F := E_{BF} = 226.84 <...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

Vert_Force_1 := $\frac{W_{soil}}{unM}$ <... Weight of soil

Dist_to_F1 := 0.51 <... distance from most upstream point of structure to point of application of vertical force

Vert_Force_2 := 0 <... Vertical Force acting on the Downward Direction. It is a stabilizing force

Dist_to_F2 := 0 <... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

WL := US_Water_Level · (unL)

F_EL := US_L_ELE · (unL)

F_EL_D := DS_L_ELE · (unL)

EL_ICE := WL − Dist_WL_to_Ice_Load · (unL)

F_ICE := Ice_Load · $\left(\frac{unM}{unL}\right)$

F_ANC := Anchor_Force · (unM)

Anc_Dist := L − Dist_to_Anc_Force · (unL)

V1 := Vert_Force_1 · (unM)

Dist_V1 := L − Dist_to_F1 · (unL)

V2 := Vert_Force_2 · (unM)

Dist_V2 := L − Dist_to_F2 · (unL)

USHF := US_Hor_Force · (unM)

EL_USF := EL_US_Hor_F · (unL)

USHF1 := US_Hor_Force1 · (unM)

EL_USF1 := EL_US_Hor_F1 · (unL)

DSHF := DS_Hor_Force · (unM)

EL_DSF := EL_DS_Hor_F · (unL)

Ueff := Uplift · %

TWL := Tailwater_Level · (unL)

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

Phu := (WL − F_EL) · γw <... upstream hydrostatic pressure

Phd := (TWL − F_EL_D) · γw <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = \blacksquare \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Net uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Net uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 54.7 \cdot kN$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF + USHF1 - DSHF)$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 572.1 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL - F_EL_D)\right] + Fi \cdot (EL_ICE - F_EL_D) \ldots = 631.66 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) + USHF1 \cdot (EL_USF1 - F_EL_D) \ldots$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}} = -0.42\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 680.92 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -404.19 \cdot kPa$

$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$

Calculation_to_be = "stoped"

(To ensure uncracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 4\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \\ (L - PI) & \text{otherwise} \end{cases}$ $= -1.26\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 0\,m^2$

$L_1 := L_c + L_t = -1.26\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 379.12 \cdot kN$ $Mr := Ms - Mo_T = -159.48 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = -0.42\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.79\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 680.92 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -404.19 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = -600.83 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$

<...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 4\,m$

<...Point of inflection of stresses at foundation from heel

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.98$

<...Factor of Safety Against Sliding.

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

$\Sigma M := Ms - Mo_1$

$\Sigma W := Wc + Fa + V1 + V2$

$A3 := Phu \cdot B$ $T := L$

$T1 := \begin{cases} \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} & \text{if } \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases}$ $T1 = -1.94\,m$

<...Uncracked section depth

$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$ $T1 = 0\,m$

<...Full base cracked

$B5 := \begin{cases} \frac{2 \cdot \frac{\Sigma W - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases}$ $B5 = 6488473879.93 \cdot kPa$

<...Stress at downstream face

$Cr := T - T1$ $Cr = 2.74 \cdot m$

<...Cracked section depth

$T1 := T - Cr = 0\,m$ $A_c := \min\left(B \cdot T1, A_c\right) = 0\,m^2$

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] + U_d = 109.4 \cdot kN$

$F_{ver} := Wc + Fa + V1 + V2 - U = 324.42 \cdot kN$

$F_{ver} := F_{ver} + N_{ten} = 324.42 \cdot kN$

$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2} \right) + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2} \right] \cdot \frac{2}{3} \cdot (T1) \right] + U_d \cdot \frac{L}{2} = 150 \cdot kN \cdot m$

$Mo_T := Mo_1 + Mo_3 = 782 \cdot kN \cdot m$

$Mr := Ms - Mo_T$

$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$

$fmax := B5$

$x_o := \begin{cases} \frac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \frac{T1}{3} & \text{otherwise} \end{cases}$

$x_o = 0\ m$

$EFF_BASE := \frac{T1}{L}$

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 0.84$

$SF_{ot} := \frac{Ms}{Mo_T} = 0.73$

<...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 1a

Initial point of inflection

$PI = 4 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 146.06 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } E \leq \frac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

CALCULATED CRACK LENGTH

$Cr = 2.74 \cdot m$

% OF CRACKED BASE

$\frac{Cr}{T} = 100 \cdot \%$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 3.65 \times 10^{-6} \cdot \%$

$q_{1min} = -404.2 \cdot kPa$

When c is taken into account

$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \geq \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 6.49 \times 10^{9} \cdot kPa$

$x_o = 0\ m$ <...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \le 0.3 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "not good"

SAFETY FACTOR AGAINST SLIDING

$FSS = 0.84$

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam

When c is taken into account

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam s

Summary of Forces and Moments

1) Vertical Forces

$Wc = 376.38 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 57.44 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U = 109.4 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 324.42 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 81.25 \cdot kN$	<... upstream hydrostatic force
$Fhd = 0 \cdot kN$	<... downstream hydrostatic force
$Fi = 0 \cdot kN$	<... Ice Force
$USHF = 0 \cdot kN$	<...horizontal force **due to hydrostatic pressure from stoplogs**
$USHF1 = 0 \cdot kN$	<...horizontal force **due to ice pressure from stoplogs**
$DSHF = -199.83 \cdot kN$	<...horizontal force acting on the Upstream Direction
$F_{hor} = 281.08 \cdot kN$	<... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 572.1 \cdot kN \cdot m$	<... stabilizing moment
$Mo_T = 781.54 \cdot kN \cdot m$	<... overturning moment
$Mr = -209.44 \cdot kN \cdot m$	<... resultant moment

DESIGN CALCULATIONS COVER SHEET

Project No. :	12-0006-028	**Project Name :**	Lock # 39		
File No. :		**Discipline :**	Civil Engineering		
Calculation Title :	**Lock# 39- Chamber Monolith Stability**				
Calculation No. :	CIV-005	**Prepared by :**	Y.F.	**Date :**	Feb 21, 2014
No. of Sheets :	33	**Checked by :**	H.H	**Date :**	March. 4, 2014
Supersedes Calc. No. :		**Approved by :**	H. H.	**Date :**	March 4, 2014

Calculation Description :

The wall has been reviewed against 2007 CDA Dam Safety Guidelines

Related Design Concept :

Stability analysis for the structures is carried out by using "Gravity Method"
Five loading cases are utilized in the analyses based 2007 CDA Guidelines

Reference Codes and Standards :

1.Design of Small Dams, Third Edition, U.S. Government Printing Office, Washington, D.C. 1999.
2. 2007 CDA Dam Safety Guidelines
3. 2009 Parks Canada Directive for Dam Safety Program of Dams and Water-Retaining Structures

ENGINEER'S SEAL

Rev. #	Rev. Description	Rev. Author	Date Revised	Checked by	Approved by	Approved Date

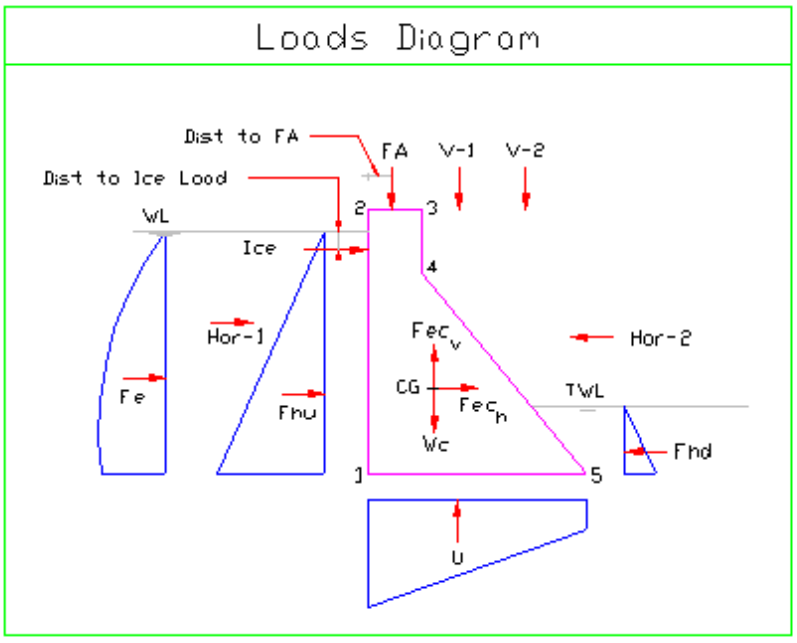
PROGRAM DESCRIPTION SECTION

This Mathcad file can calculate factors of safety against sliding and perform cracked section analysis for any shaped structure. Loading combinations conforms to CDA Dam Safety Guidelines 2007
Features

- metric or imperial units
- ice, anchor, hydrostatic and gravity loads
- crack analysis
- calculates percentage of effective base
- bearing pressures
- volume and position of center of gravity of structure (including voids)
- summary of forces and moments (including resultant and eccentricity)
- additional point loads (e.g.. backfill, vertical hydrostatic, etc.)
- hydrodynamic forces by formulas of Small Dam Design
- equivalent horizontal and vertical inertia forces on structure during earthquake

Assumptions and Limitations

- 2-dimensional
- horizontal base only
- vertical hydrostatic and backfill loads must be input as point loads
- lowest point on structure is level of base
- all vertical point loads are referenced to most upstream point of structure
- Evaluation of cracking during analyses are for unusual and extreme loading cases only.



INPUT SECTION

Unit System

1 - metric
2 - imperial

Select unit system

units := 1 <... select unit system

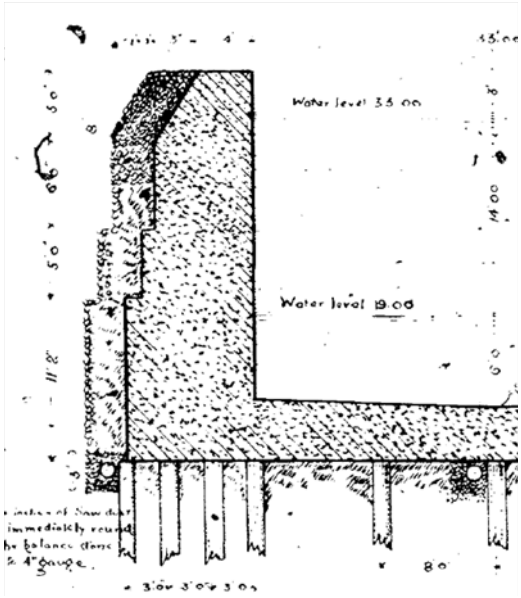
unL := if(units = 1, 1 · m, 1 · ft)

unM := if(units = 1, 1 · kN, 1 · lb)

Geometry of Structure defined by perimeter coordinates

There is no need to "close" the perimeter ,i.e., so that last point equal to first point

$$X := \begin{pmatrix} 0 \\ 0 \\ 0.3 \\ 0.3 \\ 0.61 \\ 0.61 \\ 1.52 \\ 2.74 \\ 2.74 \\ 0 \end{pmatrix} \cdot unL \qquad Y := \begin{pmatrix} 224.23 \\ 226.37 \\ 226.37 \\ 227.89 \\ 227.89 \\ 229.72 \\ 231.24 \\ 231.24 \\ 224.23 \\ 224.23 \end{pmatrix} \cdot unL$$



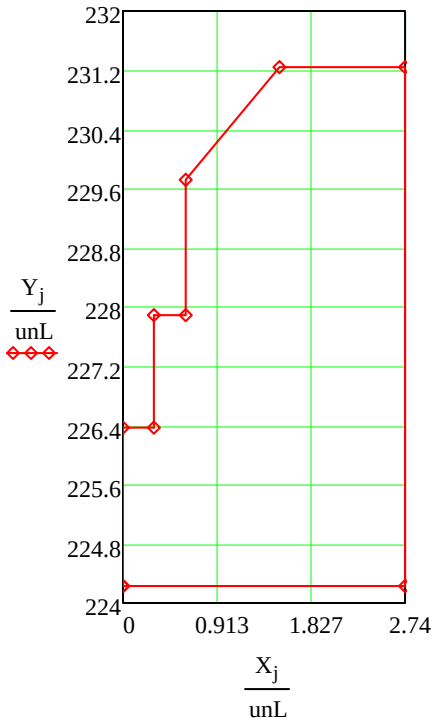
<... input X & Y coordinates

$$maxY := \frac{max(Y)}{unL} + 5 \qquad minY := \frac{min(Y)}{unL} - 5 \qquad i := 1 .. length(X)$$

$$maxX := \frac{max(X)}{unL} + 5 \qquad minX := \frac{min(X)}{unL} - 5 \qquad j := 1 .. length(X) + 1$$

$$X_{length(X)+1} := X_1 \qquad Y_{length(Y)+1} := Y_1$$

Graphical Representation of Structure defined by X & Y Coordinates



Densities of Water and Concrete, Width of Structure, of Structure defined by perimeter coordinates

$\gamma w := \text{if}\left(\text{units} = 1, 9.81 \cdot \frac{kN}{m^3}, 62.5 \cdot \frac{lb}{ft^3}\right)$ <... water density

$\gamma := \text{if}\left(\text{units} = 1, 18 \cdot \frac{kN}{m^3}, 115 \cdot \frac{lb}{ft^3}\right)$ <... backfill

$\gamma_s := \text{if}\left(\text{units} = 1, 9.1 \cdot \frac{kN}{m^3}, 49 \cdot \frac{lb}{ft^3}\right)$ <... Saturated Backfill density

$\gamma c := \text{if}\left(\text{units} = 1, 23.5 \cdot \frac{kN}{m^3}, 145 \cdot \frac{lb}{ft^3}\right)$ <... concrete density

$f'_c := \text{if}(\text{units} = 1, 17 \cdot MPa, 2030 \cdot psi)$ <... concrete compressive strength

$f'_{rock} := \text{if}(\text{units} = 1, 30 \cdot MPa, 4350 \cdot psi)$ <... mass rock compressive strength

$Structure_Width := 1.0$ <... width of structure

$\phi := 36 \cdot deg$ <... angle of internal friction for foundation material

$c := 0 \cdot f'_c$ $\quad c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $\quad f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

Computation of Area and Coordinates of Center of Gravity of Structure

$deltax_i := X_{i+1} - X_i$ $\quad deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $\quad yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot \left(deltay_i \cdot xplusx_i\right)$

$$Yginc_i := \frac{deltax_i}{8} \cdot \left[\left(yplusy_i\right)^2 + \frac{\left(deltay_i\right)^2}{3} \right] \qquad Xginc_i := \frac{deltay_i}{8} \cdot \left[\left(xplusx_i\right)^2 + \frac{\left(deltax_i\right)^2}{3} \right]$$

$$Area := \sum_i Areainc_i \qquad Area := if\left[\frac{Area}{unL^2} > 0, Area, Area \cdot (-1)\right]$$

<... Area of the Structure (defined by X & Y)

$$Xg := \sum_i Xginc_i \qquad Xg := \frac{if\left[\frac{Xg}{unL^3} > 0, Xg, Xg \cdot (-1)\right]}{Area} = 1.56\,m$$

<... X coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$$Yg := \sum_i Yginc_i \qquad Yg := \frac{if\left[\frac{Yg}{unL^3} > 0, Yg, Yg \cdot (-1)\right]}{Area} = 227.39\,m$$

<... Y coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$B := Structure_Width \cdot (unL) = 1\,m$ $\quad L := \max(X) - \min(X) = 2.74\,m$ $\quad H := \max(Y) - \min(Y) = 7.01\,m$

Computation of Concrete Volume and Weight in Structure

$Vol_conc := Area \cdot B$ $\quad Vol_conc = 16.02 \cdot m^3$

<... volume of concrete in structure

$Wc := Area \cdot B \cdot \gamma c$ $\quad Wc = 376.4 \cdot kN$

<... weight of concrete in structure

Load Case 1. Backfill in wet condition (Chamber filled with upper pool)

US_Water_Level := 228.3 <... Pizo head of backfill

Tailwater_Level := 230.34 <... Water level in chamber

SF := 2.0 <... Minimum Safety Factor

$c := 0 \cdot f'_c$ $c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

US_L_ELE := 224.23 <...Lowest Dam Elevation at upstream

DS_L_ELE := 224.23 <...Lowest Dam Elevation at downstream

$$\alpha := \operatorname{atan}\left(\frac{DS_L_ELE - US_L_ELE}{\frac{L}{unL}}\right) = 0 \cdot deg$$ <...Angle of base

$$L := \frac{L}{\cos(\alpha)} = 2.74\,m$$

Weight of soil carried by concrete wall

$W_{soil} := \gamma \cdot (H \cdot L \cdot 1.0m - Vol_conc) = 57.44 \cdot kN$ <... Weight of soil

$E_{soil} := 231.24 \cdot m - \frac{1}{3} \cdot (231.24 \cdot m - 226.37 \cdot m) = 229.62\,m$ <... Elevation of soil gravity center

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0.3 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0 <...Hydrostatic force on Gate / Stoplogs

EL_US_Hor_F := 0 <...Elevation of the Hydrostatic force on Gate / Stoplogs

US_Hor_Force1 := 0 <... Ice Load on gate / stoplogs

EL_US_Hor_F1 := 0 <...Elevation of the ice load

$i := 0 \cdot deg$ <... backfill slope

$\phi' := 28 \cdot deg$ Assumed <... angle of internal friction for backfill

$K_a := \frac{1 - \sin(\phi')}{1 + \sin(\phi')} = 0.36$ <... Active pressure

SILT_ELE := 231.24 <... top elevation of backfill

BOT_ELE := 224.23 <...bottom elevation of backfill

$H_{soil} := (SILT_ELE - BOT_ELE) \cdot unL = 7.01\,m$

$h_{sur} := 0.25 \cdot m$ consider 0.25 m surcharge on top of backfill

$q := \gamma \cdot h_{sur} = 4.5 \cdot kPa$

$H_{sur} := q \cdot H_{soil} \cdot K_a \cdot B = 11.39 \cdot kN$ Lateral force due to surcharge

$y_{sur} := \frac{H_{soil}}{2} = 3.51\,m$ Acting at

Lateral force due to backfill above water

$h'' := SILT_ELE \cdot unL - US_Water_Level \cdot unL = 2.94\,m$ backfill height above water

$h''' := (US_Water_Level - BOT_ELE) \cdot unL = 4.07\,m$ backfill submerged in water

$H_{b1} := \frac{1}{2} \cdot \gamma \cdot K_a \cdot (h'')^2 \cdot B = 28.09 \cdot kN$

$y_{b1} := h''' + \frac{h''}{3} = 5.05\,m$

Lateral force due to backfill submerged in water

$$H_{b2} := \frac{1}{2} \cdot \left[\gamma \cdot h'' + \left(\gamma \cdot h'' + \gamma_s \cdot h'''\right)\right] \cdot h''' \cdot K_a \cdot B = 104.97 \cdot kN$$

$$y_{b2} := h''' - \frac{h''' \cdot \left[2\left(\gamma \cdot h'' + \gamma_s \cdot h'''\right) + \gamma \cdot h''\right]}{3 \cdot \left(\gamma \cdot h'' + \gamma_s \cdot h''' + \gamma \cdot h''\right)} = 1.86\,m$$

Lateral force due to all backfill and surcharge

$$Hb := \left(H_{b1} + H_{b2}\right) + H_{sur} = 144.45 \cdot kN$$

$$y_a := \frac{H_{sur} \cdot y_{sur} + H_{b1} \cdot y_{b1} + H_{b2} \cdot y_{b2}}{H_{sur} + H_{b1} + H_{b2}} = 2.61\,m$$

$$E_{BF} := BOT_ELE + \frac{y_a}{unL} = 226.84$$

$$DS_Hor_Force := \frac{-Hb}{unM} = -144.45$$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$$EL_DS_Hor_F := E_{BF}$$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$$Vert_Force_1 := \frac{W_{soil}}{unM}$$

<... Weight of soil

$$Dist_to_F1 := 0.51$$

<... distance from most upstream point of structure to point of application of vertical force

$$Vert_Force_2 := 0$$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$$Dist_to_F2 := 0$$

<... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$$WL := US_Water_Level \cdot (unL) \qquad F_EL := US_L_ELE \cdot (unL) \qquad F_EL_D := DS_L_ELE \cdot (unL)$$

$$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL) \qquad F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$$

$$F_ANC := Anchor_Force \cdot (unM) \qquad Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$ $EL_USF := EL_US_Hor_F \cdot (unL)$

$USHF1 := US_Hor_Force1 \cdot (unM)$ $EL_USF1 := EL_US_Hor_F1 \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$ $EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 81.25 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 183.11 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Net uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2} = -27.42 \cdot kN$ <... Net uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B = 164.23 \cdot kN$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 136.82 \cdot kN$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF + USHF1 - DSHF)$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U = 297.01 \cdot kN$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2 = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 945.04 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$

<... Stabilizing Moment

$$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL - F_EL_D)\right] + Fi \cdot (EL_ICE - F_EL_D) \ldots + USHF \cdot (EL_USF - F_EL_D) + USHF1 \cdot (EL_USF1 - F_EL_D) \ldots + [-DSHF \cdot (EL_DSF - F_EL_D)] = 487.14 \cdot kN \cdot m$$

<... Overturning Moment (uplift not included)

$$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2} = 174.92 \cdot kN \cdot m$$

<... Overturning Moment due to Uplift

$$Mo_T := Mo_1 + Mo_2$$

<... Total Overturning Moment

$$Mr := Ms - Mo_T$$

<... Resultant Moment

$$x_o := \frac{Mr}{F_{ver}} = 0.95\,m$$

<...Resultant location. Distance from the toe.

$$E := \frac{L}{2} - x_o$$

<...Eccentricity of Resultant

$$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 207.43 \cdot kPa \qquad q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = 9.36 \cdot kPa$$

$$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$$

Calculation_to_be = "continued"

(To ensure uncracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right] & \text{otherwise} \end{cases}$$

$L_t = 0\,m$

<...Length of base under tention

$$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$$

$PI = 0\,m$

$$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases} = 2.74\,m$$

<...Length of base under compression

$$A_t := L_t \cdot B \qquad A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases} \qquad A_c = 2.74\,m^2$$

$L_1 := L_c + L_t = 2.74\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot \left| q_{1min} \right| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t \right) \right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 297.01 \cdot kN$ $Mr := Ms - Mo_T = 282.98 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 0.95\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 0.42\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L} \right) = 207.43 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L} \right) = 9.36 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 207.43 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 9.36 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 0\,m$ <...Point of inflection of stresses at foundation from heel

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha) \right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 5.07$ <...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 1

Initial point of inflection

$PI = 0 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 0 \cdot \%$

$$Result := \begin{cases} \text{"ok"} & \text{if } E \le \frac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "ok"

$q_{1min} = 9.4 \cdot kPa$

When c is taken into account

$$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \ge \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "ok"

STRESSES AT FOUNDATION

$fmin = 9.4 \cdot kPa$ $fmax = 207.43 \cdot kPa$

$x_o = 0.95\,m$

<...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \le 0.3 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

$FSS = 5.07$

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "OK"

When c is taken into account

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "OK"

Summary of Forces and Moments

1) Vertical Forces

$Wc = 376.38 \cdot kN$

<... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 57.44 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U = 136.82 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 297.01 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 81.25 \cdot kN$ <... upstream hydrostatic force

$Fhd = 183.11 \cdot kN$ <... downstream hydrostatic force

$Fi = 0 \cdot kN$ <... Ice Force

$USHF = 0 \cdot kN$ <...horizontal force **due to hydrostatic pressure from stoplogs**

$USHF1 = 0 \cdot kN$ <...horizontal force **due to ice pressure from stoplogs**

$DSHF = -144.45 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 42.58 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 945.04 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 662.06 \cdot kN \cdot m$ <... overturning moment

$Mr = 282.98 \cdot kN \cdot m$ <... resultant moment

Load Case 4. Extreme Loading Earthquake (Chamber filled with upper pool)

Horizontal & Vertical Components of the earthquake Intensity and Coefficient of Hydrodynamic Pressure Distribution

$\lambda_{hor} := 0.05$ <... **horizontal** component of earthquake intensity = ratio of earthquake acceleration to acceleration due to gravity

$\lambda_{vert} := \frac{2}{3} \cdot \lambda_{hor}$ <... **vertical** component of earthquake intensity

<... from owner's requirement 21.2.5.1

$C_m := 0.74$ <... from fig 8-6, page 325 of "Design of Small Dams", for a structure with

vertical face

SF := 1.3 <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 0 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **acceleration,** i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0 <...Hydrostatic on stoplogs

EL_US_Hor_F := 0 <...Elevation at which the Upstream Horizontal Force is applied to the Structure

$\zeta := \operatorname{atan}(\lambda_{hor}) = 2.86 \cdot deg$

$K_a := \frac{1 - \sin(\phi')}{1 + \sin(\phi')} = 0.36$ <...Active Pressure

K_{ae} = Coefficient for active earth pressure and seismic thrust

$$K_{ae} := \frac{\left[\left(\cos(i-\zeta) - \sqrt{\cos(i+\zeta)^2 - \cos(\phi')^2}\right)^2 + (\sin(i+\zeta) - \sin(i-\zeta))^2\right]^{\frac{1}{2}}}{\cos(\zeta)\cdot\left(\cos(i+\zeta) + \sqrt{\cos(i+\zeta)^2 - \cos(\phi')^2}\right)}$$

$K_{ae} = 0.370$ Using Mononobe-Rankine K_{ae}

$DS_Hor_Force := \frac{-Hb}{unM} \cdot \left(\frac{K_{ae}}{K_a}\right) = -147.94$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{BF}$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **acceleration,** i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{soil}}{unM}$

<... Weight of soil

$Dist_to_F1 := 0.51$

<... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := 0$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := 0$

<... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$

$F_EL := US_L_ELE \cdot (unL)$

$F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$

$F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$

$EL_USF := EL_US_Hor_F \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$

$EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$

$TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces and Uplift

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 81.25 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 183.11 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 164.23 \cdot kN$ <... Total Uplift force

Computation of Hydrodynamic Force Due to Earthquake

$h := US_Water_Level \cdot (unL) - F_EL$ $h = 4.07\,m$ <...total depth of the reservoir at section being analyses

$y := h$ <....vertical distance from reservoir surface to the elevation in question

$C := \frac{C_m}{2} \cdot \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right) + \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right)\right]^{0.5}\right]$ <...Coefficient of Hydrodynamic Pressure Distribution

$Pe := C \cdot \lambda_{hor} \cdot \gamma w \cdot h \cdot Structure_Width \cdot (unL)$ $Pe = 1.48 \cdot \frac{kN}{m}$ <...hydrodynamic pressure

$Fe1 := 0.726 \cdot Pe \cdot y \cdot \left(\frac{0}{100}\right)$ $Fe1 = 0 \cdot kN$ <...hydrodynamic force at pier

void ratio 0% assumed

$Me := 0.299 \cdot Pe \cdot y^2 \cdot \left(\frac{0}{100}\right)$ $Me = 0 \cdot kN \cdot m$ <...moment at the base of the structure due to hydrodynamic force

$P_{appl} := \begin{cases} F_EL + \frac{Me}{Fe1} & \text{if } Fe1 > 0kN \\ 0 \cdot m & \text{otherwise} \end{cases}$ $P_{appl} = 0 \cdot m$ <...elevation of point of application of hydrodynamic force

$Sp_{gate} := 0$ $W_{gate} := 0$

$Fe2 := 0.726 \cdot Pe \cdot y \cdot \frac{Sp_{gate}}{(Structure_Width)}$ $Fe2 = 0 \cdot kN$ <...hydrodynamic force at gate/ stoplogs

$Fe := Fe1 + Fe2$ $Fe = 0 \cdot kN$ <...Total hydrodynamic force

$Fec1_h := Wc \cdot \lambda_{hor}$ $Fec1_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec_v := Wc \cdot \lambda_{vert}$ $Fec_v = 12.55 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec2_h := W_{gate} \cdot \lambda_{hor}$ $Fec2_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Gate/stologs due to earthquake

$Fec_h := Fec1_h + Fec2_h$ $Fec_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on pier and gate due to earthquake

$Fec3_h := W_{soil} \cdot \lambda_{hor}$ $Fec3_h = 2.87 \cdot kN$ <... Equivalent Horizontal Inertia Force on Soil due to earthquake

$Fec3_v := W_{soil} \cdot \lambda_{vert}$ $Fec3_v = 1.91 \cdot kN$ <... Equivalent Vertical Inertia Force on Soil due to earthquake

$F_ANC := Anchor_Force \cdot (unM)$ $Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$F_{hor} := (Fhu - Fhd) + Fe + Fec_h + Fec3_h + (USHF - DSHF) = 67.77 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - (U + Fec_v + Fec3_v) = 255.13 \cdot kN$ <... Resultant of Vertical Forces

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 945.04 \cdot kN \cdot m$ <... Stabilizing Moment
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL - F_EL_D)\right] + Fe \cdot (P_{appl} - F_EL_D) \ldots$
$+ Fec_h \cdot (Yg - F_EL_D) + Fec_v \cdot (L - Xg) + USHF \cdot (EL_USF - F_EL_D) \ldots$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$
$Mo_1 = 570.51 \cdot kN \cdot m$

<... Overturning Moment (uplift and EQ due to slab/tower inertial force not included)

$Mo_{11} := Fec3_h \cdot (E_{soil} - F_EL_D) + Fec3_v \cdot (L - Dist_to_F1 \cdot unL)$

<... Overturning Moment due to slab and tower inertial force

$Mo_{11} = 19.74 \cdot kN \cdot m$

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ $Mo_2 = 225 \cdot kN \cdot m$ <... Overturning Moment due to Uplift

$Mo_T := (Mo_1 + Mo_{11}) + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ $x_o = 0.51\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ $E = 0.86\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 268.73 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -82.5 \cdot kPa$

(To consider cohesion and tensile strength)

$$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases}$$ $L_t = 0\,m$ <...Length of base under tention

$$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$$ $PI = 1.21\,m$

$$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases}$$ $= 1.53\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 1.53\,m^2$

$L_1 := L_c + L_t = 1.53\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 255.13 \cdot kN$ $Mr := Ms - Mo_T = 129.79 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = 0.51\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 0.86\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 268.73 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -82.5 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = 334.35 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 1.21\,m$ <...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$ <...Stress at Foundation

$SF_{ot} := \frac{Ms}{Mo_T} = 1.16$

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 2.74$ <...Factor of Safety Against Sliding.

RESULTS for Loading case 4

Initial point of inflection

$PI = 1.21 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 44.3 \cdot \%$

$EFF_BASE := 1 - \left(\frac{PI}{L}\right)$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 56 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \cdot \% \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$ <...Resultant required to be is within base.

Result = "ok"

STRESSES AT FOUNDATION

fmin = 0 · kPa

fmax = 334.3 · kPa

$f := \max(fmin, fmax)$

$x_o = 0.51\,m$

$Normal_stresses := \begin{cases} "OK" & \text{if } f \le 0.9 \cdot \min(f'_c, f'_{rock}) \\ "not\ good" & otherwise \end{cases}$

<... Allowable compressive strength =0.9 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

Not Required. The Earthquake load case is used to establish post-earthquake condition of the dam

Summary of Forces and Moments

1) Vertical Forces

Wc = 376.38 · kN <... weight of concrete in structure

Fa = 0 · kN <... anchor Force

V1 = 57.44 · kN <... Vertical Force 1 acting on the Downward Direction.

V2 = 0 · kN <... Vertical Force 2 acting on the Downward Direction.

$Fec_v = 12.55 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec3_v = 1.91 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

U = 164.23 · kN <... uplift force acting upwards

$F_{ver_U} = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 255.13 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

Fhu = 81.25 · kN <... upstream hydrostatic force

USHF = 0 · kN <...hydrostatic force on gate/stoplogs

Fhd = 183.11 · kN <... downstream hydrostatic force

Fe = 0 · kN <...hydrodynamic force on pier + gate/stoplogs

$Fec_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec3_h = 2.87 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$DSHF = -147.94 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 67.77 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 945.04 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 815.25 \cdot kN \cdot m$ <... overturning moment

$Mr = 129.79 \cdot kN \cdot m$ <... resultant moment

Load Case 1.1 Backfill in wet condition (Chamber filled with lower pool)

US_Water_Level := 228.3 <... Pizo head of backfill

Tailwater_Level := 226.04 <... Water level in chamber

SF := 2.0 <... Minimum Safety Factor

$c := 0 \cdot f'_c$ $c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0.3 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0 <...Hydrostatic force on Gate / Stoplogs

EL_US_Hor_F := 0 <...Elevation of the Hydrostatic force on Gate / Stoplogs

US_Hor_Force1 := 0 <... Ice Load on gate / stoplogs

EL_US_Hor_F1 := 0 <...Elevation of the ice load

$DS_Hor_Force := \frac{-Hb}{unM} = -144.45$

<... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

$EL_DS_Hor_F := E_{BF}$

<...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{soil}}{unM}$

<... Weight of soil

$Dist_to_F1 := 0.51$

<... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := 0$

<... Vertical Force acting on the Downward Direction. It is a stabilizing force

$Dist_to_F2 := 0$

<... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$

$F_EL := US_L_ELE \cdot (unL)$

$F_EL_D := DS_L_ELE \cdot (unL)$

$EL_ICE := WL - Dist_WL_to_Ice_Load \cdot (unL)$

$F_ICE := Ice_Load \cdot \left(\frac{unM}{unL}\right)$

$F_ANC := Anchor_Force \cdot (unM)$

$Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$

$Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$

$Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$USHF := US_Hor_Force \cdot (unM)$

$EL_USF := EL_US_Hor_F \cdot (unL)$

$USHF1 := US_Hor_Force1 \cdot (unM)$

$EL_USF1 := EL_US_Hor_F1 \cdot (unL)$

$DSHF := DS_Hor_Force \cdot (unM)$

$EL_DSF := EL_DS_Hor_F \cdot (unL)$

$Ueff := Uplift \cdot \%$

$TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 81.25 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 16.07 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd = 22.17 \cdot kPa$ <... Net uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2} = 30.37 \cdot kN$ <... Net uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B = 48.65 \cdot kN$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 79.03 \cdot kN$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF + USHF1 - DSHF)$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U = 354.8 \cdot kN$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2 = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 581.79 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL - F_EL_D)\right] + Fi \cdot (EL_ICE - F_EL_D) \ldots = 487.14 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) + USHF1 \cdot (EL_USF1 - F_EL_D) \ldots$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2} = 122.14 \cdot kN \cdot m$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}} = -0.08\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 539.92 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -280.94 \cdot kPa$

$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$

Calculation_to_be = "stoped"

(To ensure uncracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 2.97\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases}$ $= -0.23\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 0\,m^2$

$L_1 := L_c + L_t = -0.23\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 354.8 \cdot kN$ $Mr := Ms - Mo_T = -27.48 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = -0.08\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.45\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 539.92 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -280.94 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = -3053.35 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$

<...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 2.97\,m$

<...Point of inflection of stresses at foundation from heel

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.23$

<...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 1.1

Initial point of inflection

$PI = 2.97 \cdot m$

Initial point of inflection as % of the base

$\frac{PI}{L} = 108.48 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } E \leq \frac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

$q_{1min} = -280.9 \cdot kPa$

When c is taken into account

$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \geq \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$ $fmax = -3053.35 \cdot kPa$

$x_o = -0.08\,m$

<...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \le 0.3 \cdot \min\left(f'_c, f'_{rock}\right) \\ \text{"not good"} & \text{otherwise} \end{cases}$$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

FSS = 1.23

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam

When c is taken into account

$$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$$

Safety_against_sliding = "to increase dam s

Summary of Forces and Moments

1) Vertical Forces

Wc = 376.38 · kN <... weight of concrete in structure

Fa = 0 · kN <... anchor Force

V1 = 57.44 · kN <... Vertical Force 1 acting on the Downward Direction.

V2 = 0 · kN <... Vertical Force 2 acting on the Downward Direction.

U = 79.03 · kN <... uplift force acting upwards

F_{ver_U} = 433.82 · kN <... resultant of the vertical forces without uplift

F_{ver} = 354.8 · kN <... resultant of the vertical forces with uplift

2) Horizontal Forces

Fhu = 81.25 · kN <... upstream hydrostatic force

Fhd = 16.07 · kN <... downstream hydrostatic force

Fi = 0 · kN <... Ice Force

$USHF = 0 \cdot kN$ <...horizontal force **due to hydrostatic pressure from stoplogs**

$USHF1 = 0 \cdot kN$ <...horizontal force **due to ice pressure from stoplogs**

$DSHF = -144.45 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 209.63 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 581.79 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 609.28 \cdot kN \cdot m$ <... overturning moment

$Mr = -27.48 \cdot kN \cdot m$ <... resultant moment

Load Case 4.1 Extreme Loading Earthquake (Chamber filled with lower pool)

Horizontal & Vertical Components of the earthquake Intensity and Coefficient of Hydrodynamic Pressure Distribution

$\lambda_{hor} := 0.05$ <... **horizontal** component of earthquake intensity = ratio of earthquake acceleration to acceleration due to gravity

$\lambda_{vert} := \frac{2}{3} \cdot \lambda_{hor}$ <... **vertical** component of earthquake intensity

<... from owner's requirement 21.2.5.1

$C_m := 0.74$ <... from fig 8-6, page 325 of "Design of Small Dams", for a structure with **vertical face**

$SF := 1.3$ <... Minimum Safety Factor

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

$Uplift := 0$ <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

$Ice_Load := 0$ <... ice load acting per unit width of the structure

$Dist_WL_to_Ice_Load := 0$ <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **acceleration,** i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0 <...Hydrostatic on stoplogs

EL_US_Hor_F := 0 <...Elevation at which the Upstream Horizontal Force is applied to the Structure

$\zeta := \operatorname{atan}(\lambda_{hor}) = 2.86 \cdot deg$

$K_a := \frac{1 - \sin(\phi')}{1 + \sin(\phi')} = 0.36$ <...Active Pressure

K_{ae} = Coefficient for active earth pressure and seismic thrust

$$K_{ae} := \frac{\left[\left(\cos(i-\zeta) - \sqrt{\cos(i+\zeta)^2 - \cos(\phi')^2}\right)^2 + (\sin(i+\zeta) - \sin(i-\zeta))^2\right]^{\frac{1}{2}}}{\cos(\zeta)\cdot\left(\cos(i+\zeta) + \sqrt{\cos(i+\zeta)^2 - \cos(\phi')^2}\right)}$$

$K_{ae} = 0.370$ Using Mononobe-Rankine K_{ae}

$DS_Hor_Force := \frac{-Hb}{unM}\cdot\left(\frac{K_{ae}}{K_a}\right) = -147.94$ <... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

EL_DS_Hor_F := $E_{BF} = 226.84$ <...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **acceleration,** i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{soil}}{unM}$ <... Weight of soil

Dist_to_F1 := 0.51 <... distance from most upstream point of structure to point of application of vertical force

Vert_Force_2 := 0 <... Vertical Force acting on the Downward Direction. It is a stabilizing force

Dist_to_F2 := 0 <... distance from most upstream point of structure to point of application of vertical force

COMPUTATION SECTION

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

WL := US_Water_Level · (unL)

F_EL := US_L_ELE · (unL)

F_EL_D := DS_L_ELE · (unL)

EL_ICE := WL − Dist_WL_to_Ice_Load · (unL)

F_ICE := Ice_Load · $\left(\frac{unM}{unL}\right)$

F_ANC := Anchor_Force · (unM)

Anc_Dist := L − Dist_to_Anc_Force · (unL)

V1 := Vert_Force_1 · (unM)

Dist_V1 := L − Dist_to_F1 · (unL)

V2 := Vert_Force_2 · (unM)

Dist_V2 := L − Dist_to_F2 · (unL)

USHF := US_Hor_Force · (unM)

EL_USF := EL_US_Hor_F · (unL)

DSHF := DS_Hor_Force · (unM)

EL_DSF := EL_DS_Hor_F · (unL)

Ueff := Uplift · %

TWL := Tailwater_Level · (unL)

Computation of Intensity of Forces and Uplift

Phu := (WL − F_EL) · γw <... upstream hydrostatic pressure

Phd := (TWL − F_EL_D) · γw <... downstream hydrostatic pressure

Fhu := Phu · $\frac{WL - F_EL}{2}$ · B = 81.25 · kN <... upstream hydrostatic force

Fhd := Phd · $\frac{TWL - F_EL_D}{2}$ · B = 16.07 · kN <... downstream hydrostatic force

Fi := F_ICE · B <... Ice Force

Fa := F_ANC <... Anchor Force

Pu := Phu − Phd <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 48.65 \cdot kN$ <... Total Uplift force

Computation of Hydrodynamic Force Due to Earthquake

$h := US_Water_Level \cdot (unL) - F_EL$ $h = 4.07\,m$ <...total depth of the reservoir at section being analyses

$y := h$ <....vertical distance from reservoir surface to the elevation in question

$$C := \frac{C_m}{2} \cdot \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right) + \left[\frac{y}{h} \cdot \left(2 - \frac{y}{h}\right)\right]^{0.5}\right]$$ <...Coefficient of Hydrodynamic Pressure Distribution

$Pe := C \cdot \lambda_{hor} \cdot \gamma w \cdot h \cdot Structure_Width \cdot (unL)$ $Pe = 1.48 \cdot \frac{kN}{m}$ <...hydrodynamic pressure

$Fe1 := 0.726 \cdot Pe \cdot y \cdot \left(\frac{0}{100}\right)$ $Fe1 = 0 \cdot kN$ <...hydrodynamic force at pier

void ratio 0% assumed

$Me := 0.299 \cdot Pe \cdot y^2 \cdot \left(\frac{0}{100}\right)$ $Me = 0 \cdot kN \cdot m$ <...moment at the base of the structure due to hydrodynamic force

$$P_{appl} := \begin{cases} F_EL + \frac{Me}{Fe1} & \text{if } Fe1 > 0kN \\ 0 \cdot m & \text{otherwise} \end{cases}$$ $P_{appl} = 0 \cdot m$ <...elevation of point of application of hydrodynamic force

$Sp_{gate} := 0$ $W_{gate} := 0$

$Fe2 := 0.726 \cdot Pe \cdot y \cdot \frac{Sp_{gate}}{(Structure_Width)}$ $Fe2 = 0 \cdot kN$ <...hydrodynamic force at gate/ stoplogs

$Fe := Fe1 + Fe2$ $Fe = 0 \cdot kN$ <...Total hydrodynamic force

$Fec1_h := Wc \cdot \lambda_{hor}$ $Fec1_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec_v := Wc \cdot \lambda_{vert}$ $Fec_v = 12.55 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec2_h := W_{gate} \cdot \lambda_{hor}$ $Fec2_h = 0 \cdot kN$ <... Equivalent Horizontal Inertia Force on Gate/stologs due to earthquake

$Fec_h := Fec1_h + Fec2_h$ $Fec_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on pier and gate due to earthquake

$Fec3_h := W_{soil} \cdot \lambda_{hor}$ $Fec3_h = 2.87 \cdot kN$ <... Equivalent Horizontal Inertia Force on Soil due to earthquake

$Fec3_v := W_{soil} \cdot \lambda_{vert}$ $Fec3_v = 1.91 \cdot kN$ <... Equivalent Vertical Inertia Force on Soil due to earthquake

$F_ANC := Anchor_Force \cdot (unM)$ $Anc_Dist := L - Dist_to_Anc_Force \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

Computation of Intensity of Overturning & Stabilizing Moments, Etc

$F_{hor} := (Fhu - Fhd) + Fe + Fec_h + Fec3_h + (USHF - DSHF) = 234.81 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - (U + Fec_v + Fec3_v) = 370.71 \cdot kN$ <... Resultant of Vertical Forces

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 581.79 \cdot kN \cdot m$ <... Stabilizing Moment
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL - F_EL_D)\right] + Fe \cdot (P_{appl} - F_EL_D) \ldots$
$+ Fec_h \cdot (Yg - F_EL_D) + Fec_v \cdot (L - Xg) + USHF \cdot (EL_USF - F_EL_D) \ldots$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$

<... Overturning Moment (uplift and EQ due to slab/tower inertial force not included)

$Mo_1 = 570.51 \cdot kN \cdot m$

$Mo_{11} := Fec3_h \cdot (E_{soil} - F_EL_D) + Fec3_v \cdot (L - Dist_to_F1 \cdot unL)$ <... Overturning Moment due to slab and tower inertial force

$Mo_{11} = 19.74 \cdot kN \cdot m$

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ $Mo_2 = 67 \cdot kN \cdot m$ <... Overturning Moment due to Uplift

$Mo_T := (Mo_1 + Mo_{11}) + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}}$ $x_o = -0.2\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ $E = 1.57\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 601.21 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -330.62 \cdot kPa$

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{(|q_{1max}| + |q_{1min}|)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 3.35\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{(|q_{1max}| + |q_{1min}|)} & \text{if } (q_{1min} \geq f_t) \wedge (q_{1min} < 0) \\ (L - PI) & \text{otherwise} \end{cases} = -0.61\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 0\,m^2$

$L_1 := L_c + L_t = -0.61\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 370.71 \cdot kN$ $Mr := Ms - Mo_T = -75.11 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = -0.2\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.57\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 601.21 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -330.62 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = -1219.71 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$ <...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 3.35\,m$

<...Point of inflection of stresses at foundation from heel

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq 0 \\ \dfrac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases}$

$fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq 0 \\ 0 & \text{otherwise} \end{cases}$

<...Stress at Foundation

$SF_{ot} := \dfrac{Ms}{Mo_T} = 0.89$

$FSS := \dfrac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.15$

<...Factor of Safety Against Sliding.

RESULTS for Loading case 4.1

Initial point of inflection

$PI = 3.35 \cdot m$

Initial point of inflection as % of the base

$\dfrac{PI}{L} = 122.18 \cdot \%$

$EFF_BASE := 1 - \left(\dfrac{PI}{L}\right)$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = -22 \cdot \%$

$Result := \begin{cases} \text{"ok"} & \text{if } EFF_BASE \geq 0 \cdot \% \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$

<...Resultant required to be is within base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = -1219.7 \cdot kPa$

$f := \max(fmin, fmax)$

$x_o = -0.2\,m$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \leq 0.9 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.9 f'_c (and f'_{rock})

Normal_stresses = "OK"

SAFETY FACTOR AGAINST SLIDING

Not Required. The Earthquake load case is used to establish post-earthquake condition of the dam

Summary of Forces and Moments

1) Vertical Forces

$Wc = 376.38 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 57.44 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$Fec_v = 12.55 \cdot kN$ <... Equivalent Vertical Inertia Force on Pier due to earthquake

$Fec3_v = 1.91 \cdot kN$ <... Equivalent Vertical Inertia Force on Slab due to earthquake

$U = 48.65 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 370.71 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 81.25 \cdot kN$ <... upstream hydrostatic force

$USHF = 0 \cdot kN$ <...hydrostatic force on gate/stoplogs

$Fhd = 16.07 \cdot kN$ <... downstream hydrostatic force

$Fe = 0 \cdot kN$ <...hydrodynamic force on pier + gate/stoplogs

$Fec_h = 18.82 \cdot kN$ <... Equivalent Horizontal Inertia Force on Pier due to earthquake

$Fec3_h = 2.87 \cdot kN$ <... Equivalent Horizontal Inertia Force on Slab due to earthquake

$DSHF = -147.94 \cdot kN$ <...horizontal force acting on the Upstream Direction

$F_{hor} = 234.81 \cdot kN$ <... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 581.79 \cdot kN \cdot m$ <... stabilizing moment

$Mo_T = 656.91 \cdot kN \cdot m$ <... overturning moment

$Mr = -75.11 \cdot kN \cdot m$ <... resultant moment

Load Case 1a. Backfill in wet condition (Chamber to be dewatered)

US_Water_Level := 228.3 <... Pizo head of backfill

Tailwater_Level := 224.23 <... Water level in chamber

SF := 2.0 <... Minimum Safety Factor

$c := 0 \cdot f'_c$ $c = 0 \cdot kPa$ <... cohesion

$f_t := -0.5 \cdot c$ $f_t = 0 \cdot kPa$ <... tensile strength at interface of concrete /rock

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

If imperial units selected, **Ice Load** will be in **lb/ft** of Structure.
Otherwise, i.e., metric units, **Ice Load** will be in **kN/m**

Ice_Load := 0 <... ice load acting per unit width of the structure

Dist_WL_to_Ice_Load := 0.3 <... distance from water level to point of application of Ice load

If imperial units selected, **Anchor Force** will be in **lb.**
Otherwise, i.e., metric units, Anchor Force will be in **kN**

Anchor_Force := 0 <... anchor force acting per unit width of the structure

Dist_to_Anc_Force := 0 <... distance from most upstream point of structure to point of application of anchor force

Additional Horizontal Forces Acting on the Downstream and Upstream Directions

US_Hor_Force := 0 <...Hydrostatic force on Gate / Stoplogs

EL_US_Hor_F := 0 <...Elevation of the Hydrostatic force on Gate / Stoplogs

US_Hor_Force1 := 0 <... Ice Load on gate / stoplogs

EL_US_Hor_F1 := 0 <...Elevation of the ice load

$DS_Hor_Force := \frac{-Hb}{unM} = -144.45$ <... Horizontal Force acting on the Upstream Direction. For instance, this force can be used to input effect of soil pressure on the downstream side of Structure

EL_DS_Hor_F := E_{BF} = 226.84 <...Elevation at which the Downstream Horizontal Force is applied to the Structure

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

Vert_Force_1 := $\frac{W_{soil}}{unM}$ <... Weight of soil

Dist_to_F1 := 0.51 <... distance from most upstream point of structure to point of application of vertical force

Vert_Force_2 := 0 <... Vertical Force acting on the Downward Direction. It is a stabilizing force

Dist_to_F2 := 0 <... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

WL := US_Water_Level · (unL)

F_EL := US_L_ELE · (unL)

F_EL_D := DS_L_ELE · (unL)

EL_ICE := WL − Dist_WL_to_Ice_Load · (unL)

F_ICE := Ice_Load · $\left(\frac{unM}{unL}\right)$

F_ANC := Anchor_Force · (unM)

Anc_Dist := L − Dist_to_Anc_Force · (unL)

V1 := Vert_Force_1 · (unM)

Dist_V1 := L − Dist_to_F1 · (unL)

V2 := Vert_Force_2 · (unM)

Dist_V2 := L − Dist_to_F2 · (unL)

USHF := US_Hor_Force · (unM)

EL_USF := EL_US_Hor_F · (unL)

USHF1 := US_Hor_Force1 · (unM)

EL_USF1 := EL_US_Hor_F1 · (unL)

DSHF := DS_Hor_Force · (unM)

EL_DSF := EL_DS_Hor_F · (unL)

Ueff := Uplift · %

TWL := Tailwater_Level · (unL)

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

Phu := (WL − F_EL) · γw <... upstream hydrostatic pressure

Phd := (TWL − F_EL_D) · γw <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 81.25 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 0 \cdot kN$ <... downstream hydrostatic force

$Fi := F_ICE \cdot B$ <... Ice Force

$Fa := F_ANC$ <... Anchor Force

$Pu := Phu - Phd$ <... Net uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2}$ <... Net uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 54.7 \cdot kN$ <... Total Uplift force

$F_{hor} := (Fhu - Fhd) + Fi + (USHF + USHF1 - DSHF)$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + Fa + V1 + V2 - U$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + Fa + V1 + V2$ <... resultant of the vertical forces without uplift

$Ms := Wc \cdot (L - Xg) + Fhd \cdot \left(\frac{TWL - F_EL_D}{3}\right) + Fa \cdot Anc_Dist \ldots = 572.1 \cdot kN \cdot m$
$+ V1 \cdot Dist_V1 + V2 \cdot Dist_V2$ <... Stabilizing Moment

$Mo_1 := Fhu \cdot \left[\frac{WL - F_EL}{3} + (F_EL - F_EL_D)\right] + Fi \cdot (EL_ICE - F_EL_D) \ldots = 487.14 \cdot kN \cdot m$
$+ USHF \cdot (EL_USF - F_EL_D) + USHF1 \cdot (EL_USF1 - F_EL_D) \ldots$
$+ [-DSHF \cdot (EL_DSF - F_EL_D)]$ <... Overturning Moment (uplift not included)

$Mo_2 := U_u \cdot \left(\frac{2 \cdot L}{3}\right) + U_d \cdot \frac{L}{2}$ <... Overturning Moment due to Uplift

$Mo_T := Mo_1 + Mo_2$ <... Total Overturning Moment

$Mr := Ms - Mo_T$ <... Resultant Moment

$x_o := \frac{Mr}{F_{ver}} = -0.04\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 565.42 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -288.69 \cdot kPa$

$Calculation_to_be := \begin{cases} \text{"continued"} & \text{if } q_{1min} > \frac{f_t}{SF} \\ \text{"stoped"} & \text{otherwise} \end{cases}$

Calculation_to_be = "stoped"

(To ensure uncracked base for the Usual Loading Case)

(To consider cohesion and tensile strength)

$L_t := \begin{cases} 0 & \text{if } q_{1min} \geq 0 \vee q_{1min} < f_t \\ \left[\left[\frac{|q_{1min}|}{\left(|q_{1max}| + |q_{1min}|\right)} \cdot L\right]\right] & \text{otherwise} \end{cases}$ $L_t = 0\,m$ <...Length of base under tention

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases}$ $PI = 2.86\,m$

$L_c := \begin{cases} L - \frac{|q_{1min}| \cdot L}{\left(|q_{1max}| + |q_{1min}|\right)} & \text{if } \left(q_{1min} \geq f_t\right) \wedge \left(q_{1min} < 0\right) \\ (L - PI) & \text{otherwise} \end{cases}$ $= -0.12\,m$ <...Length of base under compression

$A_t := L_t \cdot B$ $A_c := \begin{cases} L_c \cdot B & \text{if } L_c \geq 0 \\ 0 & \text{otherwise} \end{cases}$ $A_c = 0\,m^2$

$L_1 := L_c + L_t = -0.12\,m$

$N_{ten} := \frac{1}{2} \cdot A_t \cdot |q_{1min}| = 0 \cdot kN$ <...vertical force due to tension

$M_{ten} := \left[N_{ten} \cdot \left(L - \frac{1}{3} \cdot L_t\right)\right] = 0 \cdot kN \cdot m$ <...stable moment due to tension

$Ms := Ms + M_{ten}$ $F_{ver} := F_{ver} + N_{ten} = 379.12 \cdot kN$ $Mr := Ms - Mo_T = -14.96 \cdot kN \cdot m$

$x_o := \frac{Mr}{F_{ver}} = -0.04\,m$ <...Resultant location. Distance from the toe.

$E := \frac{L}{2} - x_o = 1.41\,m$ <...Eccentricity of Resultant

$q_{1max} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 + \frac{6 \cdot E}{L}\right) = 565.42 \cdot kPa$ $q_{1min} := \frac{F_{ver}}{B \cdot L} \cdot \left(1 - \frac{6 \cdot E}{L}\right) = -288.69 \cdot kPa$

$fmax := \begin{cases} q_{1max} & \text{if } q_{1min} \geq f_t \\ \frac{2 \cdot F_{ver}}{3 \cdot x_o \cdot B} & \text{otherwise} \end{cases} = -6404.27 \cdot kPa$ $fmin := \begin{cases} q_{1min} & \text{if } q_{1min} \geq f_t \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kPa$

<...Stress at Foundation

$PI := \begin{cases} L - 3 \cdot x_o & \text{if } q_{1min} < f_t \\ 0 \cdot m & \text{otherwise} \end{cases} = 2.86\,m$

<...Point of inflection of stresses at foundation from heel

$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.22$

<...Factor of Safety Against Sliding.

Crack Analysis is executed only if Initial Point of Inflection of Stresses at Foundation is > 0

$\Sigma M := Ms - Mo_1$

$\Sigma W := Wc + Fa + V1 + V2$

$A3 := Phu \cdot B$ $T := L$

$T1 := \begin{cases} \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} & \text{if } \frac{3 \cdot \left(\Sigma M - A3 \cdot \frac{T^2}{2}\right)}{\Sigma W - A3 \cdot T} < T \wedge PI > 0 \\ T & \text{otherwise} \end{cases}$ $T1 = -0.6\,m$

<...Uncracked section depth

$T1 := \begin{cases} 0.0000001 \cdot (unL) & \text{if } T1 < 0 \\ T1 & \text{otherwise} \end{cases}$ $T1 = 0\,m$

<...Full base cracked

$B5 := \begin{cases} \frac{2 \cdot \frac{\Sigma W - A3 \cdot T}{T1} + A3}{B} & \text{if } T1 < T \\ fmax & \text{otherwise} \end{cases}$ $B5 = 6488473879.93 \cdot kPa$

<...Stress at downstream face

$Cr := T - T1$ $Cr = 2.74 \cdot m$

<...Cracked section depth

$T1 := T - Cr = 0\,m$ $A_c := \min\left(B \cdot T1, A_c\right) = 0\,m^2$

$U := (Pu \cdot Ueff) \cdot Cr \cdot B + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2}\right] + U_d = 109.4 \cdot kN$

$F_{ver} := Wc + Fa + V1 + V2 - U = 324.42 \cdot kN$

$F_{ver} := F_{ver} + N_{ten} = 324.42 \cdot kN$

$$Mo_3 := \left[(Pu \cdot Ueff) \cdot Cr \cdot B \cdot \left(L - \frac{Cr}{2} \right) + (Pu \cdot Ueff) \cdot \left[\frac{(T1) \cdot B}{2} \right] \cdot \frac{2}{3} \cdot (T1) \right] + U_d \cdot \frac{L}{2} = 150 \cdot kN \cdot m$$

$Mo_T := Mo_1 + Mo_3 = 637 \cdot kN \cdot m$

$Mr := Ms - Mo_T$

$fmin := \begin{cases} fmin & \text{if } T1 \geq T \\ 0 \cdot kPa & \text{otherwise} \end{cases}$

$fmax := B5$

$$x_o := \begin{cases} \frac{Mr}{F_{ver}} & \text{if } Cr = 0 \\ \frac{T1}{3} & \text{otherwise} \end{cases}$$

$x_o = 0\ m$

$$EFF_BASE := \frac{T1}{L}$$

$$FSS := \frac{\left(F_{ver} \cdot \cos(\alpha) + F_{hor} \cdot \sin(\alpha)\right) \cdot \tan(\phi) + c \cdot A_c}{F_{hor} \cdot \cos(\alpha) - F_{ver} \cdot \sin(\alpha)} = 1.04$$

$$SF_{ot} := \frac{Ms}{Mo_T} = 0.9$$

<...Factor of Safety Against Sliding.

RESULTS for Loading Combination Case 5

Initial point of inflection

$PI = 2.86 \cdot m$

Initial point of inflection as % of the base

$$\frac{PI}{L} = 104.32 \cdot \%$$

$$Result := \begin{cases} \text{"ok"} & \text{if } E \leq \frac{L}{6} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

CALCULATED CRACK LENGTH

$Cr = 2.74 \cdot m$

% OF CRACKED BASE

$$\frac{Cr}{T} = 100 \cdot \%$$

% OF EFFECTIVE BASE
(portion not cracked)

$EFF_BASE = 3.65 \times 10^{-6} \cdot \%$

$q_{1min} = -288.7 \cdot kPa$

When c is taken into account

$$Result := \begin{cases} \text{"ok"} & \text{if } q_{1min} \geq \frac{f_t}{SF} \\ \text{"to increase base size"} & \text{otherwise} \end{cases}$$

<...Resultant shall be within middle third of the base.

Result = "to increase base size"

STRESSES AT FOUNDATION

$fmin = 0 \cdot kPa$

$fmax = 6.49 \times 10^{9} \cdot kPa$

$x_o = 0\ m$ <...Resultant location. Distance from the toe.

$f := \max(fmin, fmax)$

$Normal_stresses := \begin{cases} \text{"OK"} & \text{if } f \le 0.3 \cdot \min(f'_c, f'_{rock}) \\ \text{"not good"} & \text{otherwise} \end{cases}$

<... Allowable compressive strength =0.3 f'_c (and f'_{rock})

Normal_stresses = "not good"

SAFETY FACTOR AGAINST SLIDING

$FSS = 1.04$

$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge 1.5 \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$

Safety_against_sliding = "to increase dam

When c is taken into account

$Safety_against_sliding := \begin{cases} \text{"OK"} & \text{if } FSS \ge SF \\ \text{"to increase dam size"} & \text{otherwise} \end{cases}$

Safety_against_sliding = "to increase dam s

Summary of Forces and Moments

1) Vertical Forces

$Wc = 376.38 \cdot kN$ <... weight of concrete in structure

$Fa = 0 \cdot kN$ <... anchor Force

$V1 = 57.44 \cdot kN$ <... Vertical Force 1 acting on the Downward Direction.

$V2 = 0 \cdot kN$ <... Vertical Force 2 acting on the Downward Direction.

$U = 109.4 \cdot kN$ <... uplift force acting upwards

$F_{ver_U} = 433.82 \cdot kN$ <... resultant of the vertical forces without uplift

$F_{ver} = 324.42 \cdot kN$ <... resultant of the vertical forces with uplift

2) Horizontal Forces

$Fhu = 81.25 \cdot kN$	<... upstream hydrostatic force
$Fhd = 0 \cdot kN$	<... downstream hydrostatic force
$Fi = 0 \cdot kN$	<... Ice Force
$USHF = 0 \cdot kN$	<...horizontal force **due to hydrostatic pressure from stoplogs**
$USHF1 = 0 \cdot kN$	<...horizontal force **due to ice pressure from stoplogs**
$DSHF = -144.45 \cdot kN$	<...horizontal force acting on the Upstream Direction
$F_{hor} = 225.7 \cdot kN$	<... resultant of the horizontal forces

3) Stabilizing, Overturning, and Resultant Moments

$Ms = 572.1 \cdot kN \cdot m$	<... stabilizing moment
$Mo_T = 637.02 \cdot kN \cdot m$	<... overturning moment
$Mr = -64.92 \cdot kN \cdot m$	<... resultant moment

DESIGN CALCULATIONS COVER SHEET

Project No. :	12-0006-028	**Project Name :**	DSR - Lock 39		
File No. :		**Discipline :**	Civil Engineering		
Calculation Title :	**Flotation of the U-shaped Monoliths**				
Calculation No. :	CIV-006	**Prepared by :**	Y.F.	**Date :**	April 18, 2013
No. of Sheets :	33	**Checked by :**	H.H	**Date :**	Oct. 4, 2013
Supersedes Calc. No. :		**Approved by :**	H. H.	**Date :**	Oct. 6, 2013

Calculation Description :

The wall has been reviewed against 2007 CDA Dam Safety Guidelines

Related Design Concept :

Stability analysis for the structures is carried out by using "Gravity Method"
Five loading cases are utilized in the analyses based 2007 CDA Guidelines

Reference Codes and Standards :

1.Design of Small Dams, Third Edition, U.S. Government Printing Office, Washington, D.C. 1999.
2. 2007 CDA Dam Safety Guidelines
3. 2009 Parks Canada Directive for Dam Safety Program of Dams and Water-Retaining Structures

ENGINEER'S SEAL

Rev. #	Rev. Description	Rev. Author	Date Revised	Checked by	Approved by	Approved Date

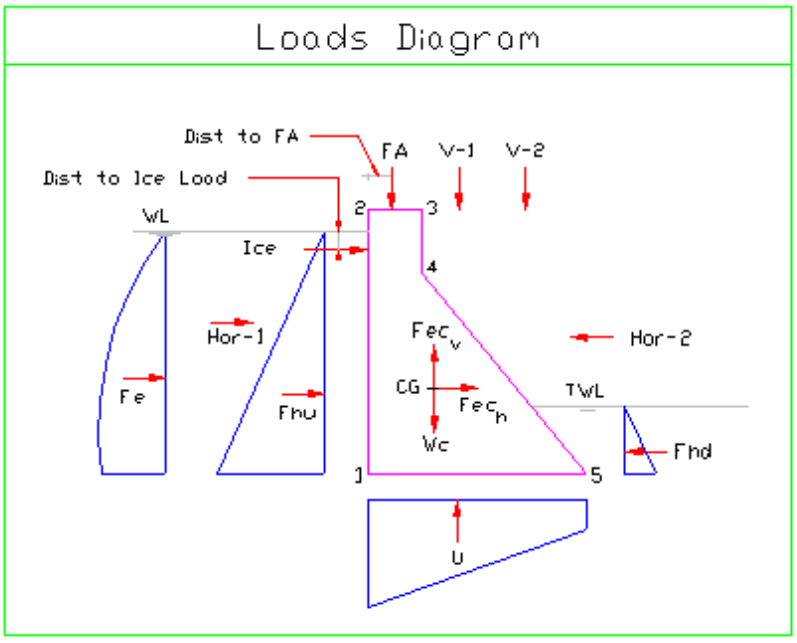
PROGRAM DESCRIPTION SECTION

This Mathcad file can calculate factors of safety against sliding and perform cracked section analysis for any shaped structure. Loading combinations conforms to CDA Dam Safety Guidelines 2007
Features

- metric or imperial units
- ice, anchor, hydrostatic and gravity loads
- crack analysis
- calculates percentage of effective base
- bearing pressures
- volume and position of center of gravity of structure (including voids)
- summary of forces and moments (including resultant and eccentricity)
- additional point loads (e.g.. backfill, vertical hydrostatic, etc.)
- hydrodynamic forces by formulas of Small Dam Design
- equivalent horizontal and vertical inertia forces on structure during earthquake

Assumptions and Limitations

- 2-dimensional
- horizontal base only
- vertical hydrostatic and backfill loads must be input as point loads
- lowest point on structure is level of base
- all vertical point loads are referenced to most upstream point of structure
- Evaluation of cracking during analyses are for unusual and extreme loading cases only.



INPUT SECTION

Unit System

1 - metric
2 - imperial

Select unit system

units := 1 <... select unit system

unL := if(units = 1, 1 · m, 1 · ft)

unM := if(units = 1, 1 · kN, 1 · lb)

Geometry of Structure defined by perimeter coordinates

There is no need to "close" the perimeter ,i.e., so that last point equal to first point

<... input X & Y coordinates

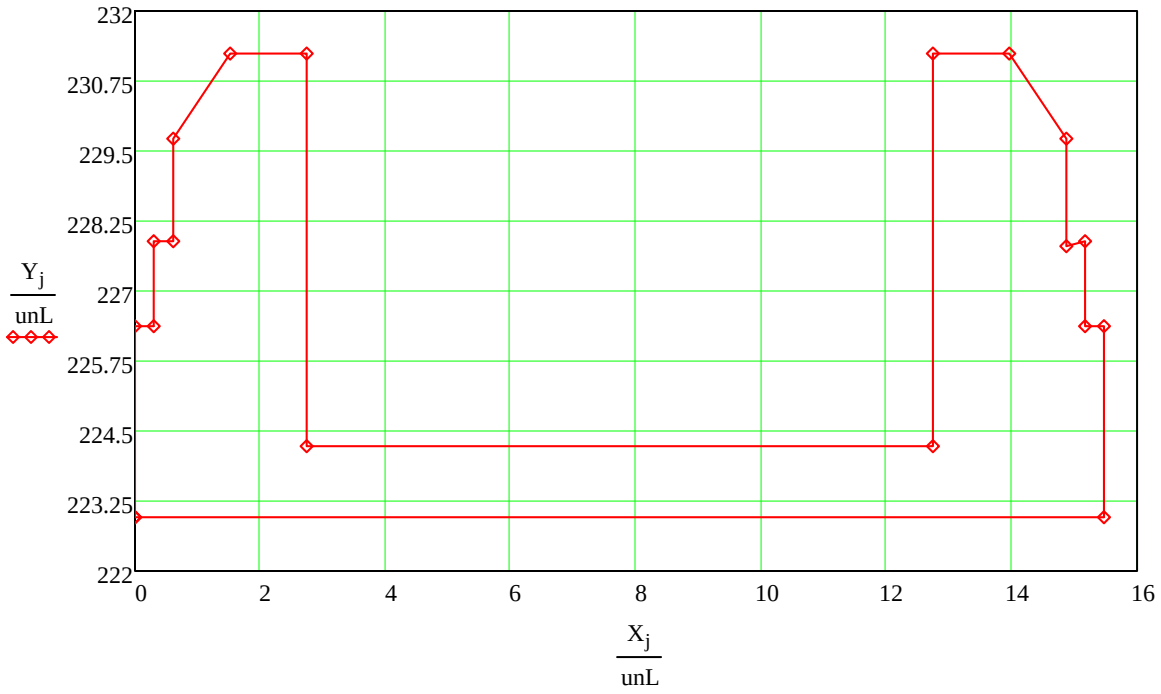
$$X := \begin{pmatrix} 0 \\ 0 \\ 0.3 \\ 0.3 \\ 0.61 \\ 0.61 \\ 1.52 \\ 2.74 \\ 2.74 \\ 12.74 \\ 12.74 \\ 13.96 \\ 14.87 \\ 14.87 \\ 15.17 \\ 15.17 \\ 15.47 \\ 15.47 \\ 0 \end{pmatrix} \cdot unL \qquad Y := \begin{pmatrix} 222.96 \\ 226.37 \\ 226.37 \\ 227.89 \\ 227.89 \\ 229.72 \\ 231.24 \\ 231.24 \\ 224.23 \\ 224.23 \\ 231.24 \\ 231.24 \\ 229.72 \\ 227.80 \\ 227.89 \\ 226.37 \\ 226.37 \\ 222.96 \\ 222.96 \end{pmatrix} \cdot unL$$

$$maxY := \frac{\max(Y)}{unL} + 5 \qquad minY := \frac{\min(Y)}{unL} - 5 \qquad i := 1 .. length(X)$$

$$maxX := \frac{\max(X)}{unL} + 5 \qquad minX := \frac{\min(X)}{unL} - 5 \qquad j := 1 .. length(X) + 1$$

$X_{length(X)+1} := X_1$ $Y_{length(Y)+1} := Y_1$

Graphical Representation of Structure defined by X & Y Coordinates



Densities of Water and Concrete, Width of Structure, of Structure defined by perimeter coordinates

$\gamma w := if\left(units = 1, 9.81 \cdot \frac{kN}{m^3}, 62.5 \cdot \frac{lb}{ft^3}\right)$ <... water density

$\gamma_{silt} := if\left(units = 1, 18 \cdot \frac{kN}{m^3}, 78 \cdot \frac{lb}{ft^3}\right)$ <... backfill

$\gamma c := if\left(units = 1, 23.5 \cdot \frac{kN}{m^3}, 145 \cdot \frac{lb}{ft^3}\right)$ <... concrete density

$Structure_Width := 1.0$ <... width of structure

Computation of Area and Coordinates of Center of Gravity of Structure

$deltax_i := X_{i+1} - X_i$ $deltay_i := Y_{i+1} - Y_i$

$xplusx_i := X_{i+1} + X_i$ $yplusy_i := Y_{i+1} + Y_i$

$Areainc_i := 0.5 \cdot (deltay_i \cdot xplusx_i)$

$Yginc_i := \frac{deltax_i}{8} \cdot \left[(yplusy_i)^2 + \frac{(deltay_i)^2}{3} \right]$ $\quad Xginc_i := \frac{deltay_i}{8} \cdot \left[(xplusx_i)^2 + \frac{(deltax_i)^2}{3} \right]$

$Area := \sum_i Areainc_i$ $\quad Area := if\left[\frac{Area}{unL^2} > 0, Area, Area \cdot (-1) \right]$ <... Area of the Structure (defined by X & Y)

$Xg := \sum_i Xginc_i$ $\quad Xg := \frac{if\left[\frac{Xg}{unL^3} > 0, Xg, Xg \cdot (-1) \right]}{Area}$ <... X coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$Yg := \sum_i Yginc_i$ $\quad Yg := \frac{if\left[\frac{Yg}{unL^3} > 0, Yg, Yg \cdot (-1) \right]}{Area}$ <... Y coordinate of Center of Gravity of the Structure (defined by X & Y) from most upstream point

$B := Structure_Width \cdot (unL)$ $\quad L := \max(X) - \min(X) = 15.47\ m$

Computation of Concrete Volume and Weight in Structure

$Vol_conc := Area \cdot B$ $\quad Vol_conc = 52 \cdot m^3$ <... volume of concrete in structure

$Wc := Area \cdot B \cdot \gamma c$ $\quad Wc = 1213.3 \cdot kN$ <... weight of concrete in structure

Load Case 1. Usual Load - Floatation for Piezo Head @ Normal Head plus Lower Pool Head

$US_Water_Level := 228.3$ <... max. headwater level

$Tailwater_Level := 228.3$ <... tailwater level

$Water_ELE_Chamber := 226.04$ <... Lower pool elevation

$Bottom_ELE_chamber := 224.23$ <... Bottom of chamber

$b_{chamber} := 10.6$ <... clear width of chamber

$US_L_ELE := 222.96$ <...Lowest Dam Elevation at upstream

$DS_L_ELE := 222.96$ <...Lowest Dam Elevation at downstream

Weight of soil carried by concrete wall

$W_{soil} := 2 \cdot 57.44 \cdot kN = 114.88 \cdot kN$ <... Weight of soil

Weight of WATER IN LOCK CHAMBER

$W_w := \gamma w \cdot \left[b_{chamber} \cdot Structure_Width \cdot (Water_ELE_Chamber - Bottom_ELE_chamber) \right] \cdot m^3 = 188.21 \cdot kN$

Uplift, Ice load & Point of Application, Anchor Force & Point of Application, & Other Vertical and Horizontal Forces

Uplift := 100 <... percent of uplift pressure acting at the upstream point of the structure foundation

Additional Vertical Forces (Stabilizing Forces)

If imperial units selected, **Vertical Forces** will be in **lb.**
Otherwise, i.e., metric units, **Vertical Forces** will be in **kN**

$Vert_Force_1 := \frac{W_{soil}}{unM}$ <... Weight of soil

Dist_to_F1 := 7.735 <... distance from most upstream point of structure to point of application of vertical force

$Vert_Force_2 := \frac{W_w}{unM}$ <... Vertical Force acting on the Downward Direction. It is a stabilizing force

Dist_to_F2 := 7.735 <... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$ $F_EL := US_L_ELE \cdot (unL)$ $F_EL_D := DS_L_ELE \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 139.87 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 139.87 \cdot kN$ <... downstream hydrostatic force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2} = 0 \cdot kN$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B = 810.4 \cdot kN$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 810.4 \cdot kN$ <... Total Uplift force

$F_{hor} := Fhu - Fhd = 0 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + V1 + V2 - U = 705.98 \cdot kN$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + V1 + V2 = 1516.39 \cdot kN$ <... resultant of the vertical forces without uplift

$SF := \frac{F_{ver_U}}{U} = 1.87$ <...Factor of Safety for floatation

Load Case 2. Extreme Load - Lock Chamber dewatered

US_Water_Level := 228.3 <... max. headwater level

Tailwater_Level := 228.3 <... tailwater level

Water_ELE_Chamber := 224.23 <... Lower pool elevation

Bottom_ELE_chamber := 224.23 <... Bottom of chamber

Weight of WATER IN LOCK CHAMBER

$W_w := \gamma w \cdot \left[b_{chamber} \cdot Structure_Width \cdot (Water_ELE_Chamber - Bottom_ELE_chamber) \right] \cdot m^3 = 0 \cdot kN$

$Vert_Force_2 := \frac{W_w}{unM}$ <... Vertical Force acting on the Downward Direction. It is a stabilizing force

Dist_to_F2 := 7.735 <... distance from most upstream point of structure to point of application of vertical force

Computation of Certain Parameters Such as the Structure Base Length, Elevation of Application of Ice Load, Etc.

$WL := US_Water_Level \cdot (unL)$ $F_EL := US_L_ELE \cdot (unL)$ $F_EL_D := DS_L_ELE \cdot (unL)$

$V1 := Vert_Force_1 \cdot (unM)$ $Dist_V1 := L - Dist_to_F1 \cdot (unL)$

$V2 := Vert_Force_2 \cdot (unM)$ $Dist_V2 := L - Dist_to_F2 \cdot (unL)$

$Ueff := Uplift \cdot \%$ $TWL := Tailwater_Level \cdot (unL)$

Computation of Intensity of Forces, Uplift, Overturning & Stabilizing Moments, Etc

$Phu := (WL - F_EL) \cdot \gamma w$ <... upstream hydrostatic pressure

$Phd := (TWL - F_EL_D) \cdot \gamma w$ <... downstream hydrostatic pressure

$Fhu := Phu \cdot \frac{WL - F_EL}{2} \cdot B = 139.87 \cdot kN$ <... upstream hydrostatic force

$Fhd := Phd \cdot \frac{TWL - F_EL_D}{2} \cdot B = 139.87 \cdot kN$ <... downstream hydrostatic force

$Pu := Phu - Phd$ <... Uplift pressure at upstream end

$U_u := \frac{(Pu \cdot Ueff) \cdot L \cdot B}{2} = 0 \cdot kN$ <... Uplift force due to upstream head water

$U_d := Phd \cdot L \cdot B = 810.4 \cdot kN$ <... Uplift force due to downstream tailwater

$U := U_u + U_d = 810.4 \cdot kN$ <... Total Uplift force

$F_{hor} := Fhu - Fhd = 0 \cdot kN$ <... Resultant of Horizontal Forces

$F_{ver} := Wc + V1 + V2 - U = 517.77 \cdot kN$ <... Resultant of Vertical Forces

$F_{ver_U} := Wc + V1 + V2 = 1328.17 \cdot kN$ <... resultant of the vertical forces without uplift

$SF := \frac{F_{ver_U}}{U} = 1.639$ <...Factor of Safety for floatation

APPENDIX K

STRUCTURAL ASSESSMENT - STOPLOGS & LOCK GATES

STOPLOGS STRUCTURAL REVIEW

DESIGN CALCULATIONS COVER SHEET

Project No. :	12-0006-028	**Project Name :**	Talbot River Dam (Dam @ Lock 39)		
File No. :		**Discipline :**	Civil Engineering		
Calculation Title :	Stoplog Design Review				
Calculation No. :	MEC -001	**Prepared by :**	Y.F	**Date :**	March 22, 2013
No. of Sheets :	7	**Checked by :**	H.H	**Date :**	April 4, 2013
Supersedes Calc. No. :		**Approved by :**	H.H	**Date :**	April 4, 2013

Calculation Description :

Design review on Wooden Stoplog

Related Design Concept :

Two loading cases are utilized in the analysis

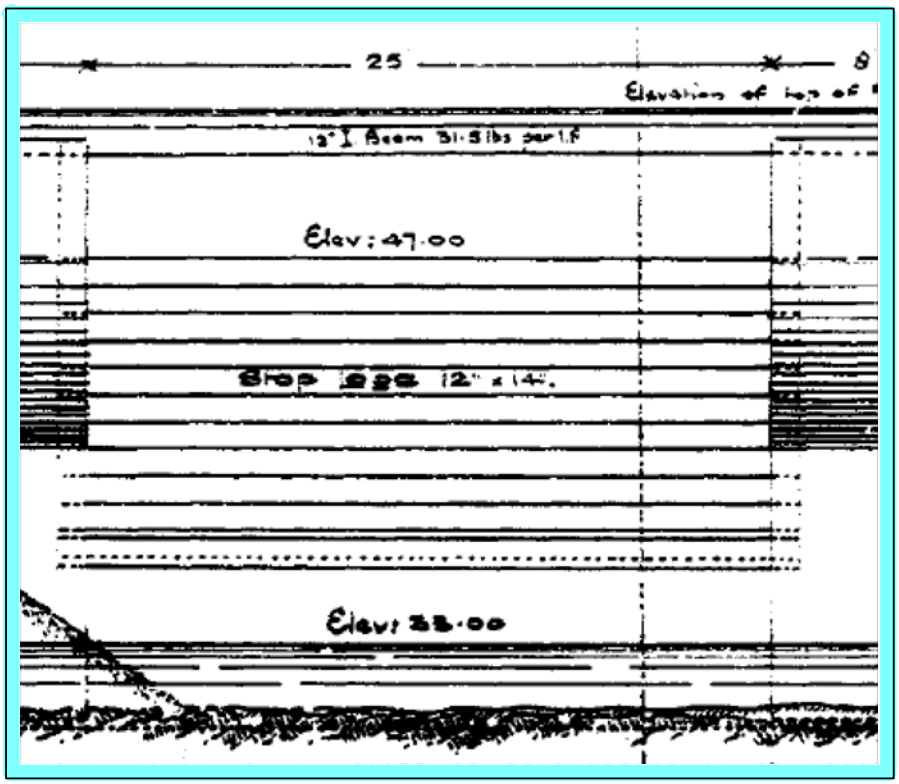
Reference Codes and Standards :

CSA Standards, O86-01, Engineering Design in wood

ENGINEER'S SEAL

Rev. #	Rev. Description	Rev. Author	Date Revised	Checked by	Approved by	Approved Date

Sketch - Sluice



Load Case 1a- Bottom Log against normal summer operating water head

Stoplog Size

$b := 0.356 \cdot m$ $d := 0.305 \cdot m$ $L := 7.92 \cdot m$ Span

$\gamma w := 9.81 \frac{kN}{m^3}$ Specific weight of water

$HWL := 230.44 \cdot m$ Head water level

$Sill_ele := 227.28 \cdot m$ sill elevation

$h := HWL - Sill_ele - \frac{d}{2} = 3.01\,m$ Head at the center of Bottom stoplog

$H := \gamma w \cdot h \cdot d = 9 \cdot \frac{kN}{m}$ Hydrostatic Load on Stoplog

$Hf := 1.5 \cdot H$ Factored Hydrostatic Load on Stoplog

$Vf1 := \frac{Hf \cdot (L)}{2}$ $Vf1 = 53.45 \cdot kN$ factored Shear

$Mf1 := \frac{Hf \cdot L^2}{8}$ $Mf1 = 105.83 \cdot kN \cdot m$ Factored Moment

Load Case 2- Top stoplog against ice load

$Ice_Load := 30 \cdot \frac{kN}{m}$ Load on stoplogs

$Ice_Load_{ff} := 1.5 \cdot Ice_Load$ Factored Ice Load

$Vf2 := \frac{Ice_Load_{ff} \cdot L}{2} = 178.2 \cdot kN$ factored shear due to ice load

$Mf2 := \frac{Ice_Load_{ff} \cdot L^2}{8} = 352.84 \cdot kN \cdot m$ factored moment due to ice load

STOPLOG LOAD CAPACITY (Assumption Timber grade to be D Fir-L, Selected Structure)

Load Case 1 - Permnanent loading case

Bending moment Resistance

$fb := 19.5 \cdot MPa$ Specified strength in bending Table 5.3.1D

$Kd := 0.65$ Load duration factor -Permanent for load case 1

$Kh := 1$ System factor

$Ksb := 1$ Service condition factor

$Kt := 1$ Traetment factor

$Fb := fb \cdot Kd \cdot Kh \cdot Ksb \cdot Kt = 12.68 \cdot MPa$

$\Phi := 0.9$

$S := \frac{b^2 \cdot d}{6} = 6442413.33 \cdot mm^3$ section modulus

$Kzb := 0.95$ Size factor in bending for 305mmx365 mm wood

$Kl := 1$ lateral stability factor

$Mr1 := \Phi \cdot Fb \cdot S \cdot Kzb \cdot Kl$ clause 5.5.4.1

$Mr1 = 69.8 \cdot kN \cdot m$

Shear Resistance

$fv := 1.2 \cdot MPa$ specified strength in shear Table 5.3.1D

$Ksv := 1$ service condition factor

$Fv := fv \cdot Kd \cdot Kh \cdot Ksv \cdot Kt = 0.78 \cdot MPa$

$An := b \cdot d$ Net area

$Kzv := 0.95$ Size factor in shear

$Vr1 := \frac{\Phi \cdot Fv \cdot 2 \cdot An \cdot Kzv}{3}$ Clause 5.5.5.1

$Vr1 = 48.3 \cdot kN$

$stoplog_structural_capacity1 := \begin{cases} \text{"ok"} & \text{if } Vr1 \geq Vf1 \wedge Mr1 > Mf1 \\ \text{"not good"} & \text{otherwise} \end{cases}$

$stoplog_structural_capacity1 = \text{"not good"}$

Load Case 2 - Standard duration loading case

$Kd := 1.0$ Load duration factor -Standard Duration

$Fb := fb \cdot Kd \cdot Kh \cdot Ksb \cdot Kt = 19.5 \cdot MPa$

$Fv := fv \cdot Kd \cdot Kh \cdot Ksv \cdot Kt = 1.2 \cdot MPa$

$Mr2 := \Phi \cdot Fb \cdot S \cdot Kzb \cdot Kl = 107.41 \cdot kN \cdot m$

$Vr2 := \frac{\Phi \cdot Fv \cdot 2 \cdot An \cdot Kzv}{3} = 74.27 \cdot kN$

$stoplog_structural_capacity2 := \begin{cases} \text{"ok"} & \text{if } Vr2 \geq Vf2 \wedge Mr2 > Mf2 \\ \text{"not good"} & \text{otherwise} \end{cases}$

$stoplog_structural_capacity2 = \text{"not good"}$

$\frac{Mf2}{Mr2} = 3.28$ $\frac{Vf2}{Vr2} = 2.4$

LOCK 39 GATE STRUCTURAL REVIEW

DESIGN CALCULATIONS COVER SHEET

Project No. :	12-0006-028	**Project Name :**	Lock 39		
File No. :		**Discipline :**	Civil Engineering		
Calculation Title :	Gate Design Review				
Calculation No. :	MEC -002	**Prepared by :**	Y.F	**Date :**	March 22, 2013
No. of Sheets :	5	**Checked by :**	H.H	**Date :**	
Supersedes Calc. No. :		**Approved by :**	H.H	**Date :**	

Calculation Description :

Design review on the wooden logs of the gate

Related Design Concept :

Two loading cases are utilized in the analysis

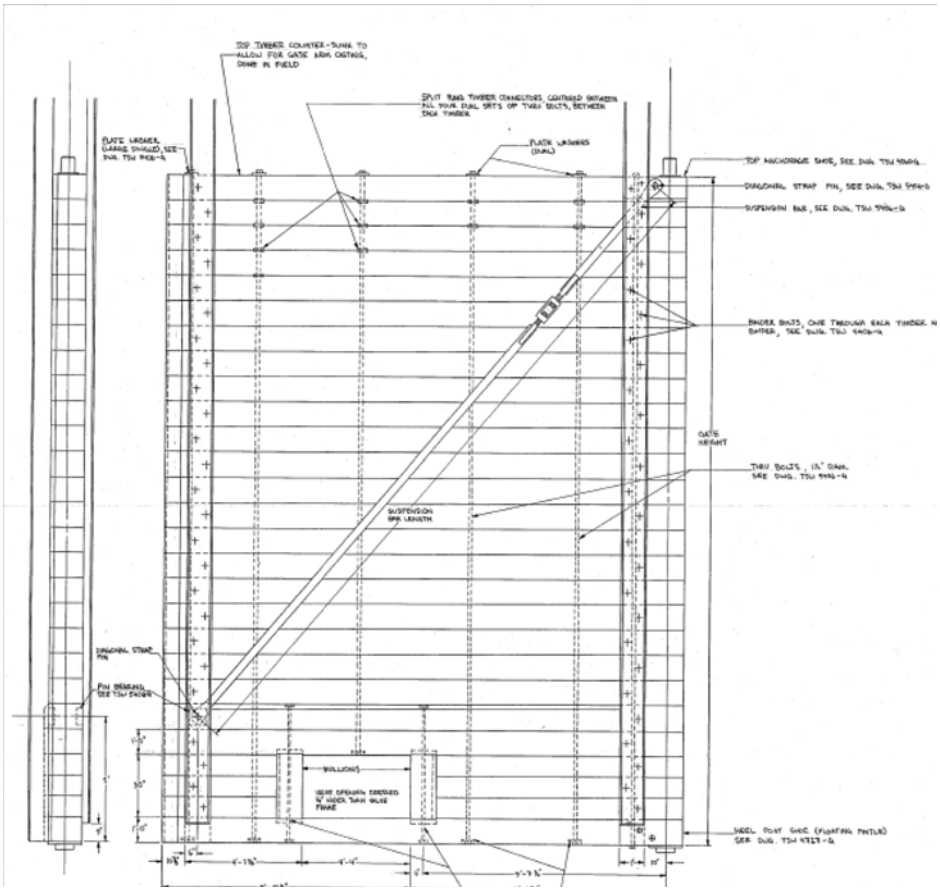
Reference Codes and Standards :

CSA Standards, O86-01, Engineering Design in wood

ENGINEER'S SEAL

Rev. #	Rev. Description	Rev. Author	Date Revised	Checked by	Approved by	Approved Date

Sketch - Gate Leaf



Gate log Size @ Bottom

$b := 0.457 \cdot m$ $d := 0.305 \cdot m$ $L := 6.0 \cdot m$ Span

Load Case 1a- Bottom Logs against normal summer operating water head

Normal operating water Level $H_{wl} := 230.5m$

Gate sill elevation $H_{sill} := 224.23 \cdot m$

$H_{max} := H_{wl} - H_{sill} = 6.27\,m$

Hydrostatic on bottom logs

@ 1st.bottom Log $P_{1h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot \left(H_{max} - \frac{d}{2}\right) = 18.3 \cdot \frac{kN}{m}$

@ 2nd bottom Log $P_{2h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot 0.25 \cdot m \cdot (H_{max} - 0.429 \cdot m) = 14.33 \cdot \frac{kN}{m}$

@ 3rd bottom Log $P_{3h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot 0.25 \cdot m \cdot (H_{max} - 0.683 \cdot m) = 13.7 \cdot \frac{kN}{m}$

@4th bottom Log $P_{4h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot 0.25 \cdot m \cdot (H_{max} - 0.937 \cdot m) = 13.08 \cdot \frac{kN}{m}$

@ 5th bottom Log $P_{5h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot (H_{max} - 1.216 \cdot m) = 15.12 \cdot \frac{kN}{m}$

@ 6th bottom Log $P_{6h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot (H_{max} - 1.521 \cdot m) = 14.21 \cdot \frac{kN}{m}$

@ 7th bottom Log $P_{7h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot (H_{max} - 1.826 \cdot m) = 13.3 \cdot \frac{kN}{m}$

Point loads from Valves $A_{valve} := 1.626 \cdot m \cdot (0.762 \cdot m) = 1.24\,m^2$

$P_h := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot (H_{max} - 0.683 \cdot m) \cdot A_{valve} = 67.91 \cdot kN$

Load Case 1b- Bottom Logs against normal summer operating water head plus tailwater

Tail water Level $H_{twl} := 226.04m$

$H_{max} := H_{twl} - H_{sill} = 1.81\,m$

Hydrostatic on bottom logs

@ 1st.bottom Log $P_{1t} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot \left(H_{max} - \frac{d}{2}\right) = 4.96 \cdot \frac{kN}{m}$

@ 2nd bottom Log $P_{2t} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot 0.25 \cdot m \cdot (H_{max} - 0.429 \cdot m) = 3.39 \cdot \frac{kN}{m}$

@ 3rd bottom Log $P_{3t} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot 0.25 \cdot m \cdot \left(H_{max} - 0.683 \cdot m\right) = 2.76 \cdot \frac{kN}{m}$

@4th bottom Log $P_{4t} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot 0.25 \cdot m \cdot \left(H_{max} - 0.937 \cdot m\right) = 2.14 \cdot \frac{kN}{m}$

@ 5th bottom Log $P_{5t} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot \left(H_{max} - 1.216 \cdot m\right) = 1.78 \cdot \frac{kN}{m}$

@ 6th bottom Log $P_{6t} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot \left(H_{max} - 1.521 \cdot m\right) = 0.86 \cdot \frac{kN}{m}$

@ 7th bottom Log $P_{7t} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot \left(H_{max} - 1.826 \cdot m\right) = -0.05 \cdot \frac{kN}{m}$

Point loads from Valves $P_t := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot \left(H_{max} - 0.683 \cdot m\right) \cdot A_{valve} = 13.7 \cdot kN$

$P_1 := P_{1h} - P_{1t} = 13.34 \cdot \frac{kN}{m}$

$P_2 := P_{2h} - P_{2t} = 10.94 \cdot \frac{kN}{m}$

$P_3 := P_{3h} - P_{3t} = 10.94 \cdot \frac{kN}{m}$

$P_4 := P_{4h} - P_{4t} = 10.94 \cdot \frac{kN}{m}$

$P_5 := P_{5h} - P_{5t} = 13.34 \cdot \frac{kN}{m}$

$P_6 := P_{6h} - P_{6t} = 13.34 \cdot \frac{kN}{m}$

$P_7 := P_{7h} - P_{7t} = 13.34 \cdot \frac{kN}{m}$

$P := P_h - P_t = 54.21 \cdot kN$

Load Case 2- Bottom Logs against normal winter operating water head (Tailwater to be dry)

Winter water Level $H_{wl} := 230.14m$

$H_{max} := H_{wl} - H_{sill} = 5.91\,m$

Hydrostatic on bottom logs

@ 1st.bottom Log $P_{1h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot \left(H_{max} - \frac{d}{2}\right) = 17.23 \cdot \frac{kN}{m}$

@ 2nd bottom Log

$$P_{2h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot 0.25 \cdot m \cdot \left(H_{max} - 0.429 \cdot m\right) = 13.44 \cdot \frac{kN}{m}$$

@ 3rd bottom Log

$$P_{3h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot 0.25 \cdot m \cdot \left(H_{max} - 0.683 \cdot m\right) = 12.82 \cdot \frac{kN}{m}$$

@4th bottom Log

$$P_{4h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot 0.25 \cdot m \cdot \left(H_{max} - 0.937 \cdot m\right) = 12.2 \cdot \frac{kN}{m}$$

@ 5th bottom Log

$$P_{5h} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot \left(H_{max} - 1.216 \cdot m\right) = 14.04 \cdot \frac{kN}{m}$$

@ 6th bottom Log

$$P_{6t} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot \left(H_{max} - 1.521 \cdot m\right) = 13.13 \cdot \frac{kN}{m}$$

@ 7th bottom Log

$$P_{7t} := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot d \cdot \left(H_{max} - 1.826 \cdot m\right) = 12.22 \cdot \frac{kN}{m}$$

Point loads from Valves

$$A_{valve} := 1.626 \cdot m \cdot (0.762 \cdot m) = 1.24\,m^2$$

$$P_h := 9.81 \cdot \left(\frac{kN}{m^3}\right) \cdot \left(H_{max} - 0.683 \cdot m\right) \cdot A_{valve} = 63.53 \cdot kN$$

Forces calculated from STAAD PRO (see STAAD Output)

Load Case 1

For Log 305mm x 457mm

$V_{f1} := 121.0 \cdot kN$ factored Shear

$M_{f1} := 209.0 \cdot kN \cdot m$ Factored Moment

$P_{f1} := 294.0 \cdot kN$ Factored compressive forces

For Log 305mm x 406mm

$V_{f1a} := 61.0 \cdot kN$ factored Shear

$M_{f1a} := 93.0 \cdot kN \cdot m$ Factored Moment

$P_{f1a} := 117.2 \cdot kN$ Factored compressive forces

Load Case 2

For Log 305mm x 457mm

$V_{f2} := 129.1 \cdot kN$ factored Shear

$M_{f2} := 231.39 \cdot kN \cdot m$ Factored Moment

$P_{f2} := 309.9 \cdot kN$ Factored compressive forces

For Log 305mm x 406mm

$V_{f2a} := 60.1 \cdot kN$ factored Shear

$M_{f2a} := 91.5 \cdot kN \cdot m$ Factored Moment

$P_{f2a} := 115.4 \cdot kN$ Factored compressive forces

Under factored Ice load

$V_{f2b} := 137.3 \cdot kN$ factored Shear

$M_{f2b} := 209.0 \cdot kN \cdot m$ Factored Moment

$P_{f2b} := 263.7 \cdot kN$ Factored compressive forces

LOG STRUCTURAL CAPACITY (Assumption Timber grade to be D Fir-L, Selected Structure)

Permanent loading case

Bending moment Resistance For Log 305mm x 457m

$b := 0.457 \cdot m$ $d := 0.305 \cdot m$

$V_f := \max(V_{f1}, V_{f2}) = 129.1 \cdot kN$

$M_f := \max(M_{f1}, M_{f2}) = 231.39\,m \cdot kN$

$P_f := \max(P_{f1}, P_{f2}) = 309.9 \cdot kN$

$fb := 19.5 \cdot MPa$ Specified strength in bending Table 5.3.1C

$Kd := 0.65$ Load duration factor -Permanent for load case 1

$Kh := 1$ System factor

$Ksb := 1$ Service condition factor

$Kt := 1$ Treatment factor

$Fb := fb \cdot Kd \cdot Kh \cdot Ksb \cdot Kt = 12.68 \cdot MPa$

$\Phi := 0.9$

$S := \frac{b^2 \cdot d}{6} = 10616490.83 \cdot mm^3$ section modulus

$Kzb := 0.90$ Size factor in bending for 305mmx457 mm wood

$Kl := 1$ lateral stability factor

$M_r := \Phi \cdot Fb \cdot S \cdot Kzb \cdot Kl$ clause 5.5.4.1

$M_r = 109 \cdot kN \cdot m$

$\frac{M_f}{M_r} = 2.12$

Shear Resistance

$fv := 1.5 \cdot MPa$ specified strength in shear Table 5.3.1D

$Ksv := 1$ service condition factor

$Fv := fv \cdot Kd \cdot Kh \cdot Ksv \cdot Kt = 0.98 \cdot MPa$

$An := b \cdot d$ Net area

$Kzv := 0.90$ Size factor in shear

$V_r := \frac{\Phi \cdot Fv \cdot 2 \cdot An \cdot Kzv}{3}$ Clause 5.5.5.1

$V_r = 73.4 \cdot kN$

$\frac{V_f}{V_r} = 1.76$

Compressive Resistance Parallel to Grain

$fc := 13.2 \cdot MPa$ specified strength in compression Table 5.3.1D

$Ksc := 0.91$ service condition factor Table 5.4.2

$Fc := fc \cdot Kd \cdot Kh \cdot Ksc \cdot Kt = 7.81 \cdot MPa$

$$Kzc := \min\left[6.3 \cdot \left(\frac{b \cdot L}{m^2}\right)^{-0.13}, 1.3\right] = 1.3$$

$K_{SE} := 1.0$ Table 5.4.2

$E_{05} := 8000 \cdot MPa$ Table 5.3.1D

$$C_c := \frac{L}{b} = 13.13$$

$$K_c := \left(1.0 + \frac{Fc \cdot Kzc \cdot C_c^{3}}{35 \cdot E_{05} \cdot K_{SE} \cdot Kt}\right)^{-1} = 0.92$$

$\Phi_c := 0.8$

$P_r := \Phi_c \cdot Fc \cdot An \cdot Kzc \cdot K_c = 1046.01 \cdot kN$ Clause 5.5.6.2.2

$$Check := \frac{M_f}{M_r} + \frac{P_f}{P_r} = 2.42$$

$$Gate_structural_capacity1 := \begin{cases} \text{"ok"} & \text{if } V_r \geq V_f \wedge \frac{M_f}{M_r} + \frac{P_f}{P_r} \leq 1.0 \\ \text{"not good"} & \text{otherwise} \end{cases}$$

Gate_structural_capacity1 = "not good"

Bending moment Resistance For Log 305mm x 406mm

$b := 0.406 \cdot m$ $d := 0.305 \cdot m$

$V_f := \max(V_{f1a}, V_{f2a}) = 61 \cdot kN$

$M_f := \max(M_{f1a}, M_{f2a}) = 93\,m \cdot kN$

$P_f := \max(P_{f1a}, P_{f2a}) = 117.2 \cdot kN$

$fb := 19.5 \cdot MPa$ Specified strength in bending Table 5.3.1C

$Kd := 0.65$ Load duration factor -Permanent for load case 1

$Kh := 1$ System factor

$Ksb := 1$ Service condition factor

$Kt := 1$ Treatment factor

$Fb := fb \cdot Kd \cdot Kh \cdot Ksb \cdot Kt = 12.68 \cdot MPa$

$\Phi := 0.9$

$S := \frac{b^2 \cdot d}{6} = 8379163.33 \cdot mm^3$ section modulus

$Kzb := 0.950$ Size factor in bending for 305mmx356 mm wood

$Kl := 1$ lateral stability factor

$M_r := \Phi \cdot Fb \cdot S \cdot Kzb \cdot Kl$ clause 5.5.4.1

$M_r = 90.8 \cdot kN \cdot m$

$\frac{M_f}{M_r} = 1.02$

Shear Resistance

$fv := 1.5 \cdot MPa$ specified strength in shear Table 5.3.1D

$Ksv := 1$ service condition factor

$Fv := fv \cdot Kd \cdot Kh \cdot Ksv \cdot Kt = 0.98 \cdot MPa$

$An := b \cdot d$ Net area

$Kzv := 0.95$ Size factor in shear

$V_r := \frac{\Phi \cdot Fv \cdot 2 \cdot An \cdot Kzv}{3}$ Clause 5.5.5.1

$V_r = 68.8 \cdot kN$

$\frac{V_f}{V_r} = 0.89$

Compressive Resistance Parallel to Grain

$fc := 13.2 \cdot MPa$ specified strength in compression Table 5.3.1D

$Ksc := 0.91$ service condition factor Table 5.4.2

$Fc := fc \cdot Kd \cdot Kh \cdot Ksc \cdot Kt = 7.81 \cdot MPa$

$Kzc := \min\left[6.3 \cdot \left(\frac{b \cdot L}{m^2}\right)^{-0.13}, 1.3\right] = 1.3$

$K_{SE} := 1.0$ Table 5.4.2

$E_{05} := 8000 \cdot MPa$ Table 5.3.1D

$C_c := \frac{L}{b} = 14.78$

$K_c := \left(1.0 + \frac{Fc \cdot Kzc \cdot C_c^{3}}{35 \cdot E_{05} \cdot K_{SE} \cdot Kt}\right)^{-1} = 0.9$

$\Phi_c := 0.8$

$P_r := \Phi_c \cdot Fc \cdot An \cdot Kzc \cdot K_c = 900.19 \cdot kN$ Clause 5.5.6.2.2

$Check := \frac{M_f}{M_r} + \frac{P_f}{P_r} = 1.15$

$Gate_structural_capacity1 := \begin{cases} \text{"ok"} & \text{if } V_r \geq V_f \wedge \frac{M_f}{M_r} + \frac{P_f}{P_r} \leq 1.0 \\ \text{"not good"} & \text{otherwise} \end{cases}$

Gate_structural_capacity1 = "not good"

Standard Duration Condition - Ice load condition

Bending moment Resistance For Log 305mm x 406mm

$b := 0.406 \cdot m$ $d := 0.305 \cdot m$

$V_f := V_{f2b} = 137.3 \cdot kN$

$M_f := M_{f2b} = 209\,m \cdot kN$

$P_f := P_{f2b} = 263.7 \cdot kN$

$fb := 19.5 \cdot MPa$ Specified strength in bending Table 5.3.1C

$Kd := 1.0$ Load duration factor -Permanent for load case 2

$Kh := 1$ System factor

$Ksb := 1$ Service condition factor

$Kt := 1$ Treatment factor

$Fb := fb \cdot Kd \cdot Kh \cdot Ksb \cdot Kt = 19.5 \cdot MPa$

$\Phi := 0.9$

$S := \frac{b^2 \cdot d}{6} = 8379163.33 \cdot mm^3$ section modulus

$Kzb := 0.950$ Size factor in bending for 305mmx356 mm wood

$Kl := 1$ lateral stability factor

$M_r := \Phi \cdot Fb \cdot S \cdot Kzb \cdot Kl$ clause 5.5.4.1

$M_r = 139.7 \cdot kN \cdot m$

$\frac{M_f}{M_r} = 1.5$

Shear Resistance

$fv := 1.5 \cdot MPa$ specified strength in shear Table 5.3.1D

$Ksv := 1$ service condition factor

$Fv := fv \cdot Kd \cdot Kh \cdot Ksv \cdot Kt = 1.5 \cdot MPa$

$An := b \cdot d$ Net area

$Kzv := 0.95$ Size factor in shear

$V_r := \frac{\Phi \cdot Fv \cdot 2 \cdot An \cdot Kzv}{3}$ Clause 5.5.5.1

$V_r = 105.9 \cdot kN$

$\frac{V_f}{V_r} = 1.3$

Compressive Resistance Parallel to Grain

$fc := 13.2 \cdot MPa$ specified strength in compression — Table 5.3.1D

$Ksc := 0.91$ service condition factor — Table 5.4.2

$Fc := fc \cdot Kd \cdot Kh \cdot Ksc \cdot Kt = 12.01 \cdot MPa$

$Kzc := \min\left[6.3 \cdot \left(\frac{b \cdot L}{m^2}\right)^{-0.13}, 1.3\right] = 1.3$

$K_{SE} := 1.0$ — Table 5.4.2

$E_{05} := 8000 \cdot MPa$ — Table 5.3.1D

$C_c := \frac{L}{b} = 14.78$

$K_c := \left(1.0 + \frac{Fc \cdot Kzc \cdot C_c^{\,3}}{35 \cdot E_{05} \cdot K_{SE} \cdot Kt}\right)^{-1} = 0.85$

$\Phi_c := 0.8$

$P_r := \Phi_c \cdot Fc \cdot An \cdot Kzc \cdot K_c = 1310.97 \cdot kN$ — Clause 5.5.6.2.2

$Check := \frac{M_f}{M_r} + \frac{P_f}{P_r} = 1.7$

$$Gate_structural_capacity1 := \begin{cases} \text{"ok"} & \text{if } V_r \geq V_f \wedge \frac{M_f}{M_r} + \frac{P_f}{P_r} \leq 1.0 \\ \text{"not good"} & \text{otherwise} \end{cases}$$

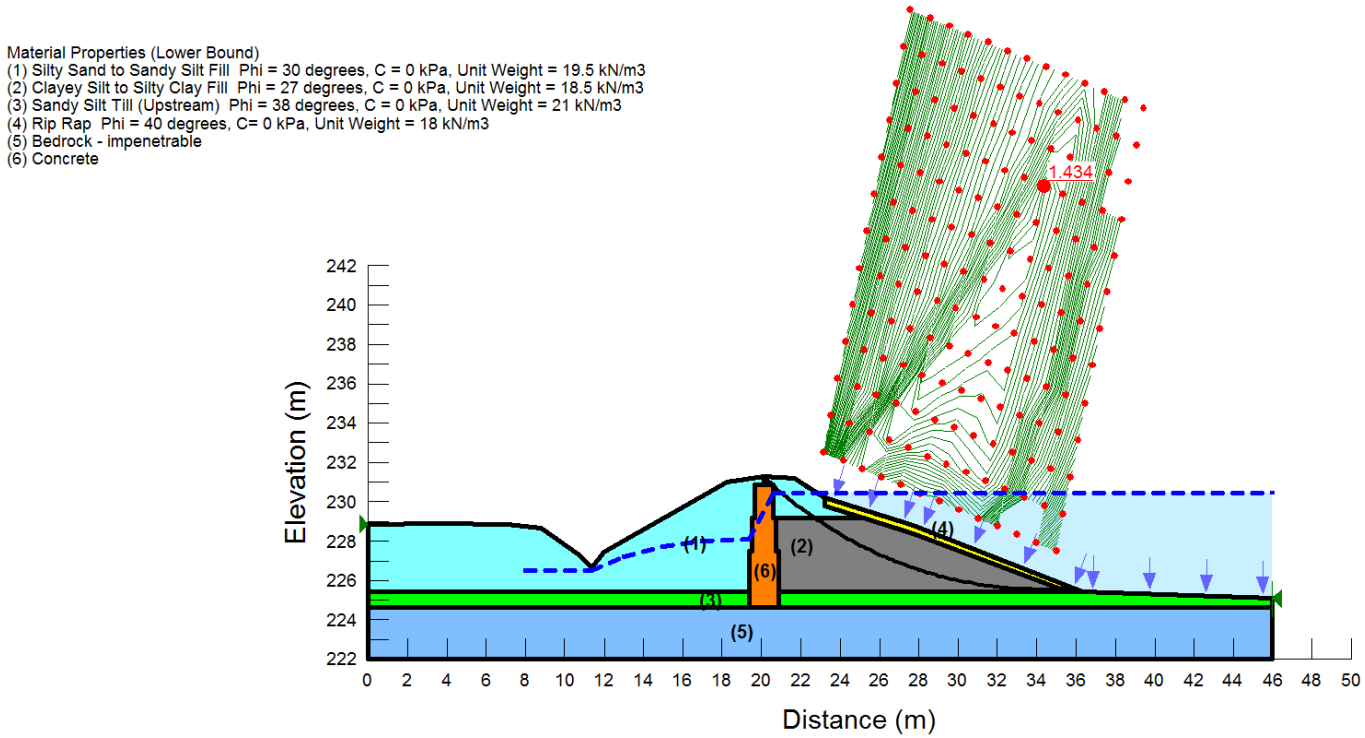
Gate_structural_capacity1 = "not good"

APPENDIX L

GEOTECHNICAL EMBANKMENT STABILITY ANALYSES

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties



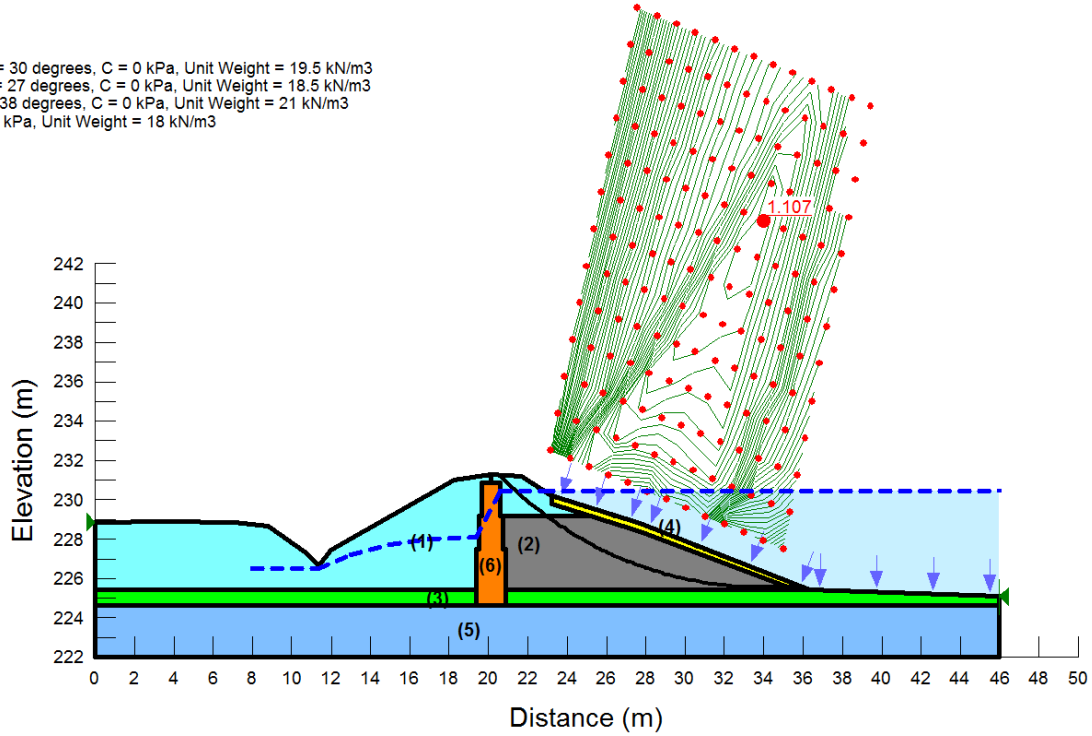
Case I: Long-term Normal Operating Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-1	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

Material Properties (Lower Bound)
(1) Silty Sand to Sandy Silt Fill Phi = 30 degrees, C = 0 kPa, Unit Weight = 19.5 kN/m3
(2) Clayey Silt to Silty Clay Fill Phi = 27 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(3) Sandy Silt Till (Upstream) Phi = 38 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(4) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(5) Bedrock - impenetrable
(6) Concrete



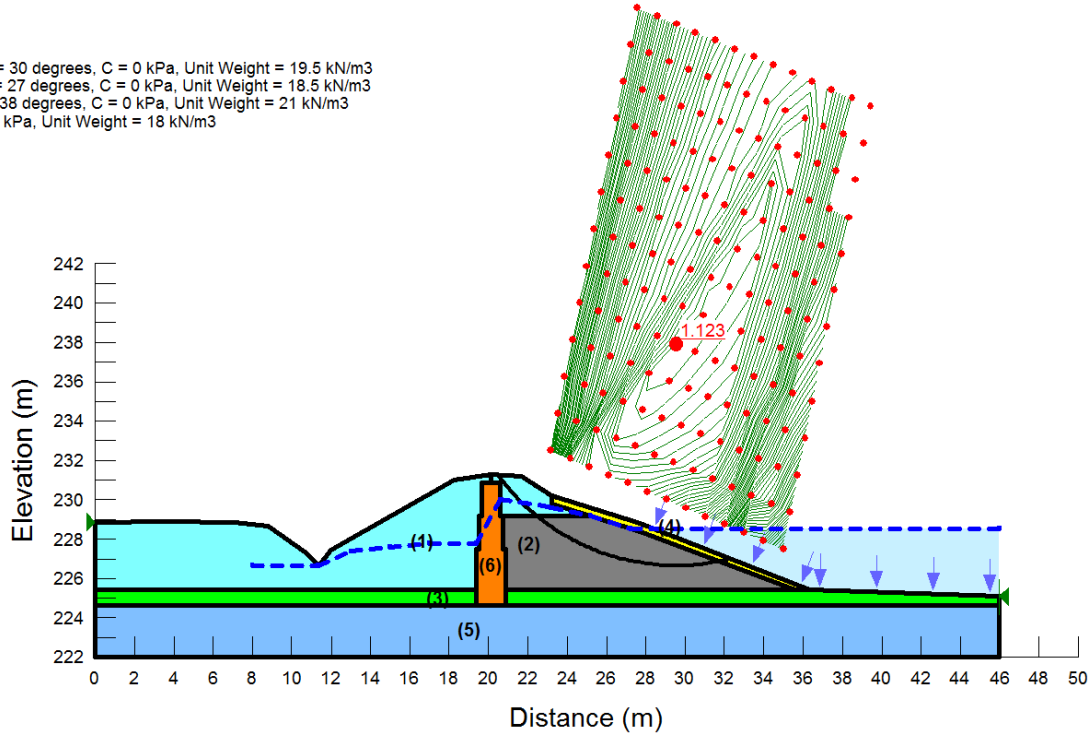
Case II: Extreme Short-term Earthquake Loading (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-2	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

Material Properties (Lower Bound)
(1) Silty Sand to Sandy Silt Fill Phi = 30 degrees, C = 0 kPa, Unit Weight = 19.5 kN/m3
(2) Clayey Silt to Silty Clay Fill Phi = 27 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(3) Sandy Silt Till (Upstream) Phi = 38 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(4) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(5) Bedrock - impenetrable
(6) Concrete

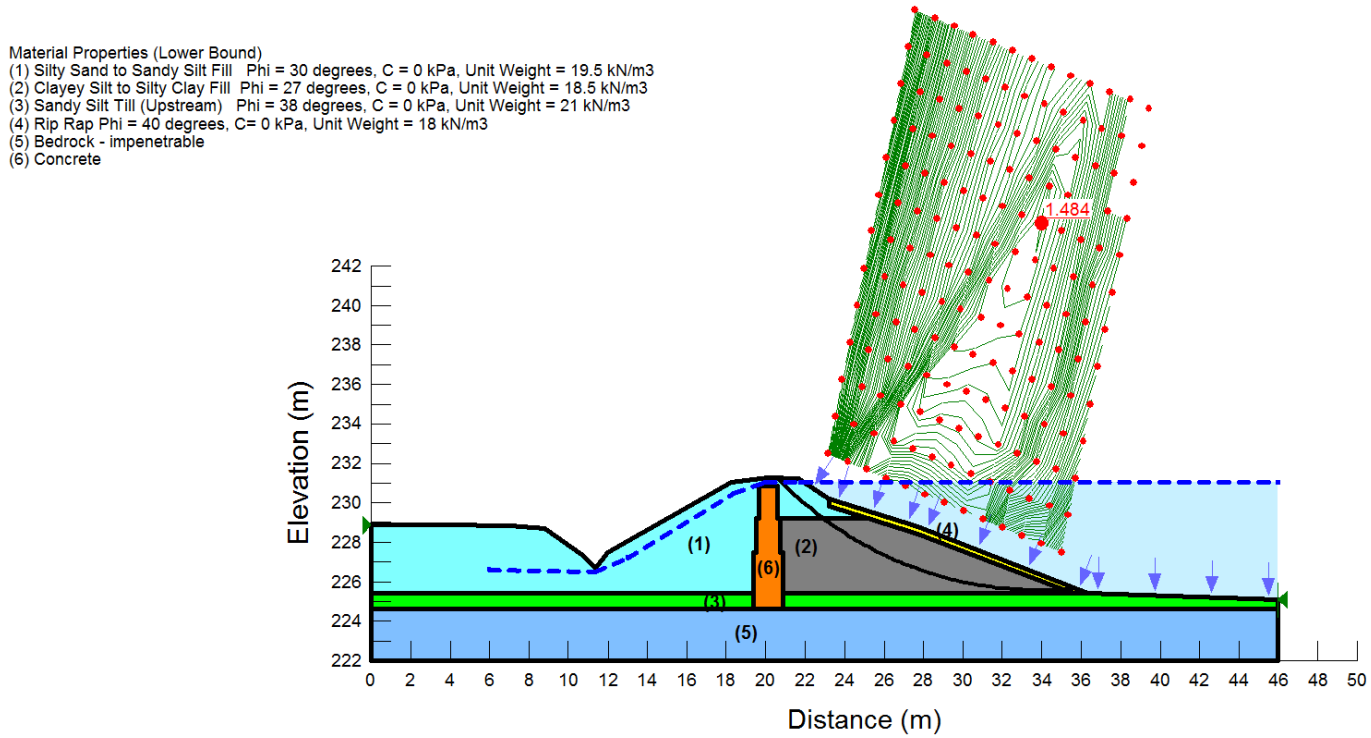


Case III: Short Rapid Drawdown Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-3	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

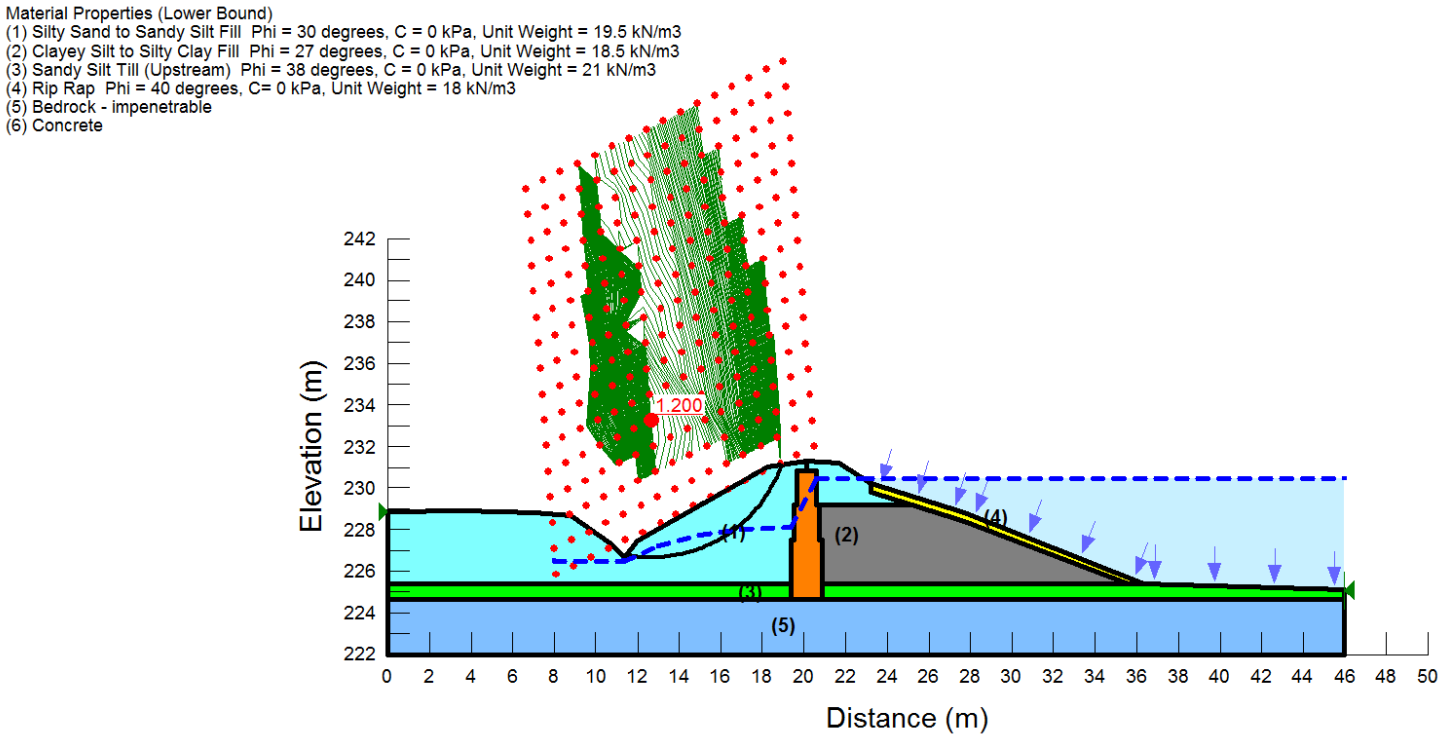


Case IV: Short term IDF condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-4	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

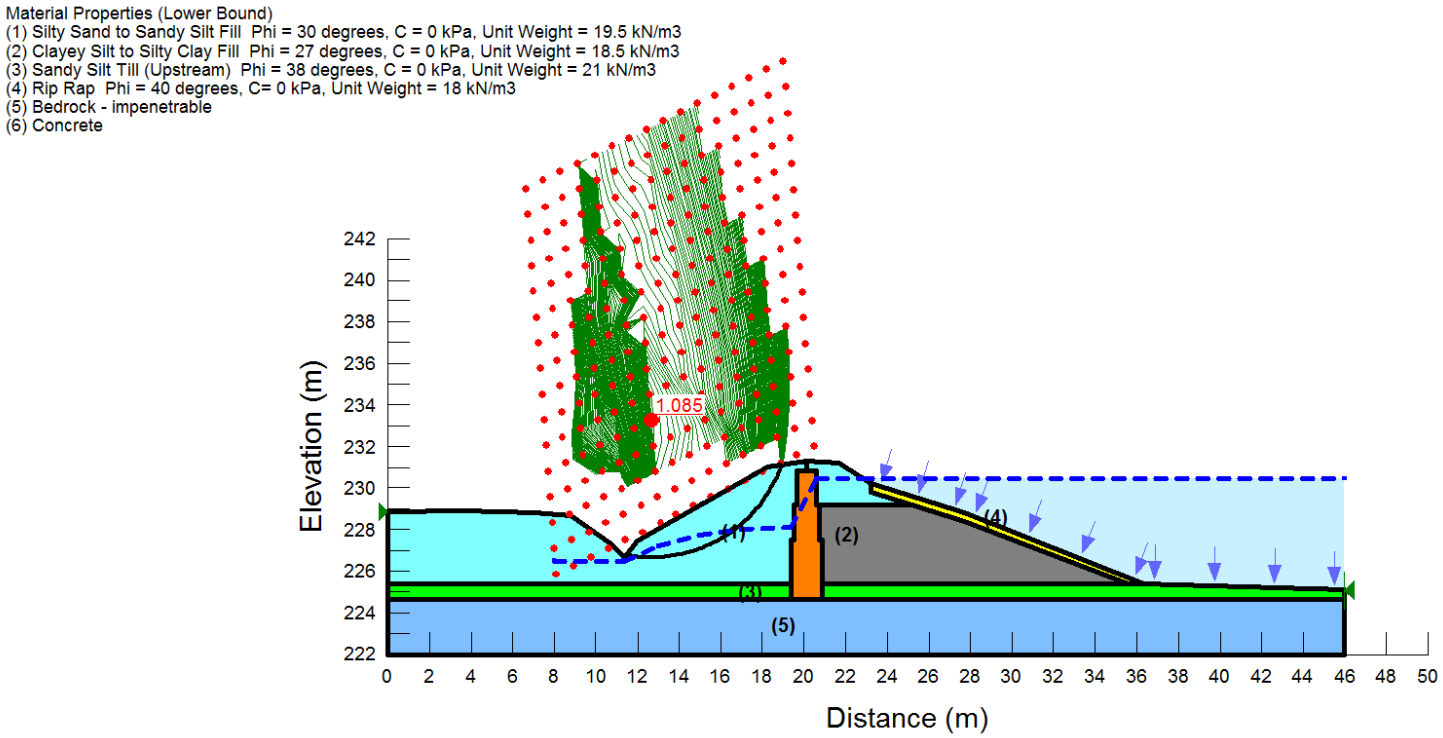


Case I: Long-term Normal Operating Condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-5	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

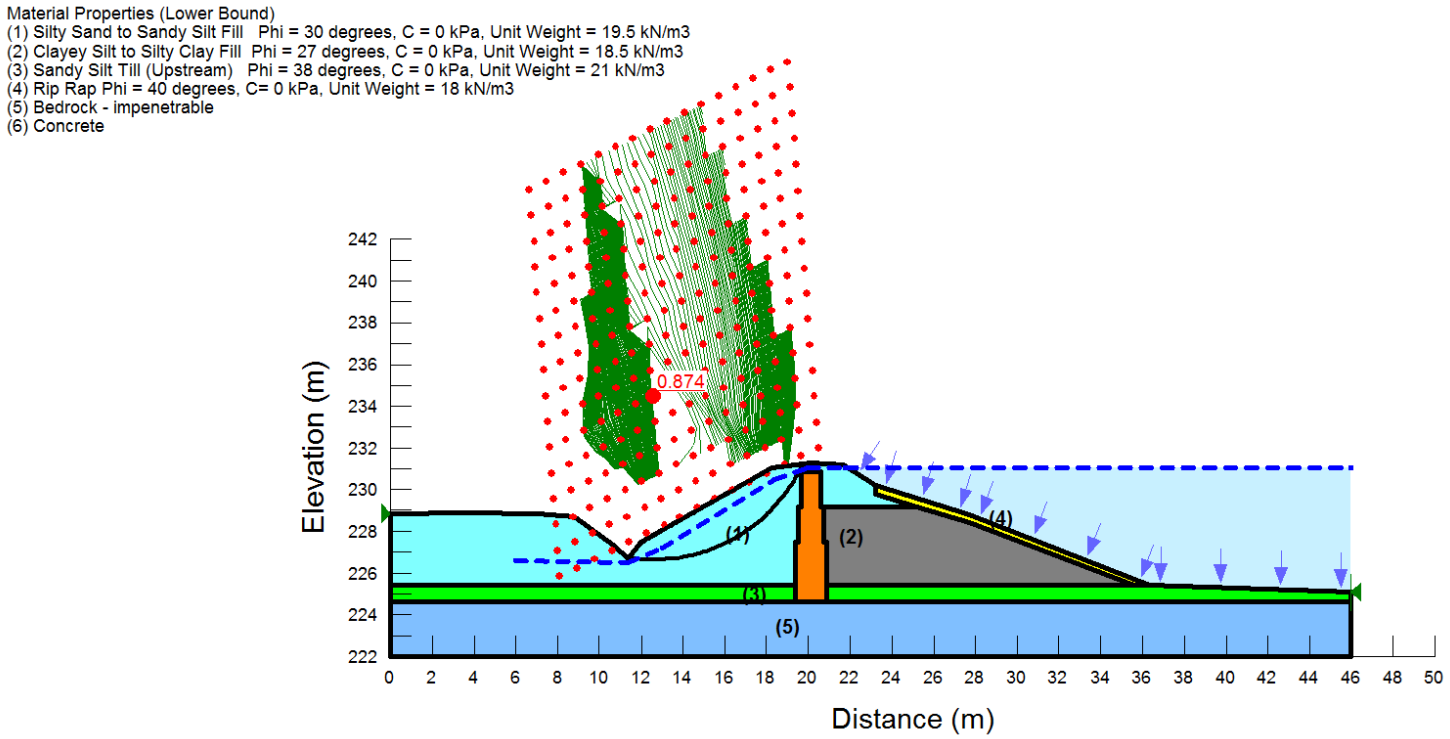


Case II: Extreme Short-term Earthquake Loading (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-6	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

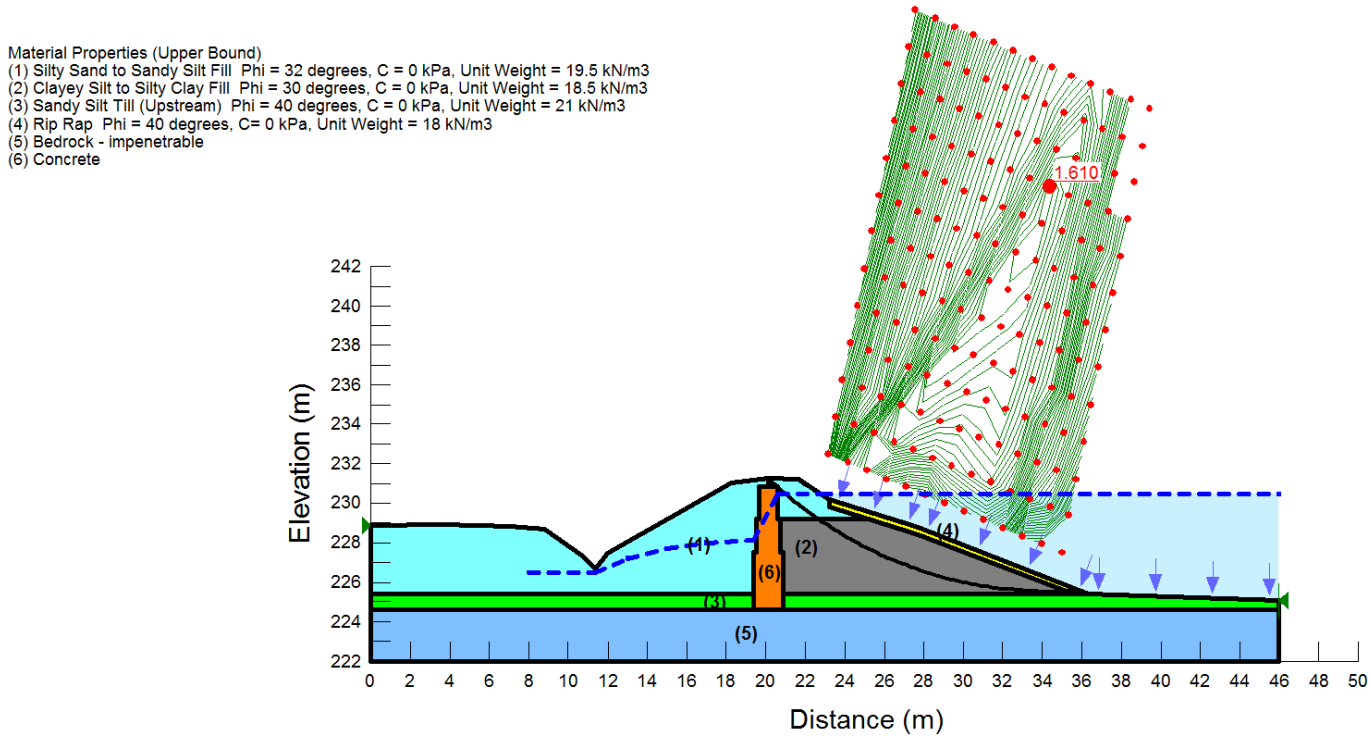


Case IV: Short term IDF condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-7	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

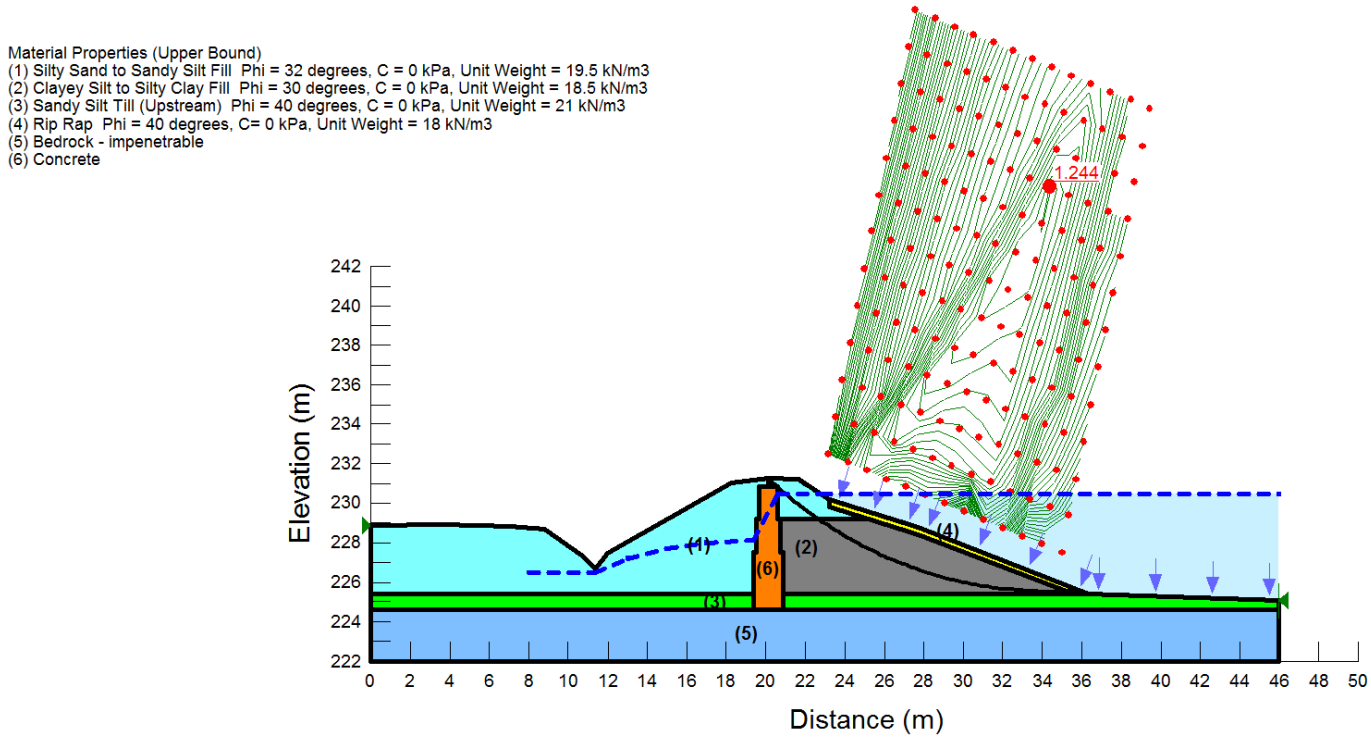


Case I: Long-term Normal Operating Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-1	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

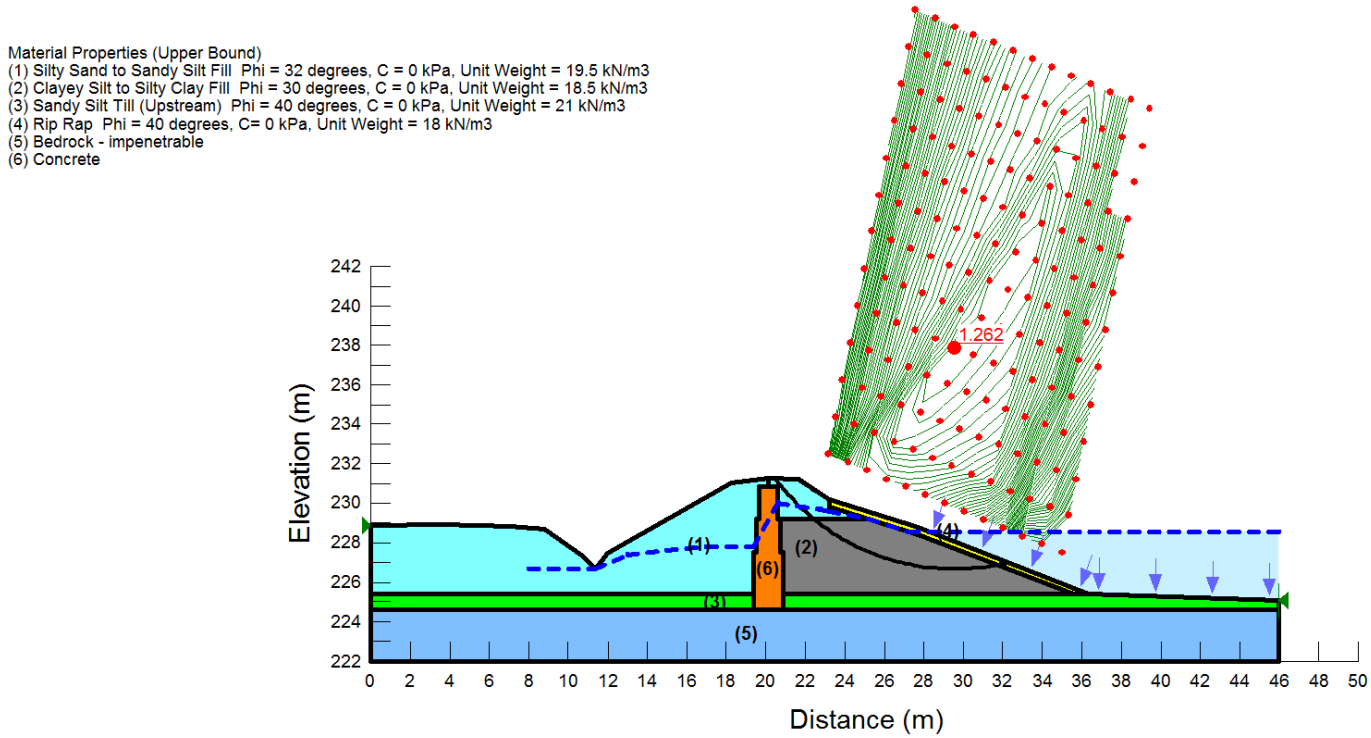


Case II: Extreme Short-term Earthquake Loading (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-2	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

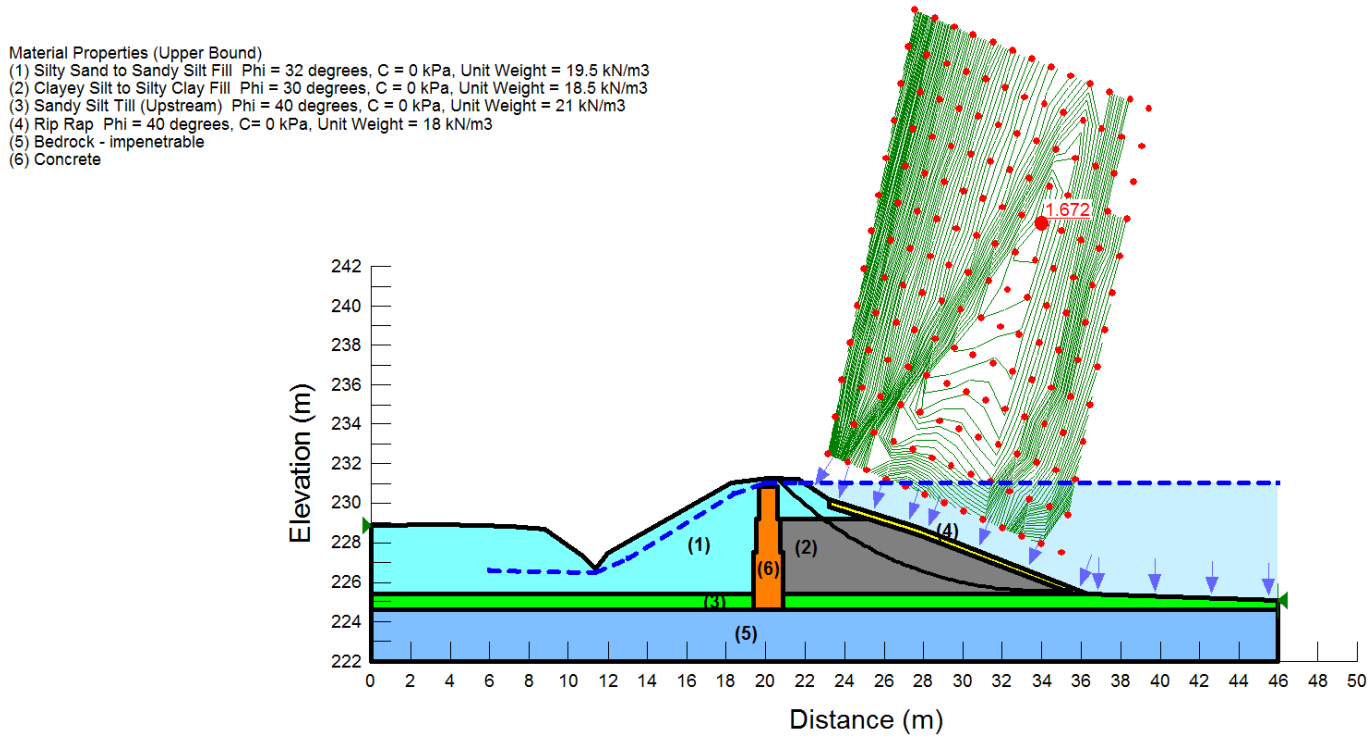


Case III: Short Rapid Drawdown Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-3	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

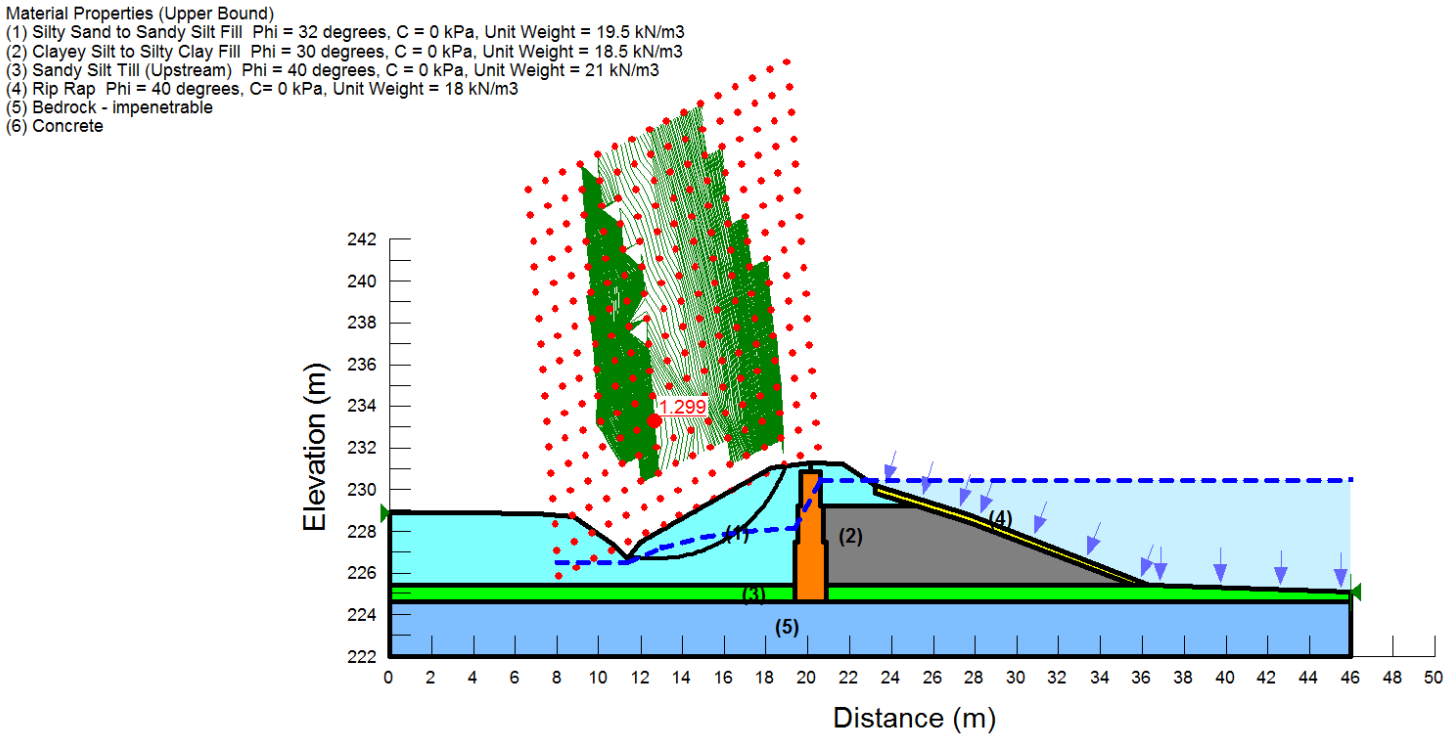


Case IV: Short term IDF condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-4	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

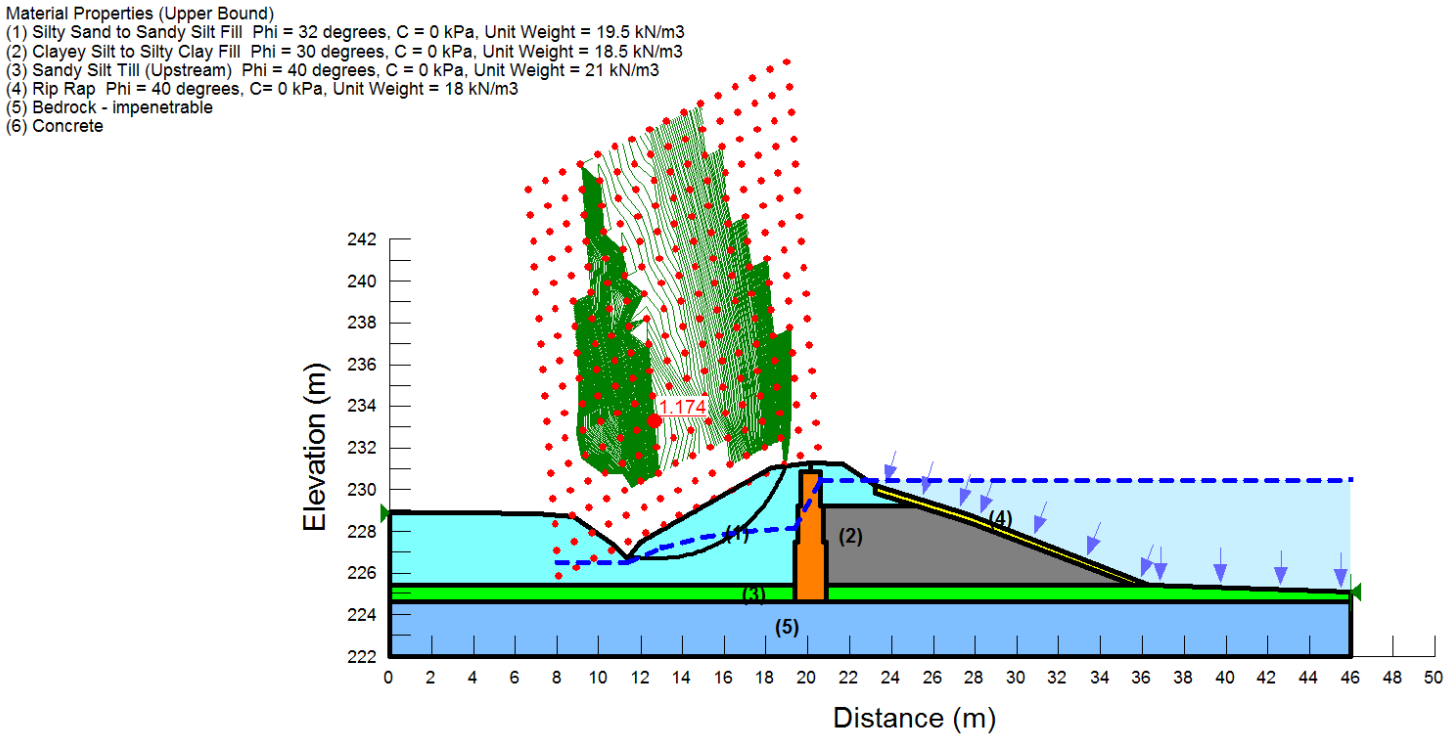


Case I: Long-term Normal Operating Condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-5	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

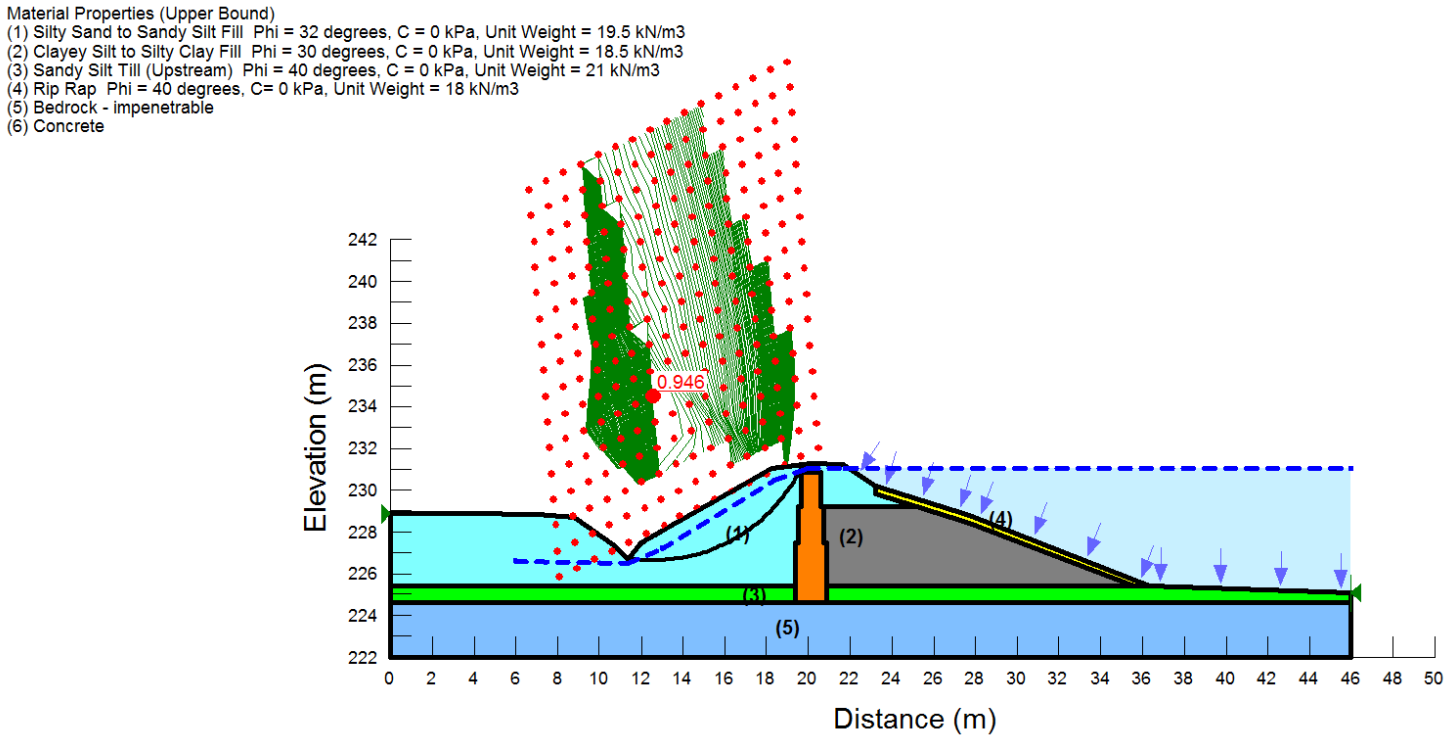


Case II: Extreme Short-term Earthquake Loading (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-6	

Talbot Dam and Lock 38 – Right Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties



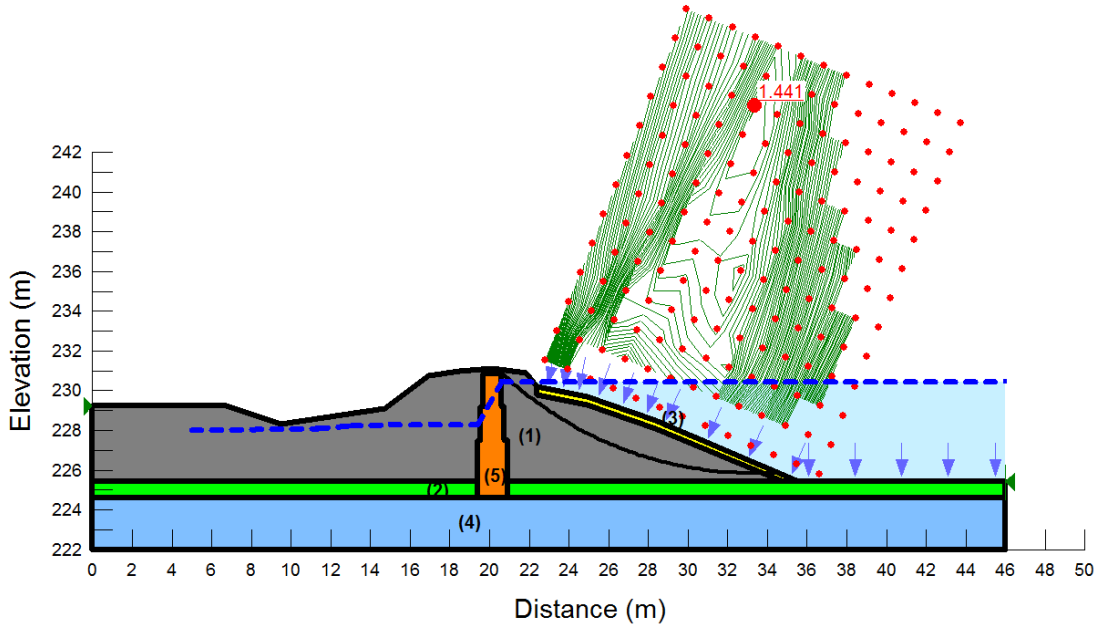
Case IV: Short term IDF condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
RIGHT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-7	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

Material Properties (Lower Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 27 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 38 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete



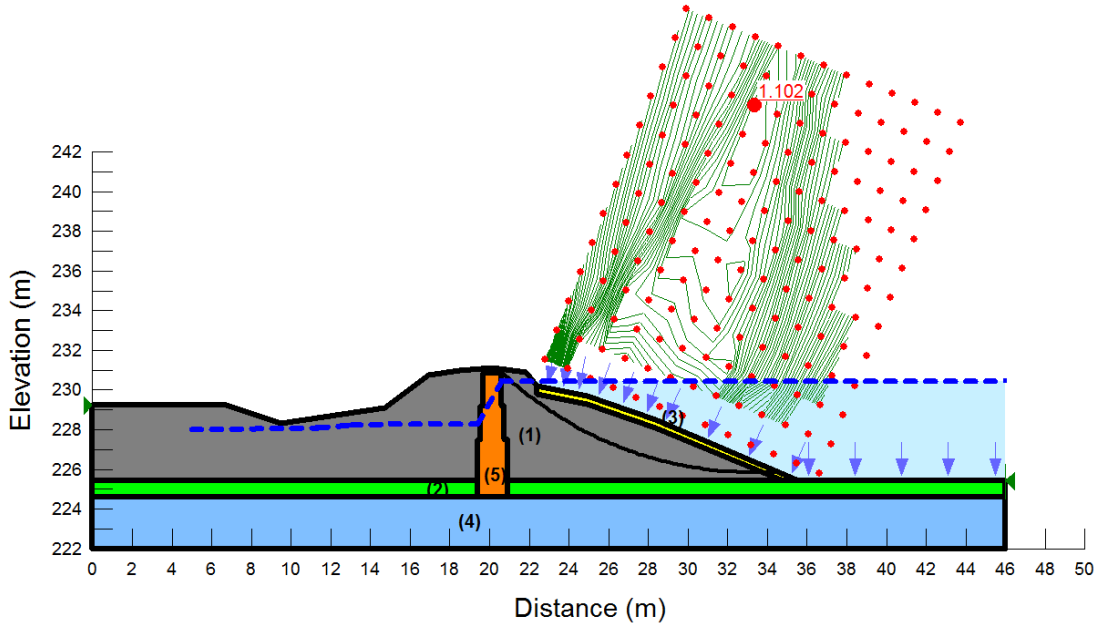
Case I: Long-term Normal Operating Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-1	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

Material Properties (Lower Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 27 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 38 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete



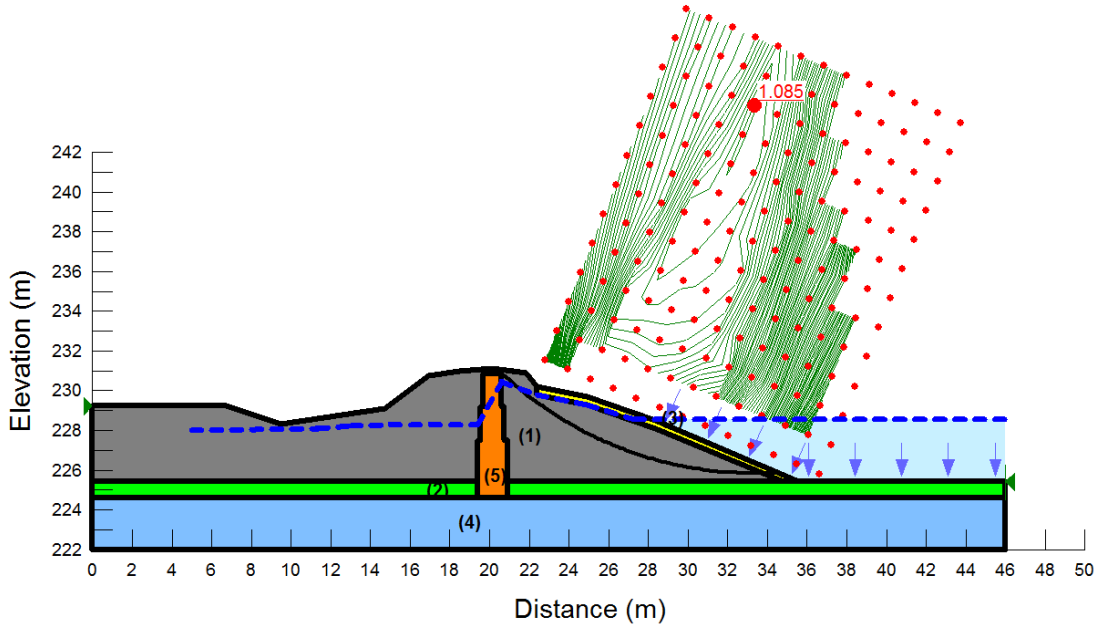
Case II: Extreme Short-term Earthquake Loading (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-2	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

Material Properties (Lower Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 27 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 38 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete



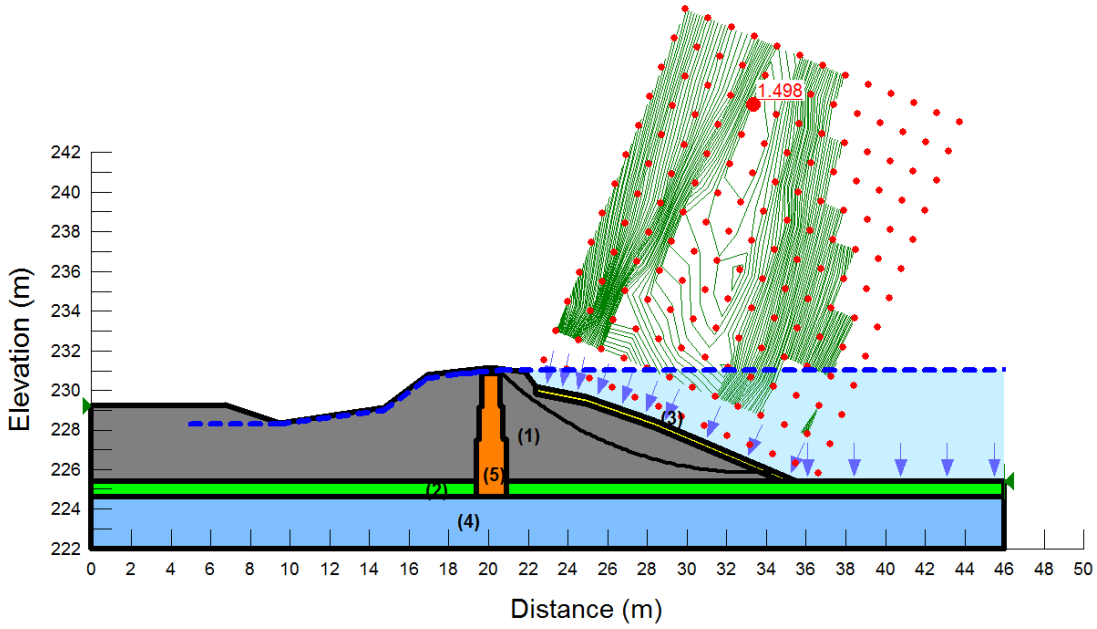
Case III: Short Rapid Drawdown Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-3	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

Material Properties (Lower Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 27 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 38 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete



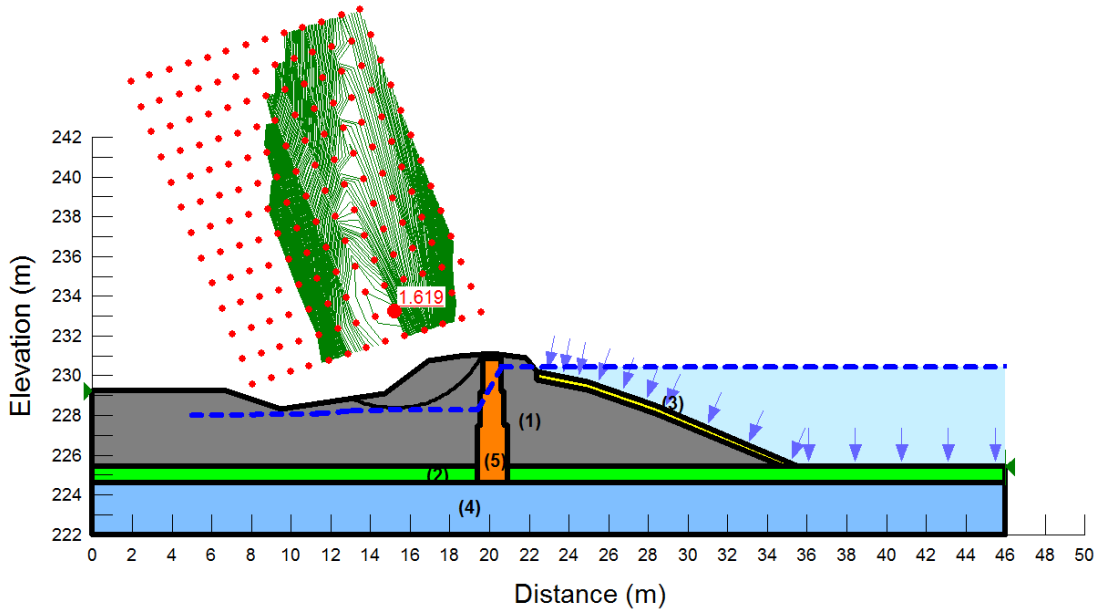
Case IV: Short term IDF condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-4	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

Material Properties (Lower Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 27 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 38 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete



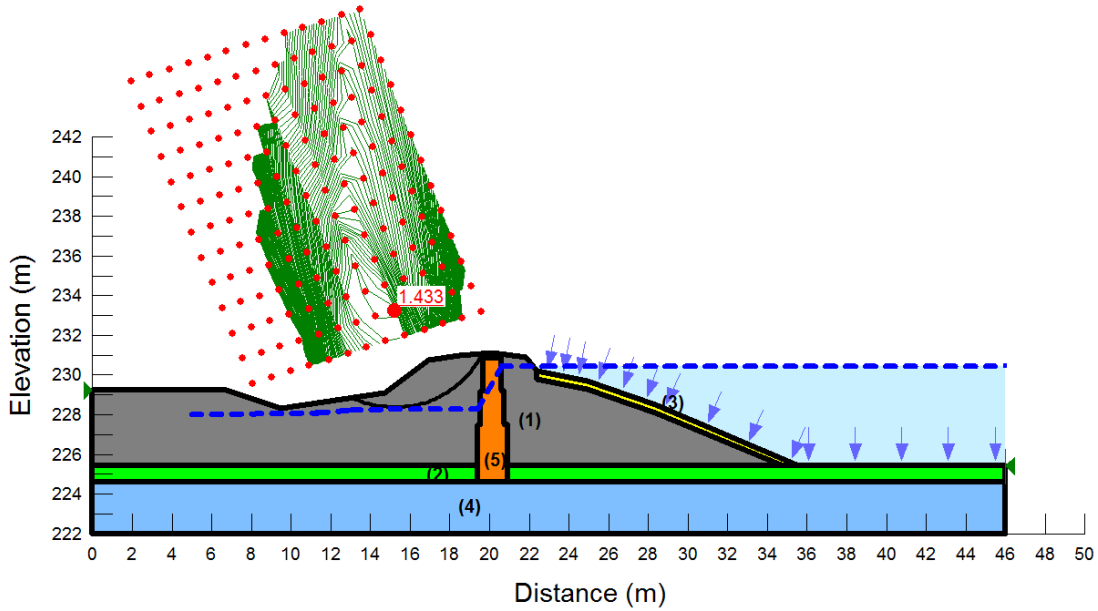
Case I: Long-term Normal Operating Condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-5	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

Material Properties (Lower Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 27 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 38 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete



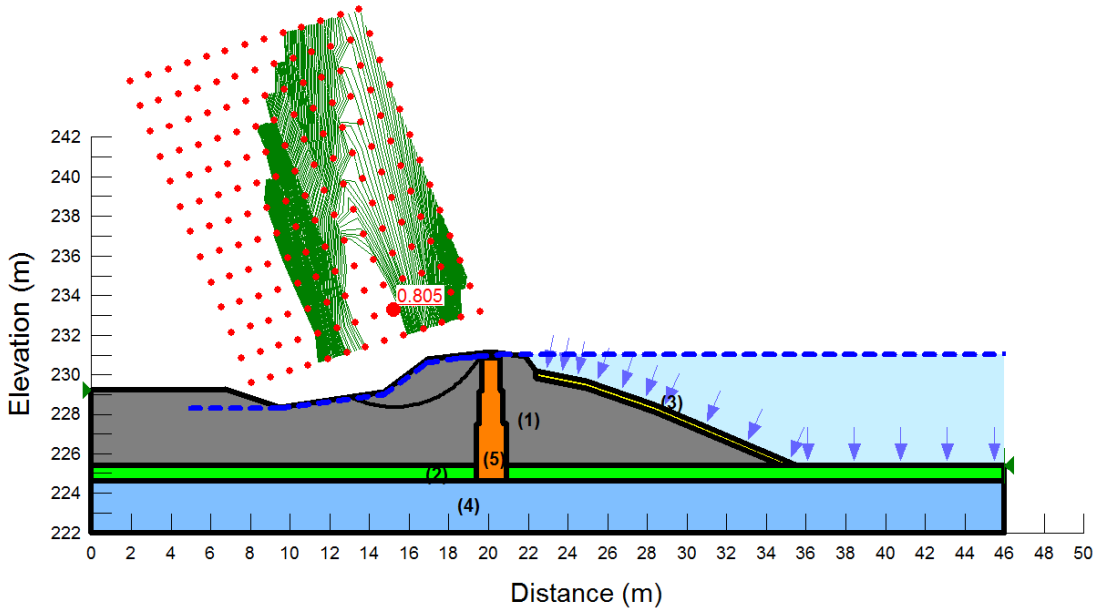
Case II: Extreme Short-term Earthquake Loading (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-6	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

Material Properties (Lower Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 27 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 38 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete



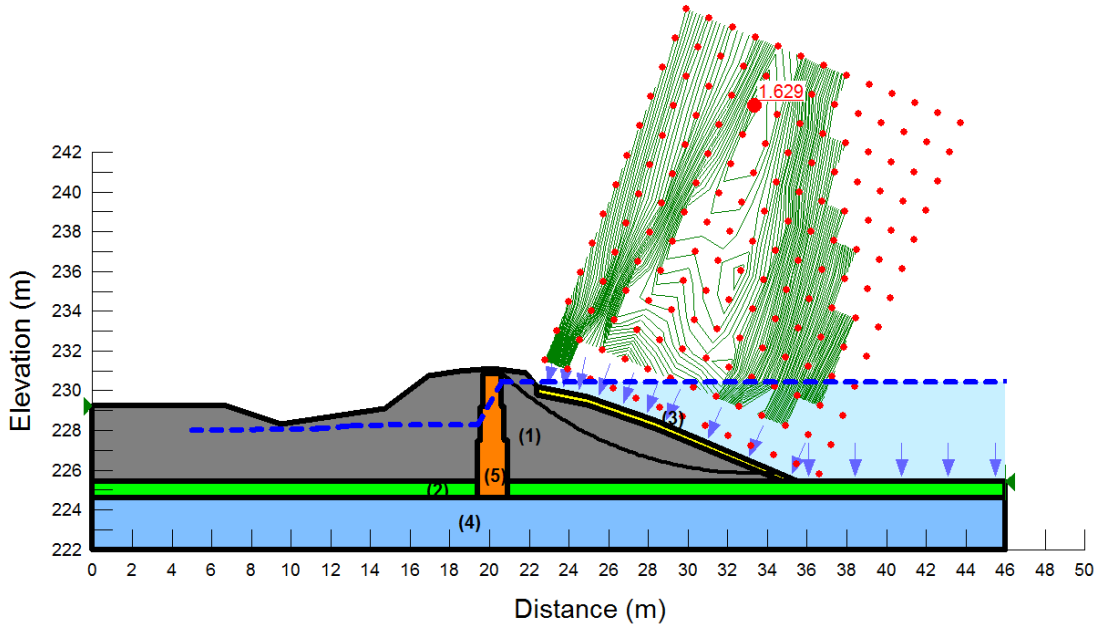
Case IV: Short term IDF condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-7	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

Material Properties (Upper Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 30 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 40 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete

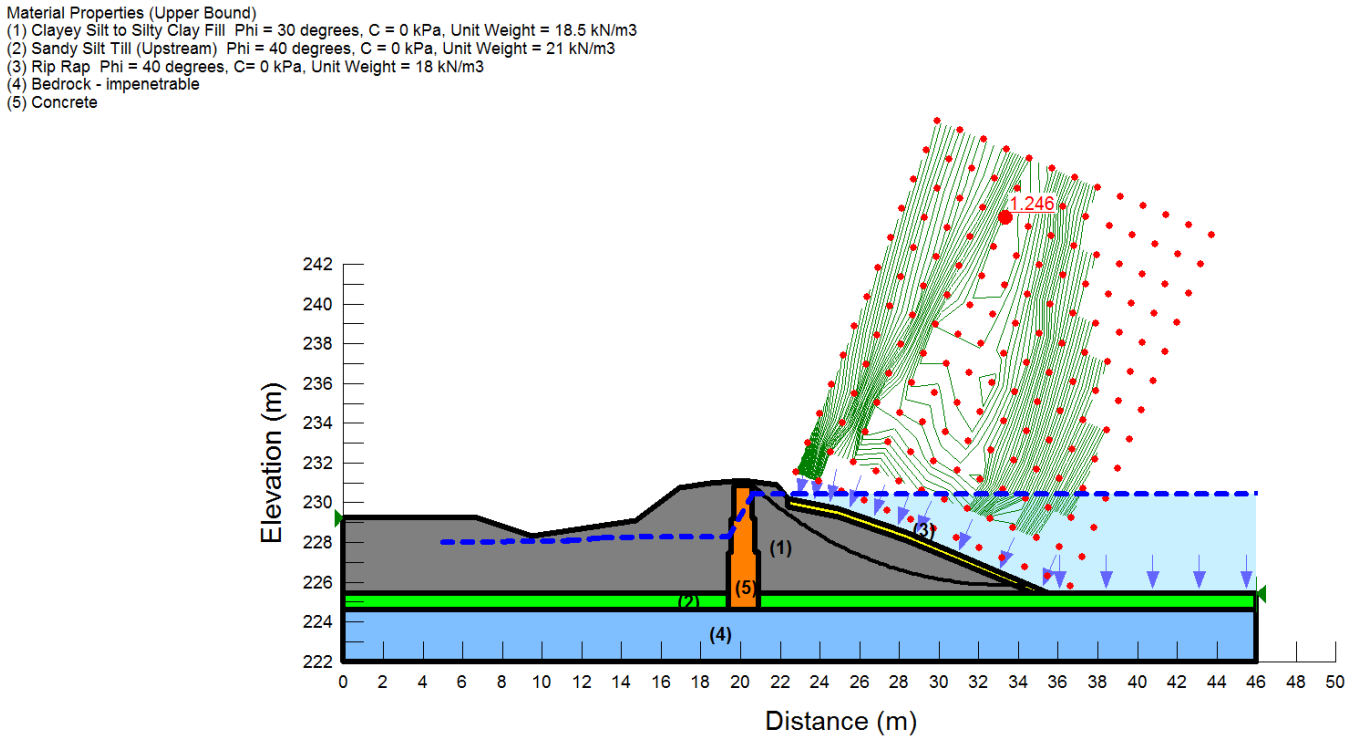


Case I: Long-term Normal Operating Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-1	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties



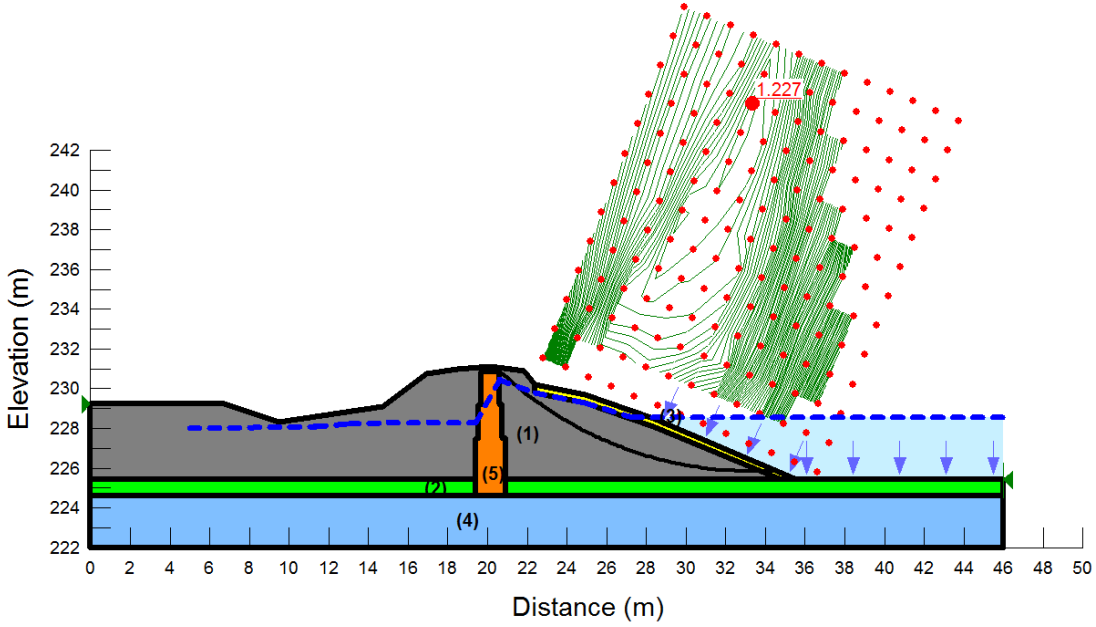
Case II: Extreme Short-term Earthquake Loading (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-2	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

Material Properties (Upper Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 30 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 40 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete



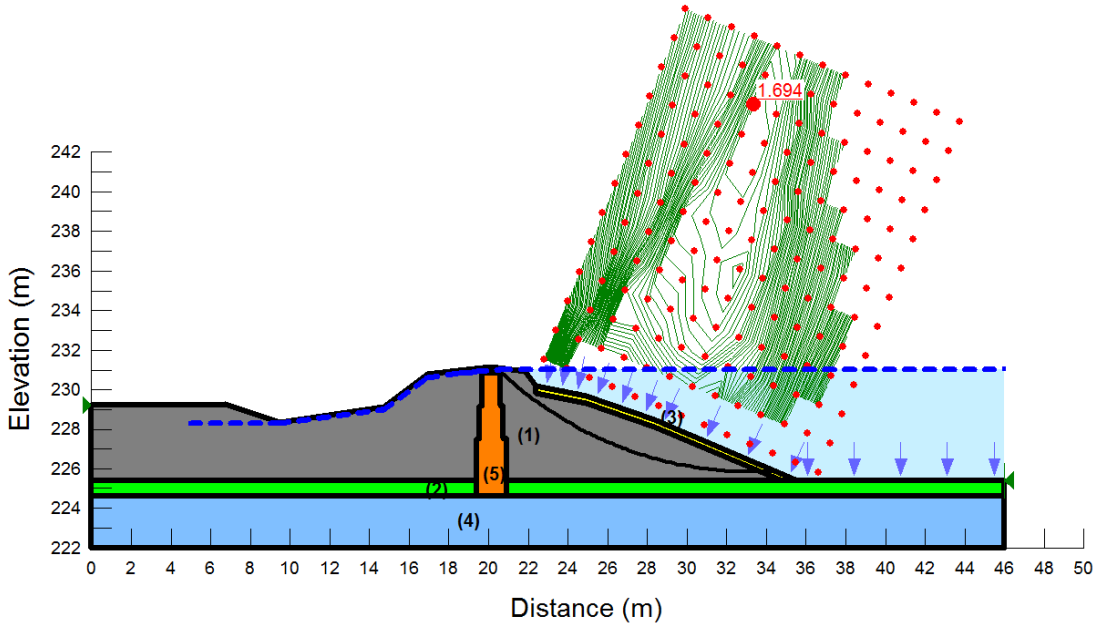
Case III: Short Rapid Drawdown Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-3	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

Material Properties (Upper Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 30 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 40 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete



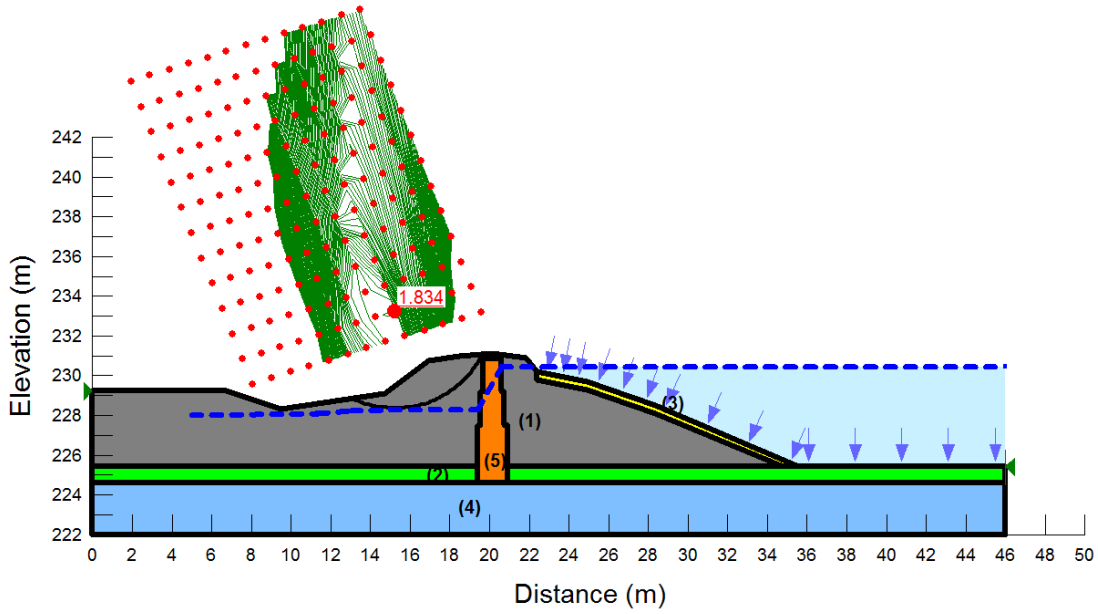
Case IV: Short term IDF condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-4	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

Material Properties (Upper Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 30 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 40 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete



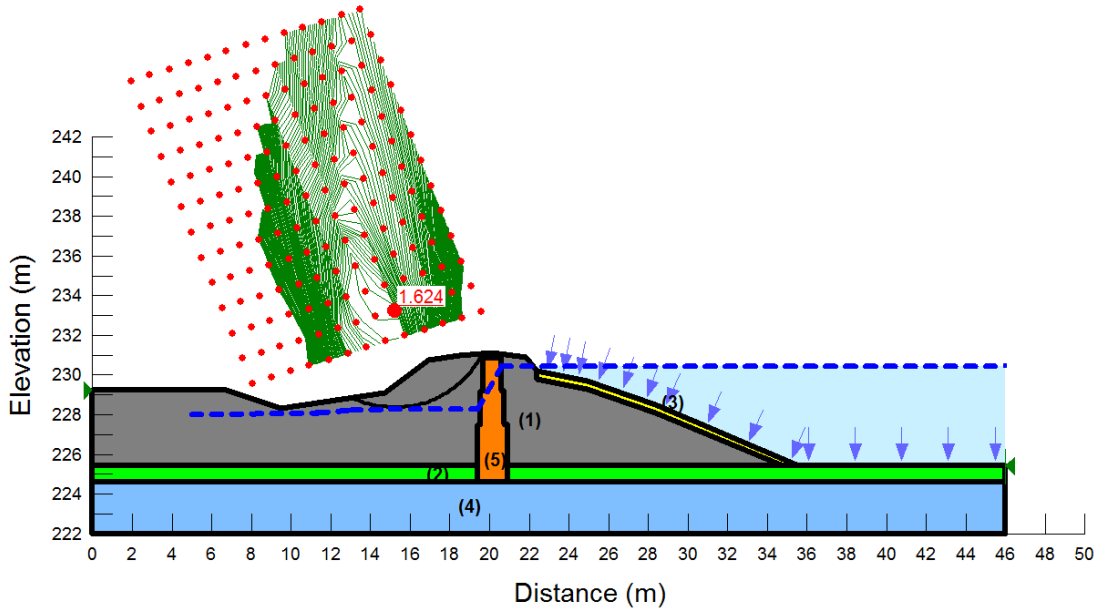
Case I: Long-term Normal Operating Condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-5	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

Material Properties (Upper Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 30 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 40 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete



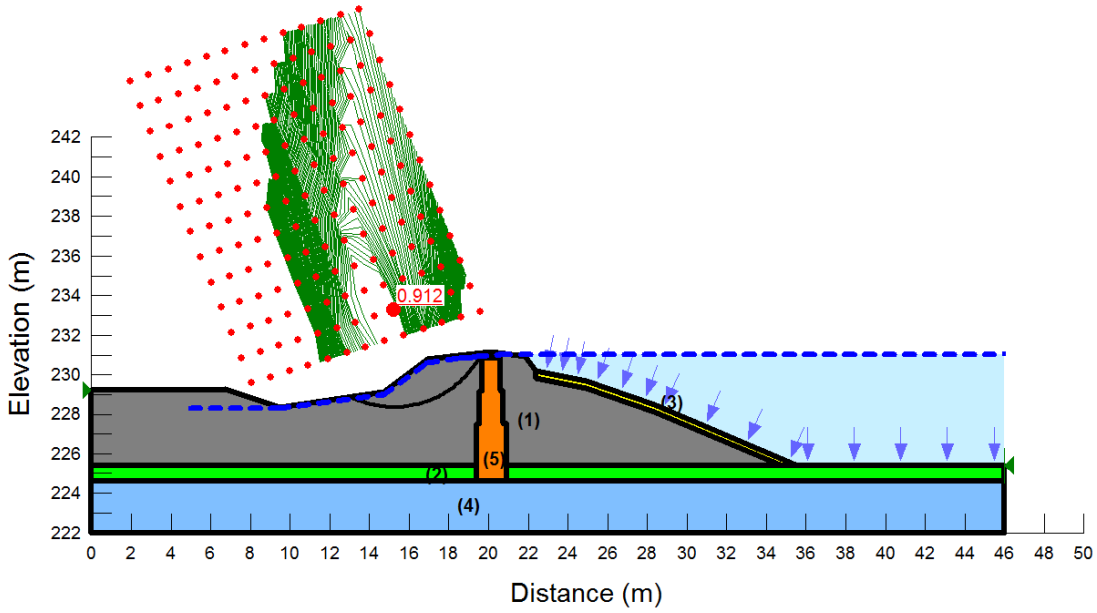
Case II: Extreme Short-term Earthquake Loading (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-6	

Talbot Dam and Lock 38 – Left Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

Material Properties (Upper Bound)
(1) Clayey Silt to Silty Clay Fill Phi = 30 degrees, C = 0 kPa, Unit Weight = 18.5 kN/m3
(2) Sandy Silt Till (Upstream) Phi = 40 degrees, C = 0 kPa, Unit Weight = 21 kN/m3
(3) Rip Rap Phi = 40 degrees, C= 0 kPa, Unit Weight = 18 kN/m3
(4) Bedrock - impenetrable
(5) Concrete

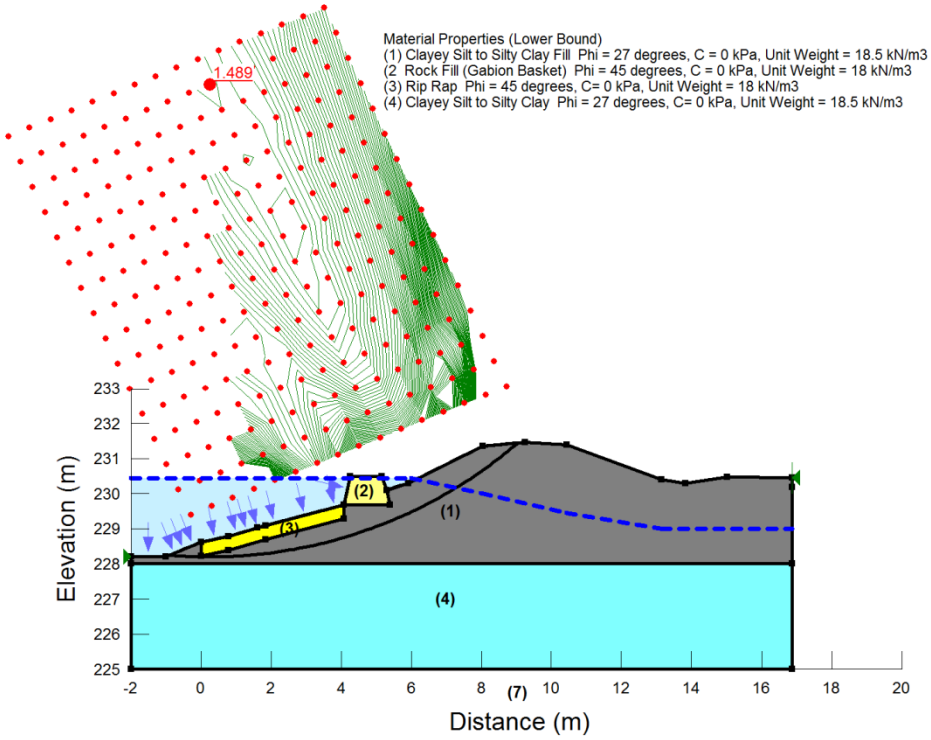


Case IV: Short term IDF condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
LEFT EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-7	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

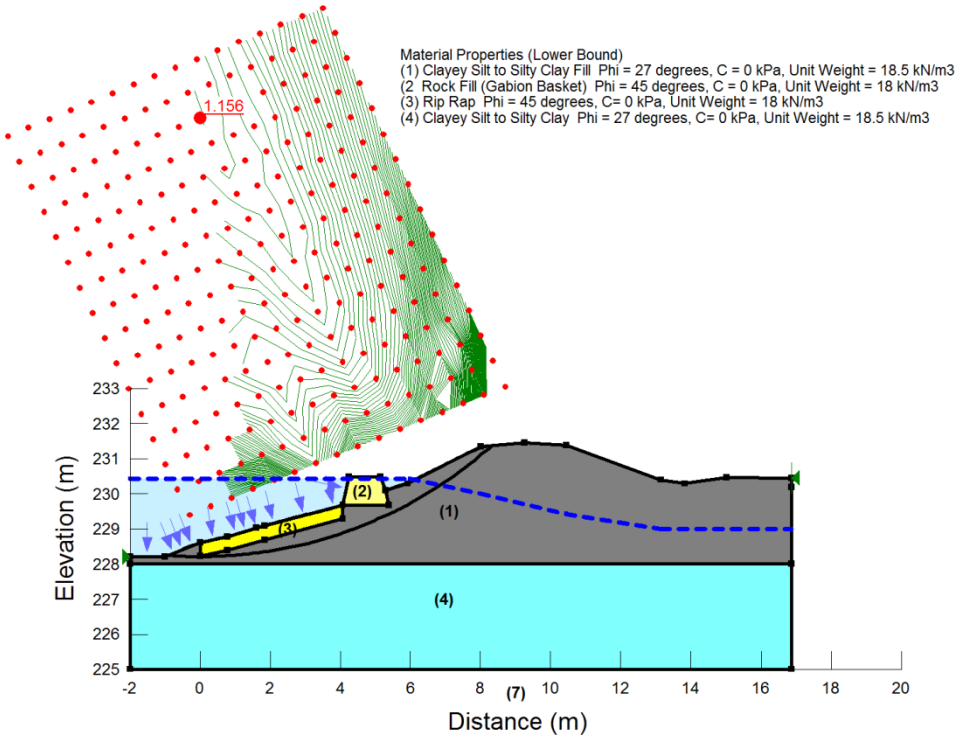


Case I: Long-term Normal Operating Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-1	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

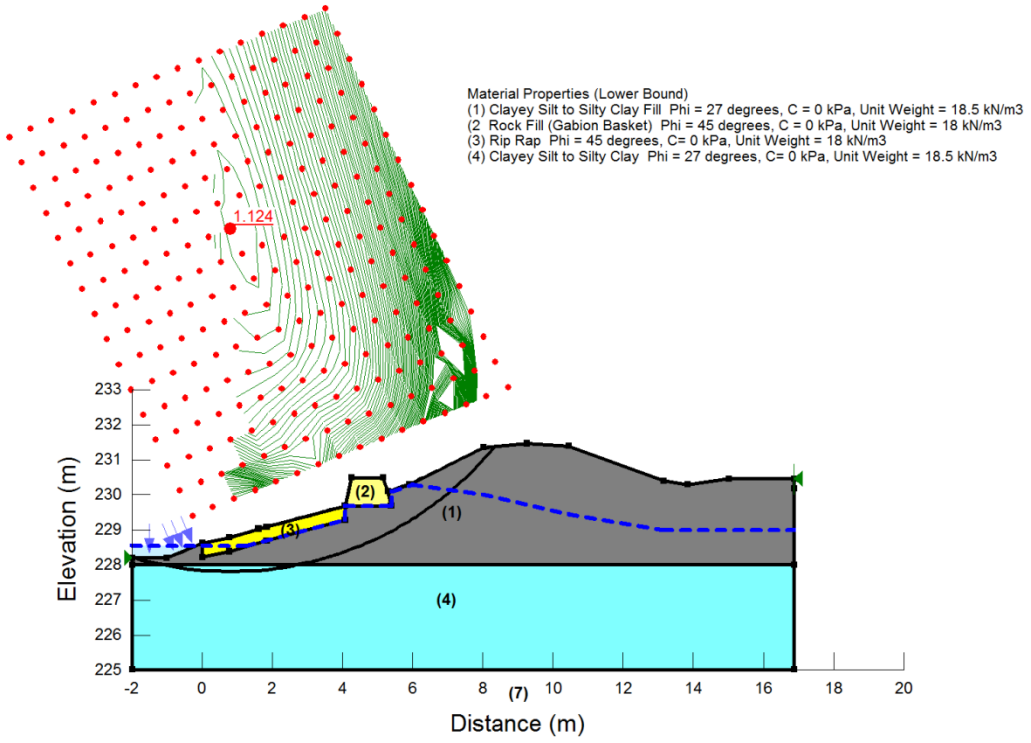


Case II: Extreme Short-term Earthquake Loading (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-2	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

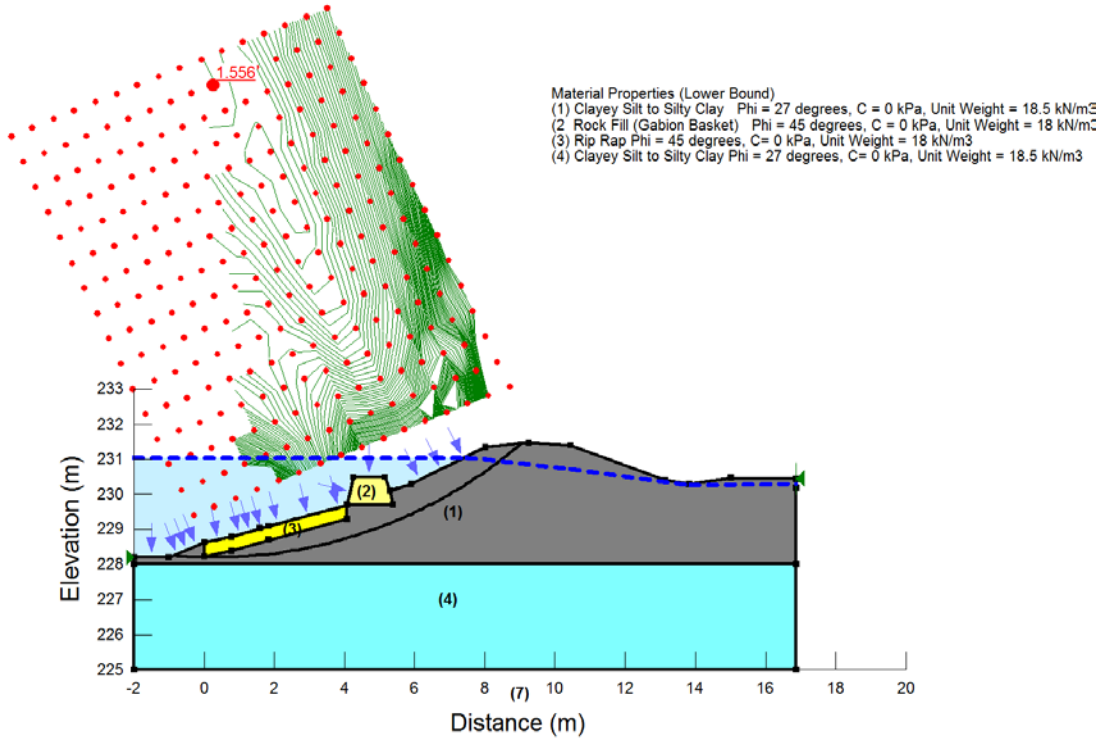


Case III: Short Rapid Drawdown Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-3	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

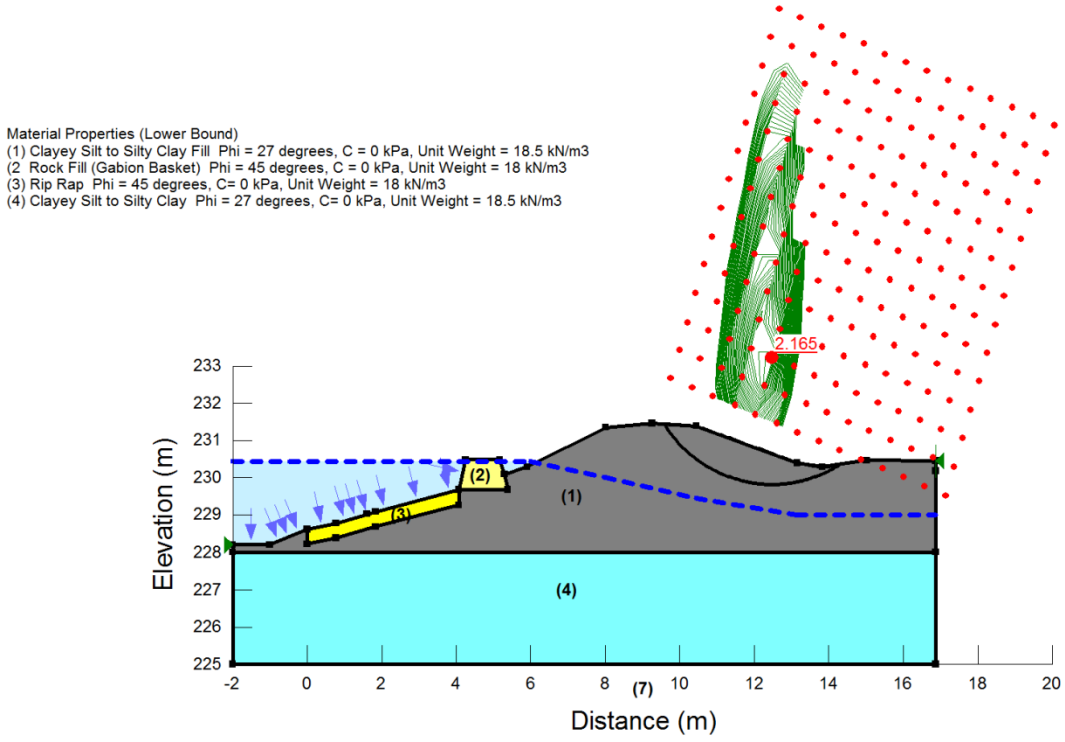


Case IV: Short term IDF condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-4	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

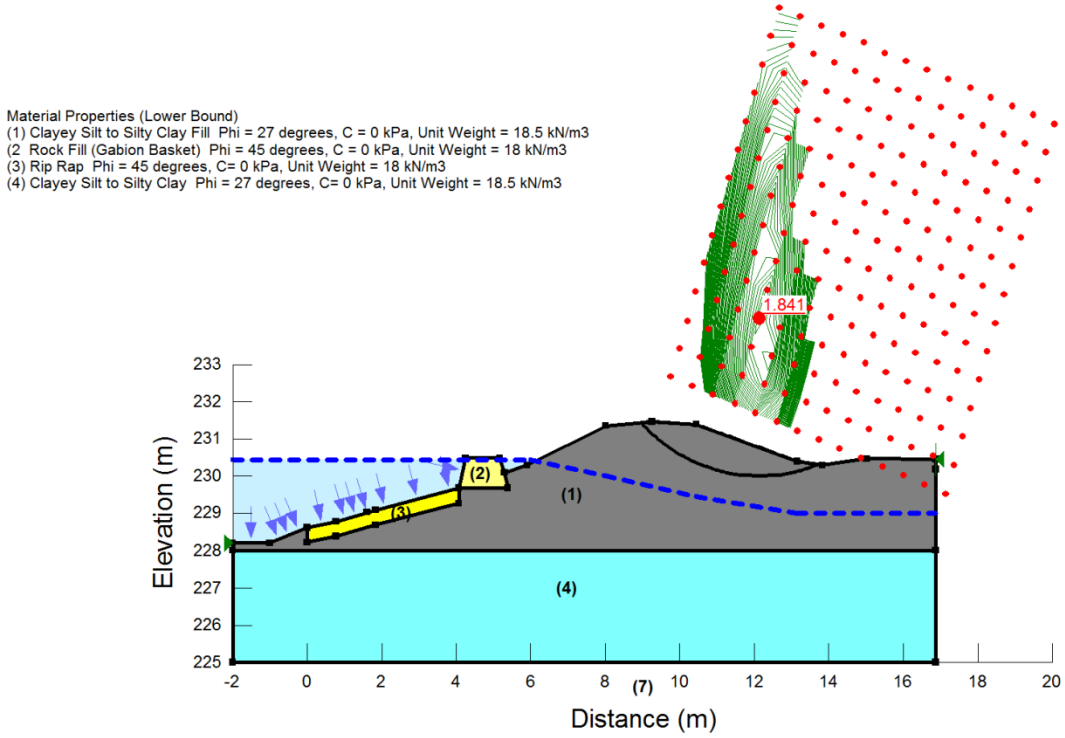


Case I: Long-term Normal Operating Condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-5	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

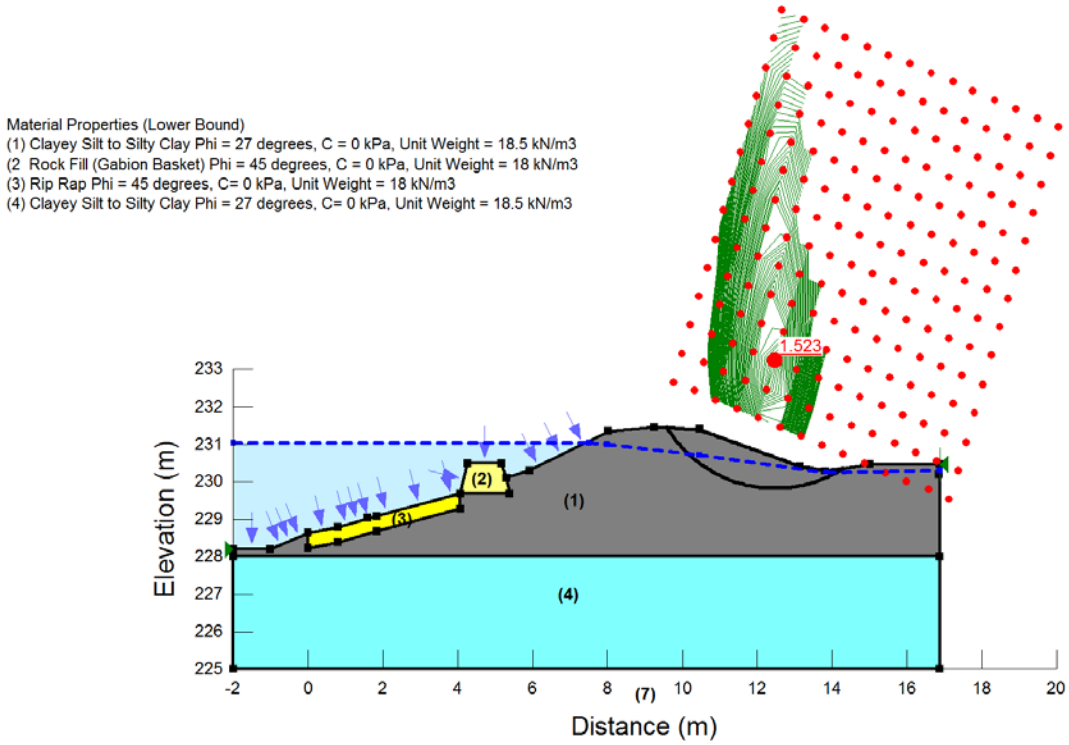


Case II: Extreme Short-term Earthquake Loading (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-6	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

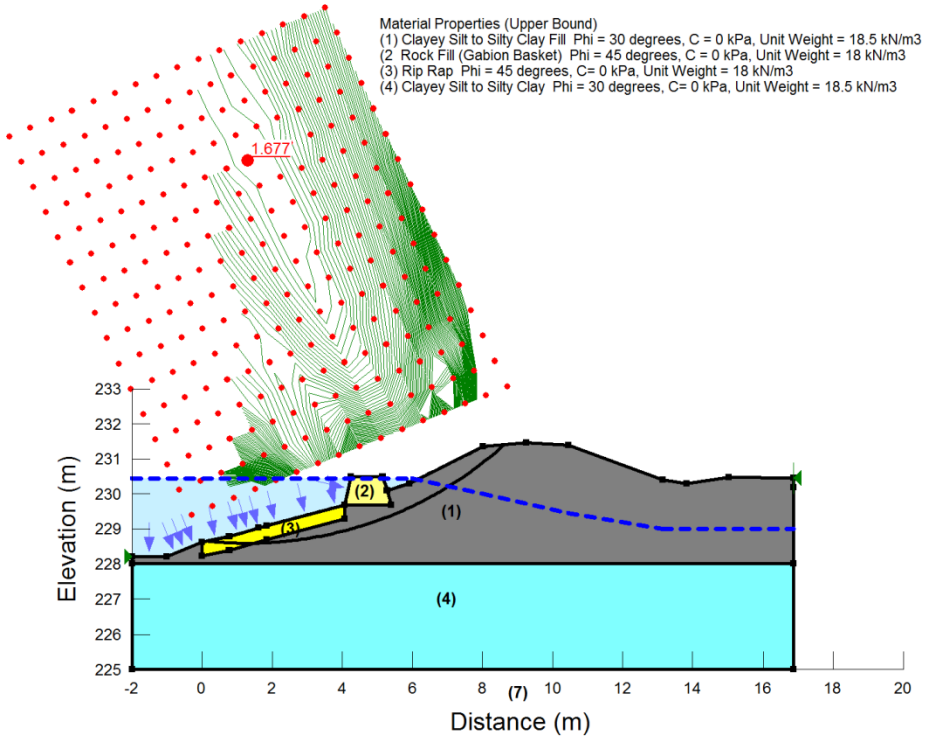


Case IV: Short term IDF condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-7	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

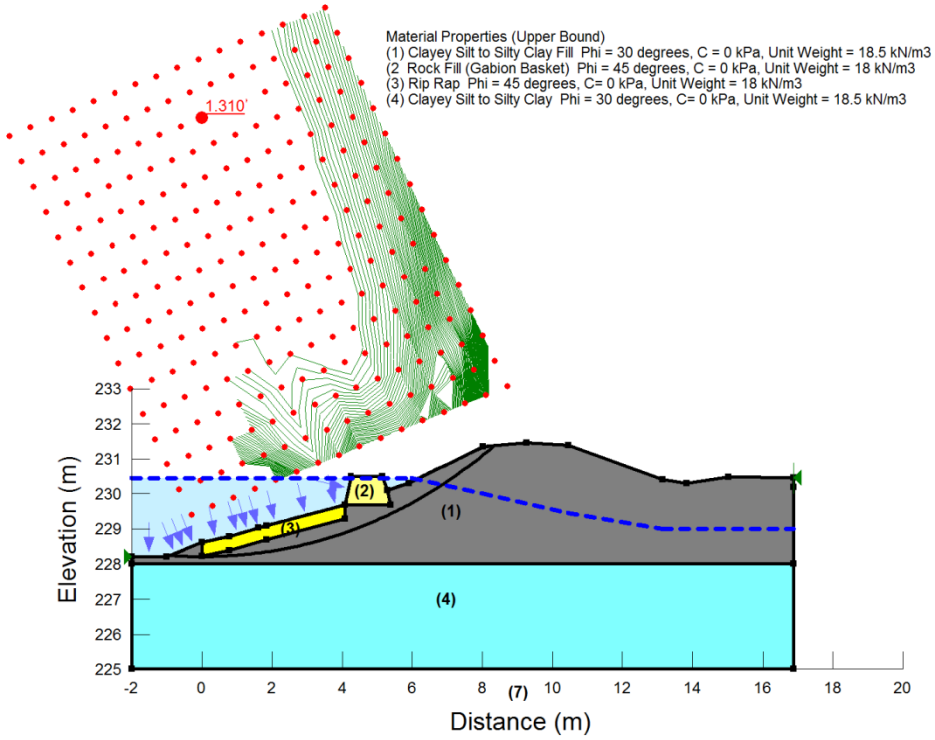


Case I: Long-term Normal Operating Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-1	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

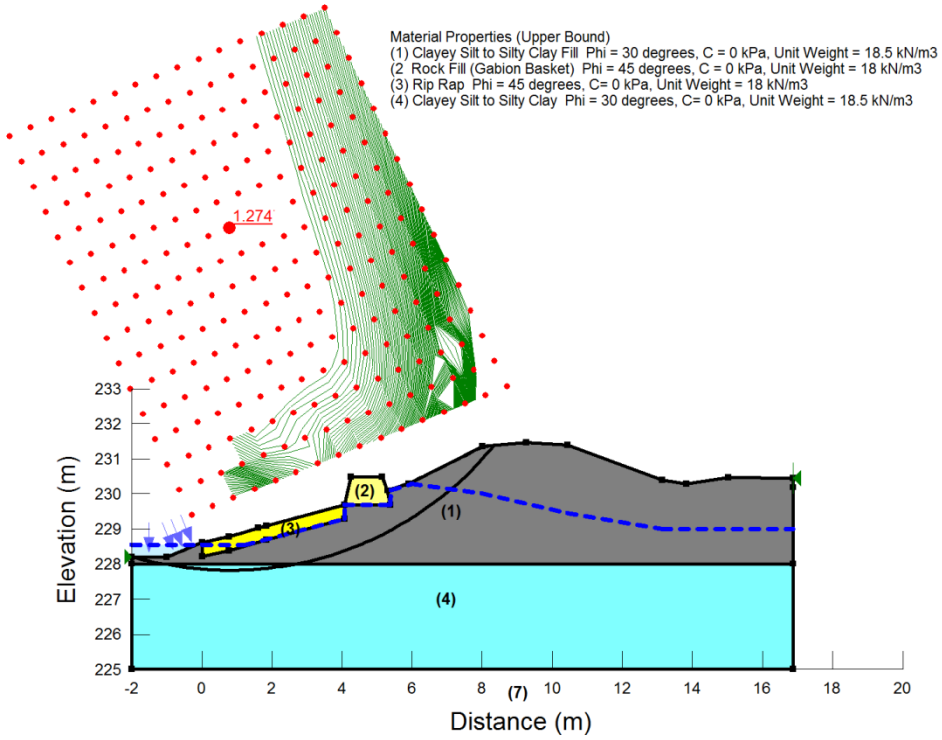


Case II: Extreme Short-term Earthquake Loading (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-2	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

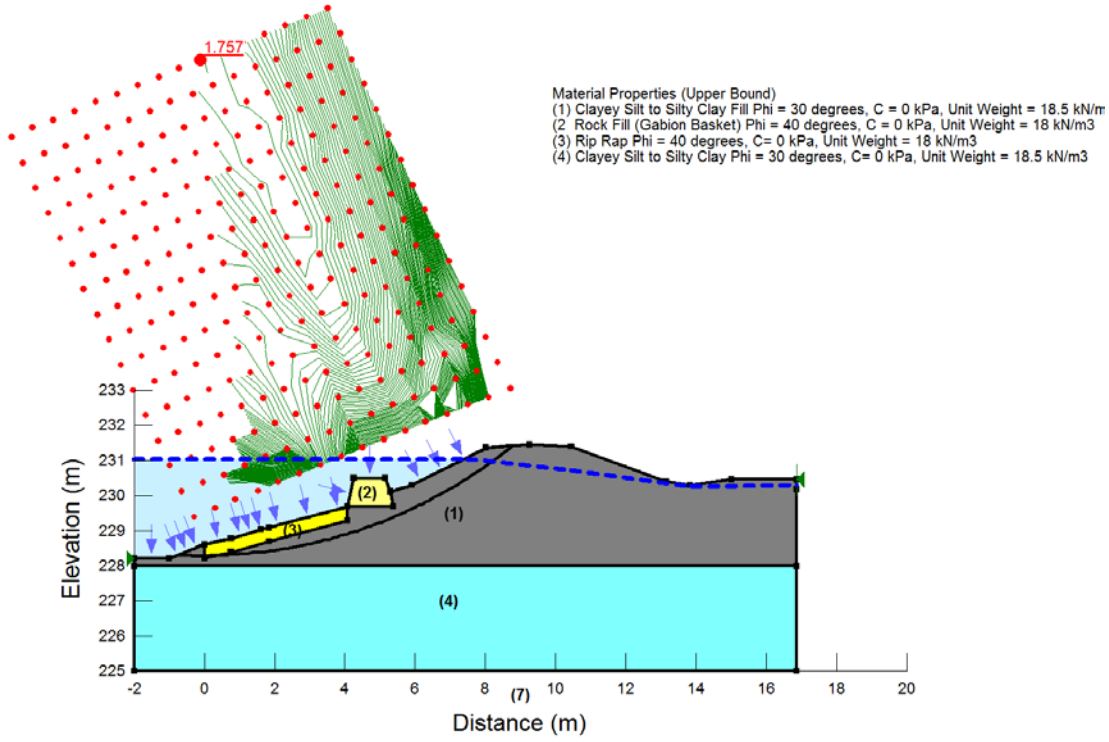


Case III: Short Rapid Drawdown Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-3	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

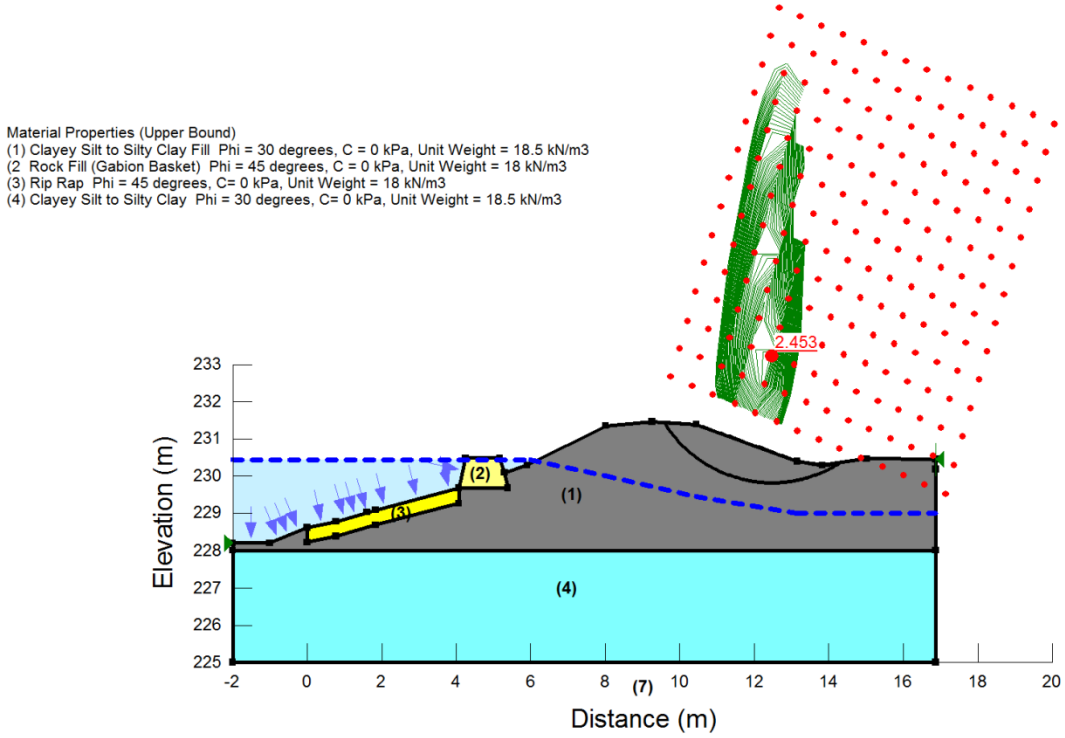


Case IV: Short term IDF condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-4	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

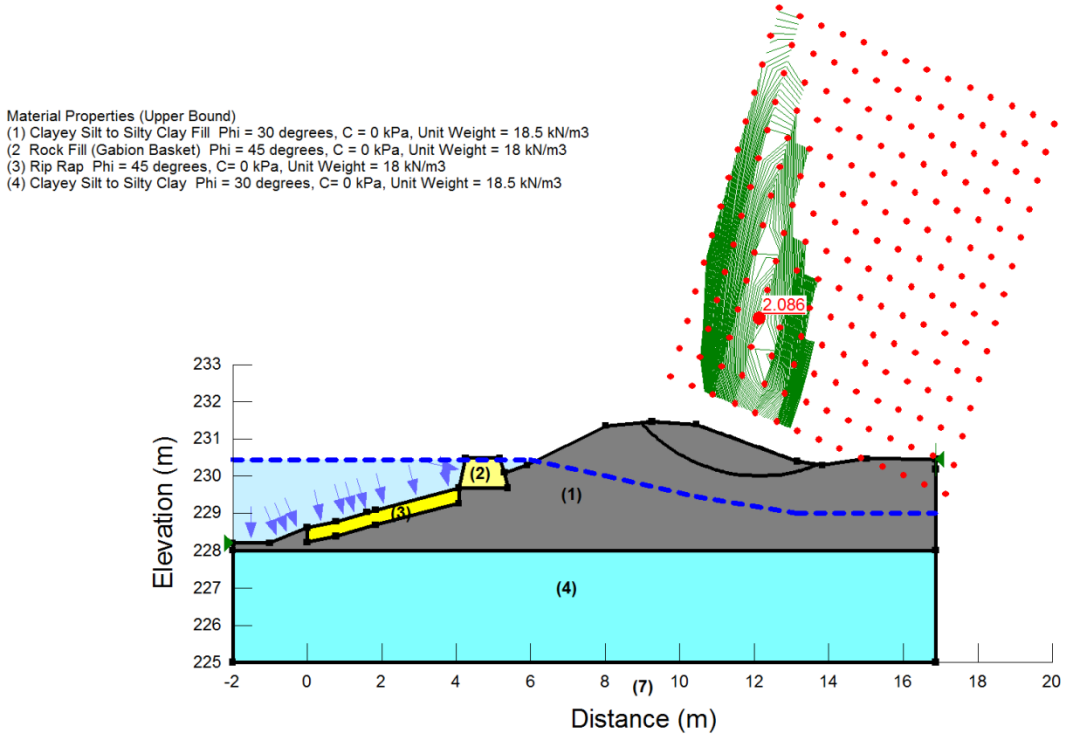


Case I: Long-term Normal Operating Condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-5	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

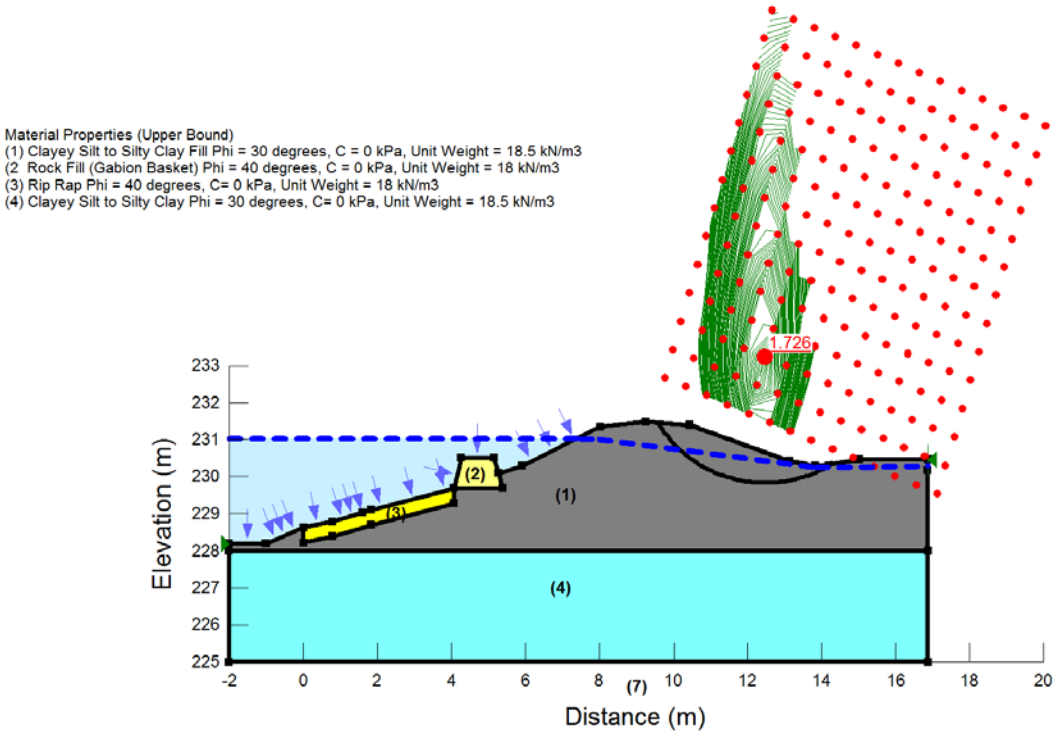


Case II: Extreme Short-term Earthquake Loading (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-6	

Talbot Dam and Lock 38 – Northwest Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

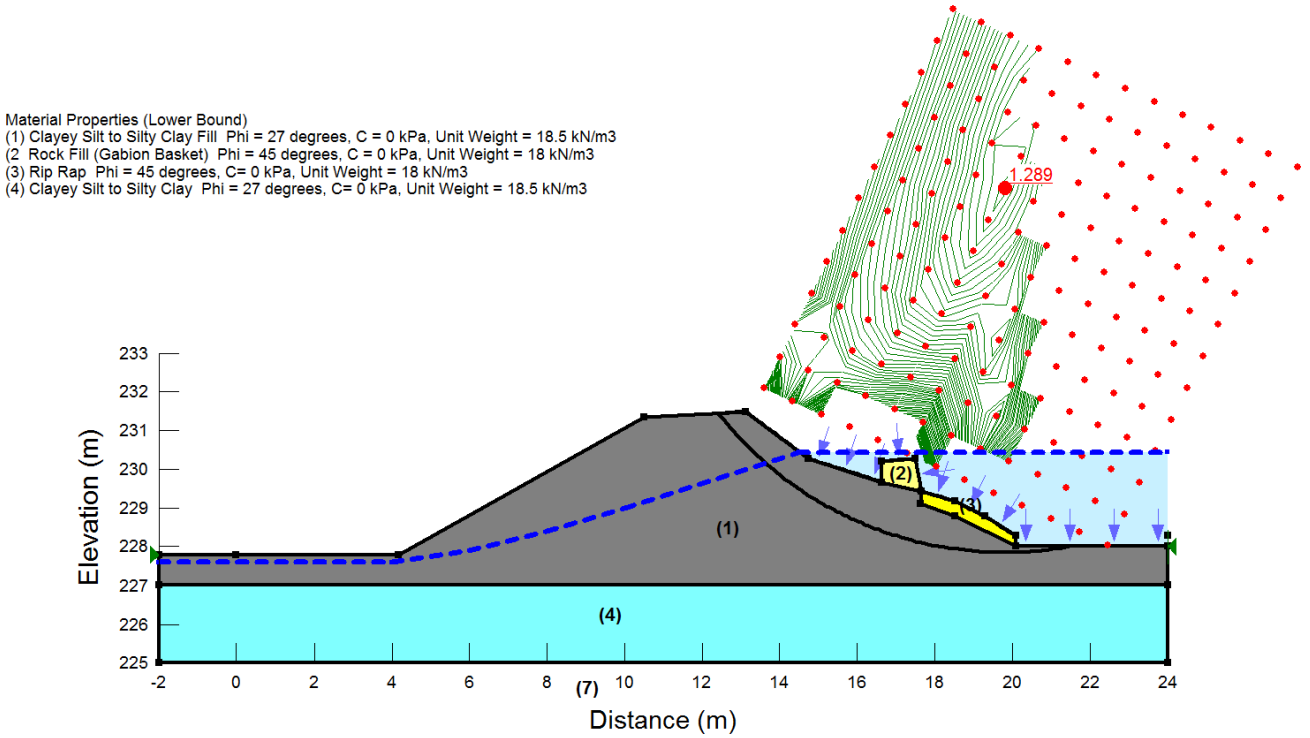


Case IV: Short term IDF condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
NORTHWEST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-7	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

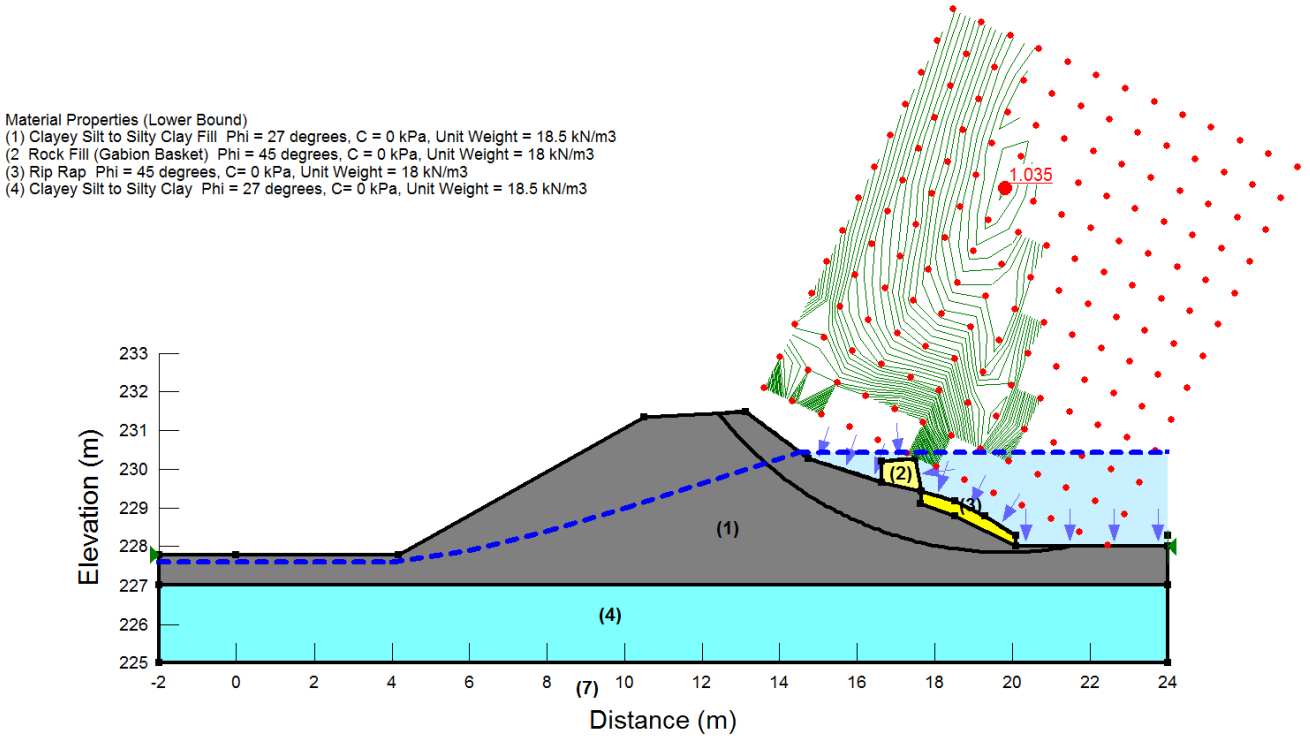


Case I: Long-term Normal Operating Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-1	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

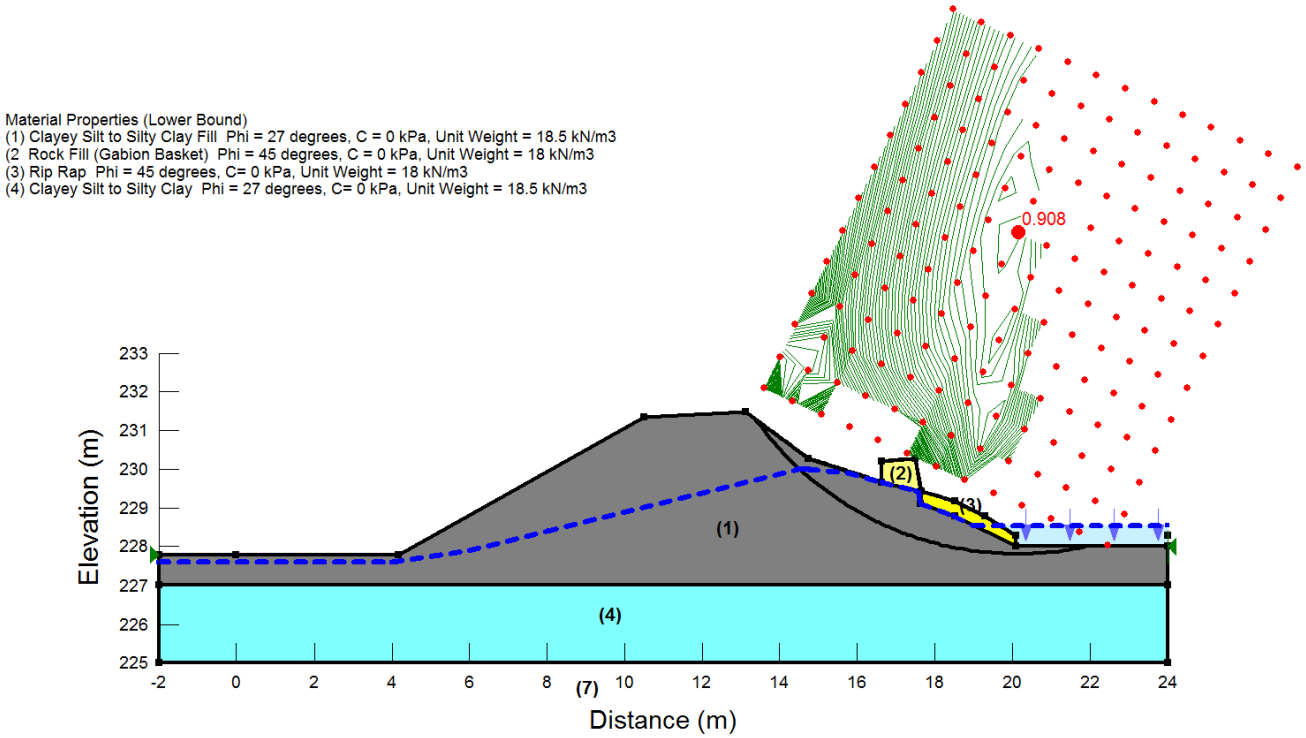


Case II: Extreme Short-term Earthquake Loading (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-2	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

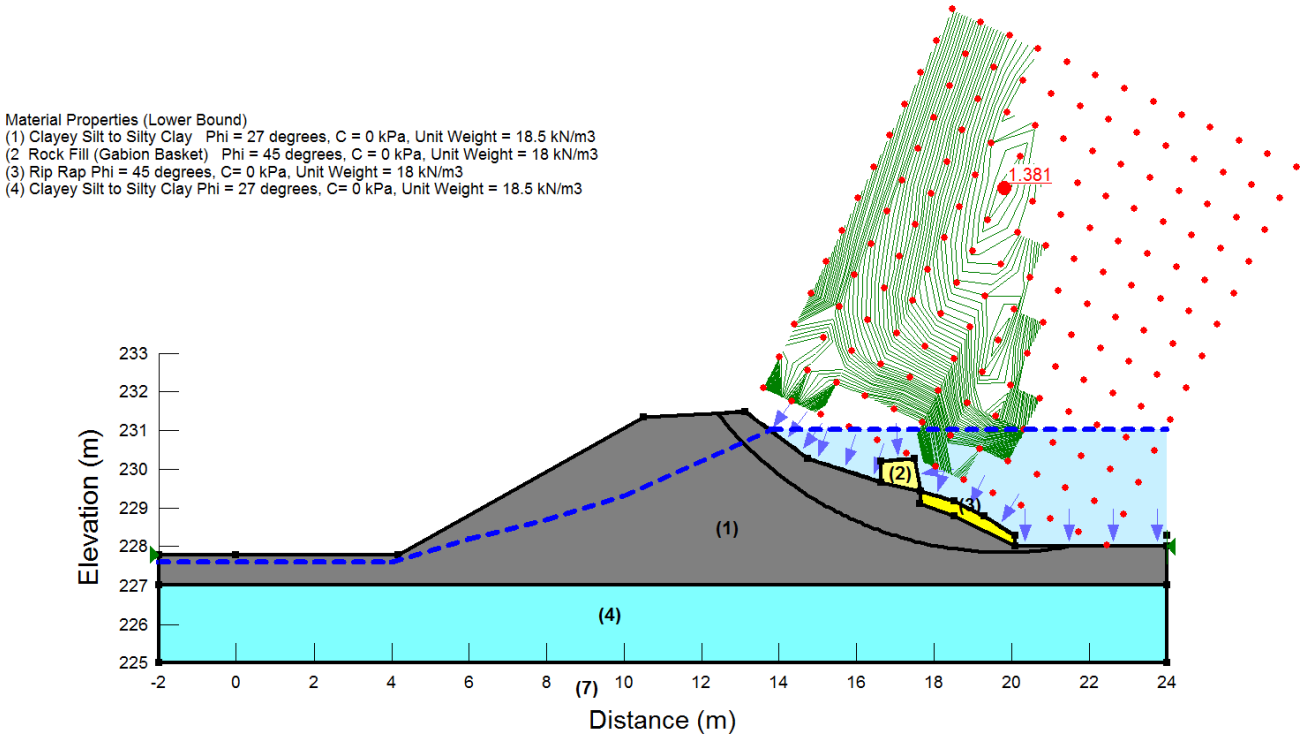


Case III: Short Rapid Drawdown Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-3	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

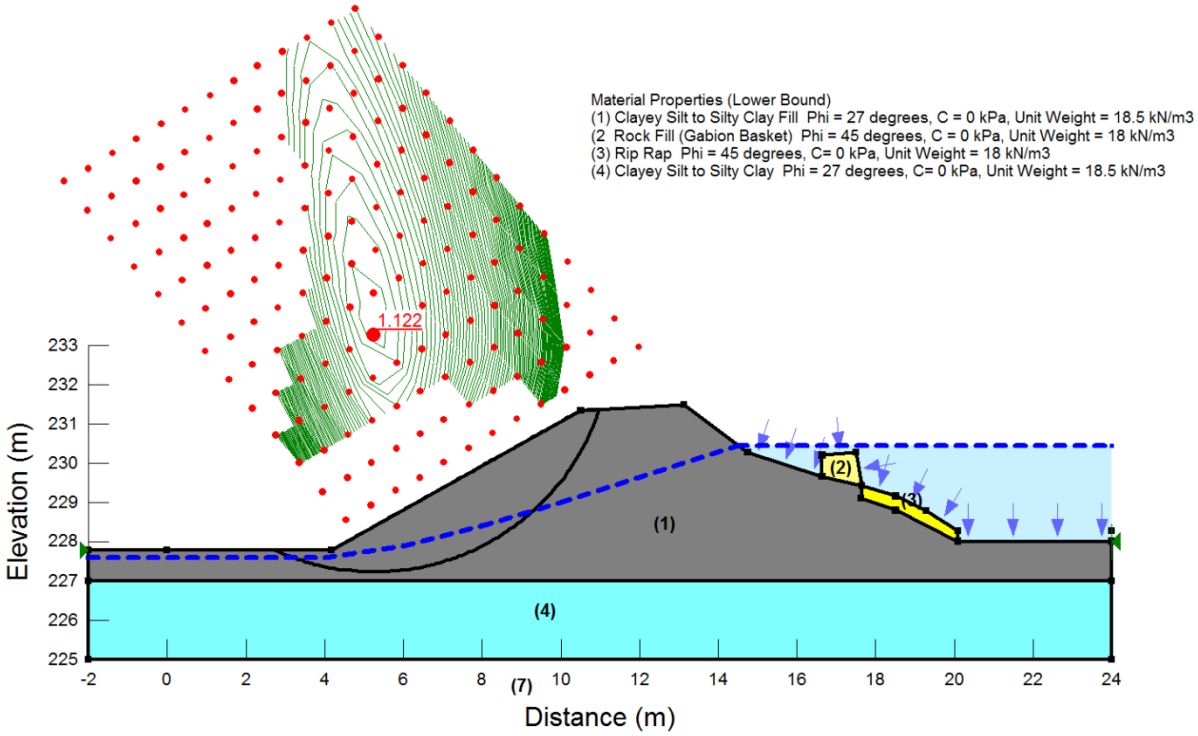


Case IV: Short term IDF condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-4	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

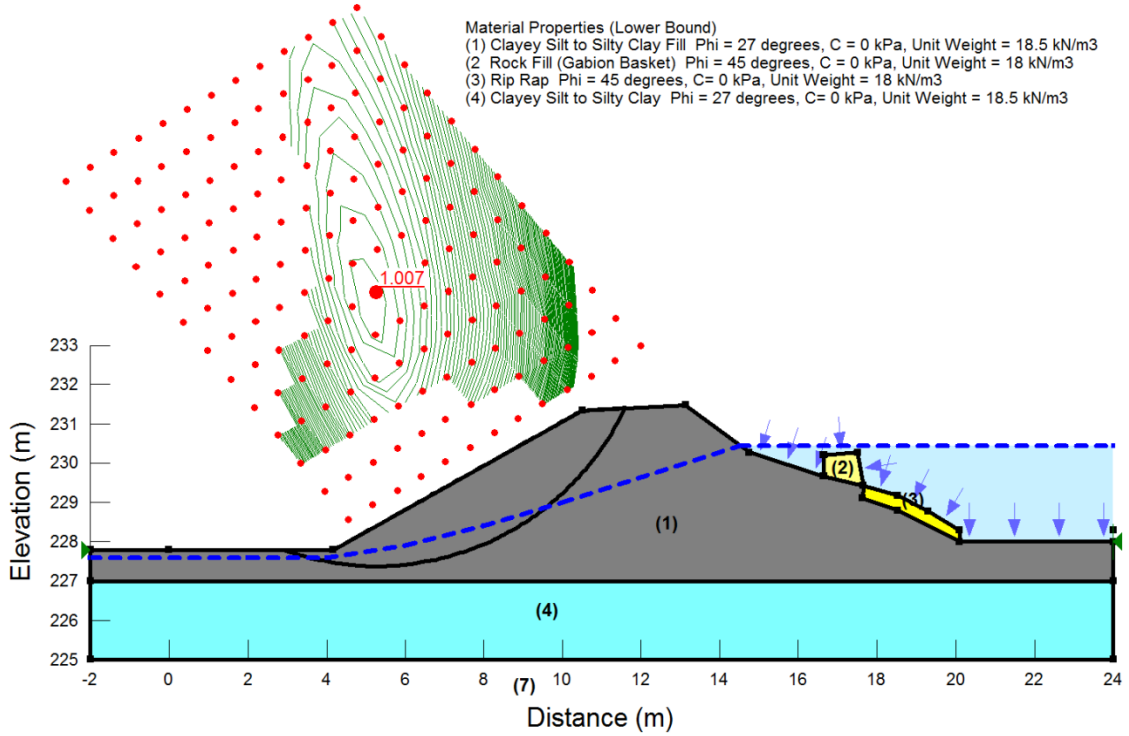


Case I: Long-term Normal Operating Condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-5	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

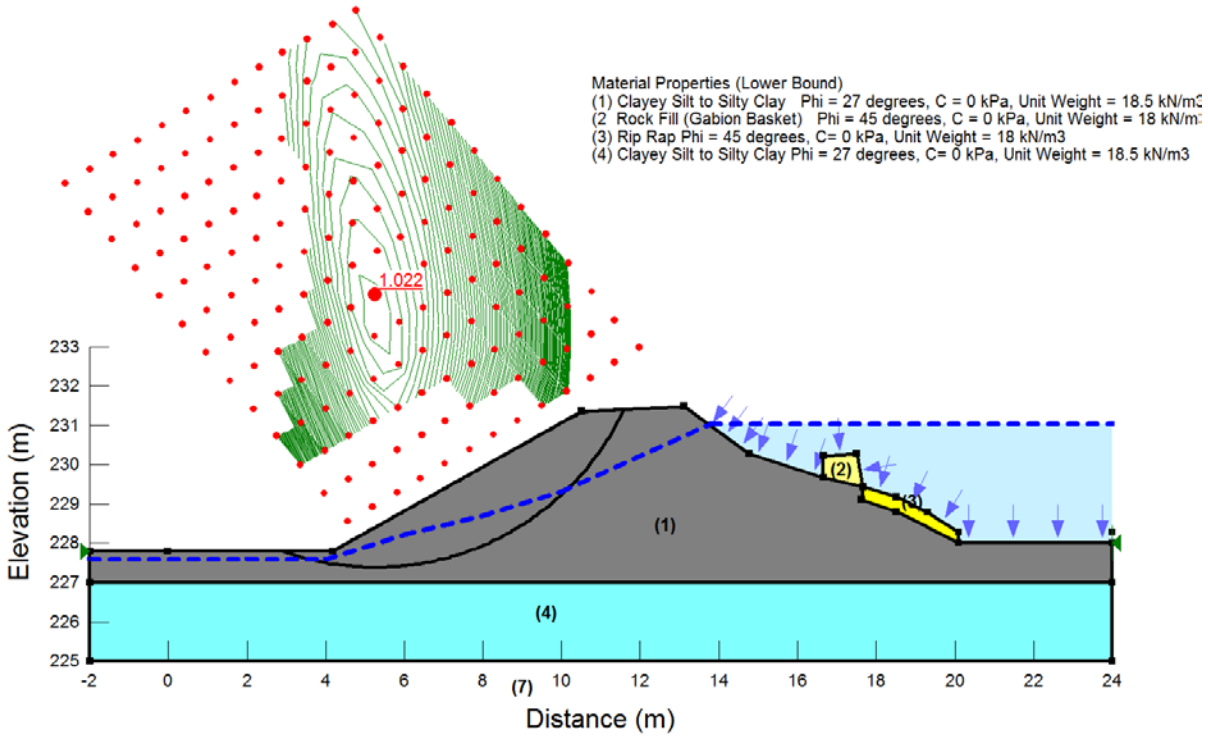


Case II: Extreme Short-term Earthquake Loading (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-6	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Lower Bound Material Properties

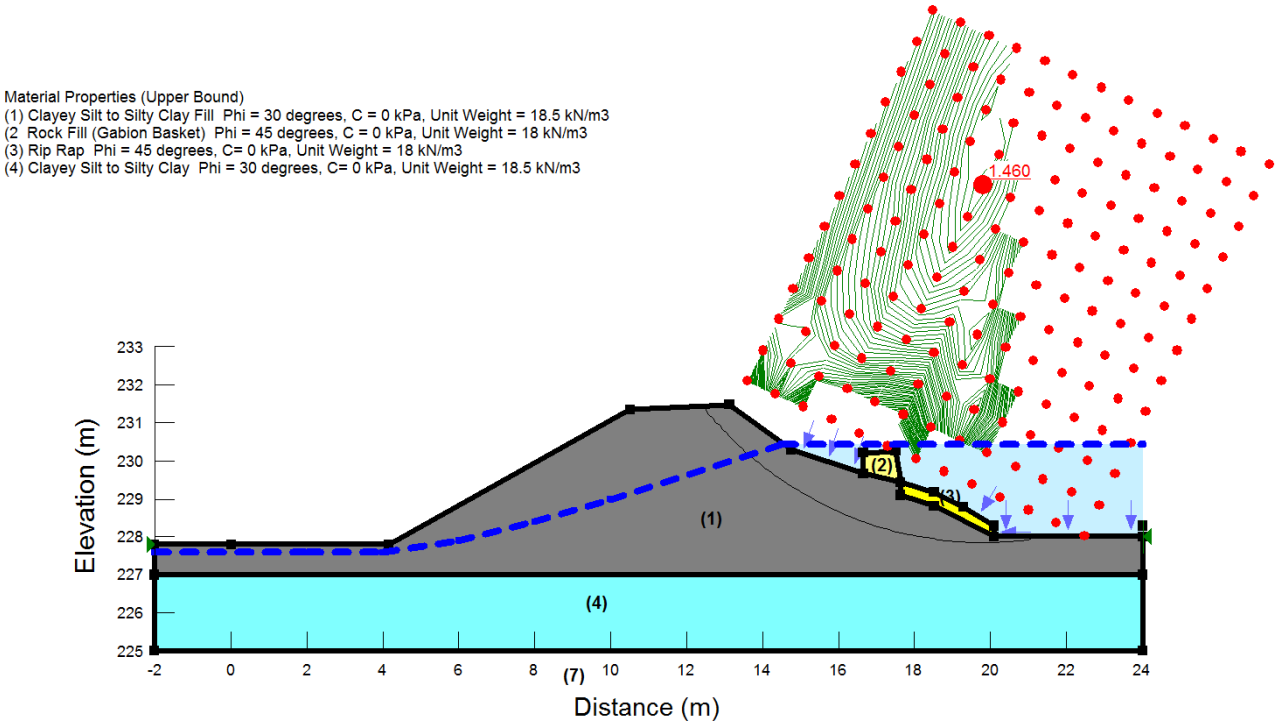


Case IV: Short term IDF condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-7	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

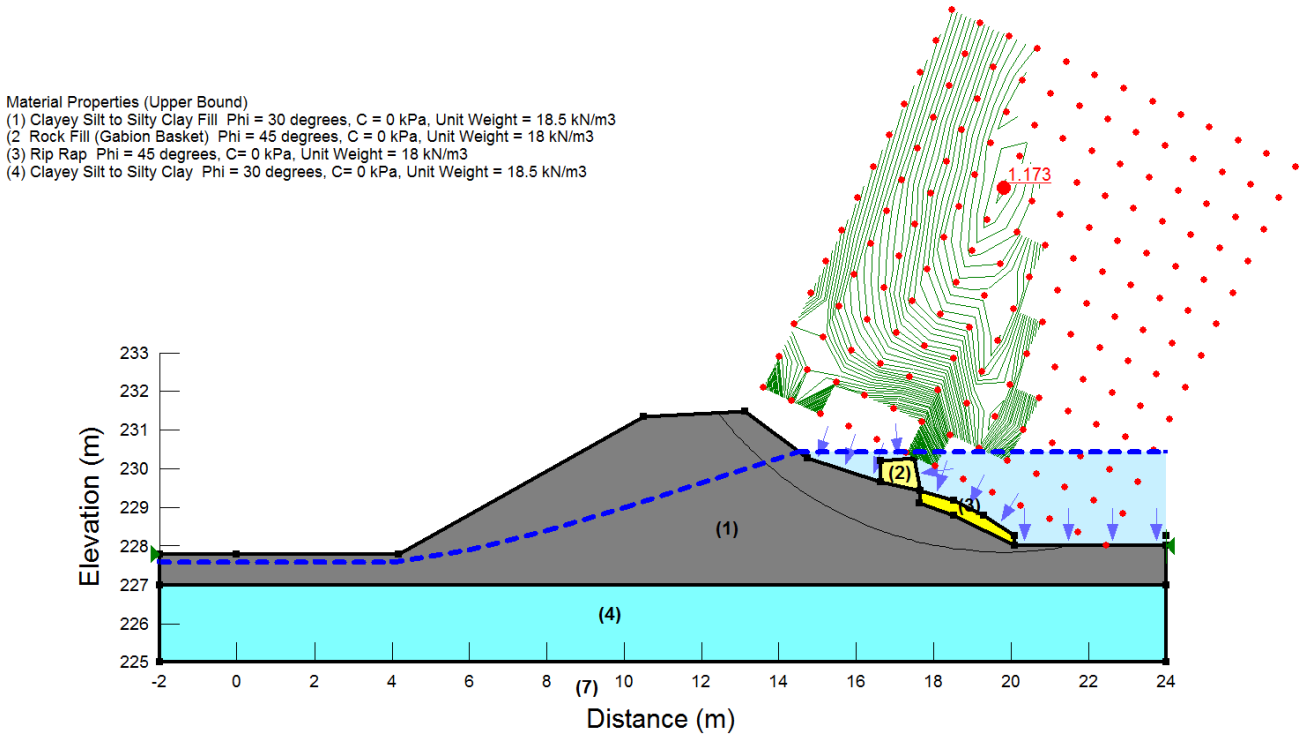


Case I: Long-term Normal Operating Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-1	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

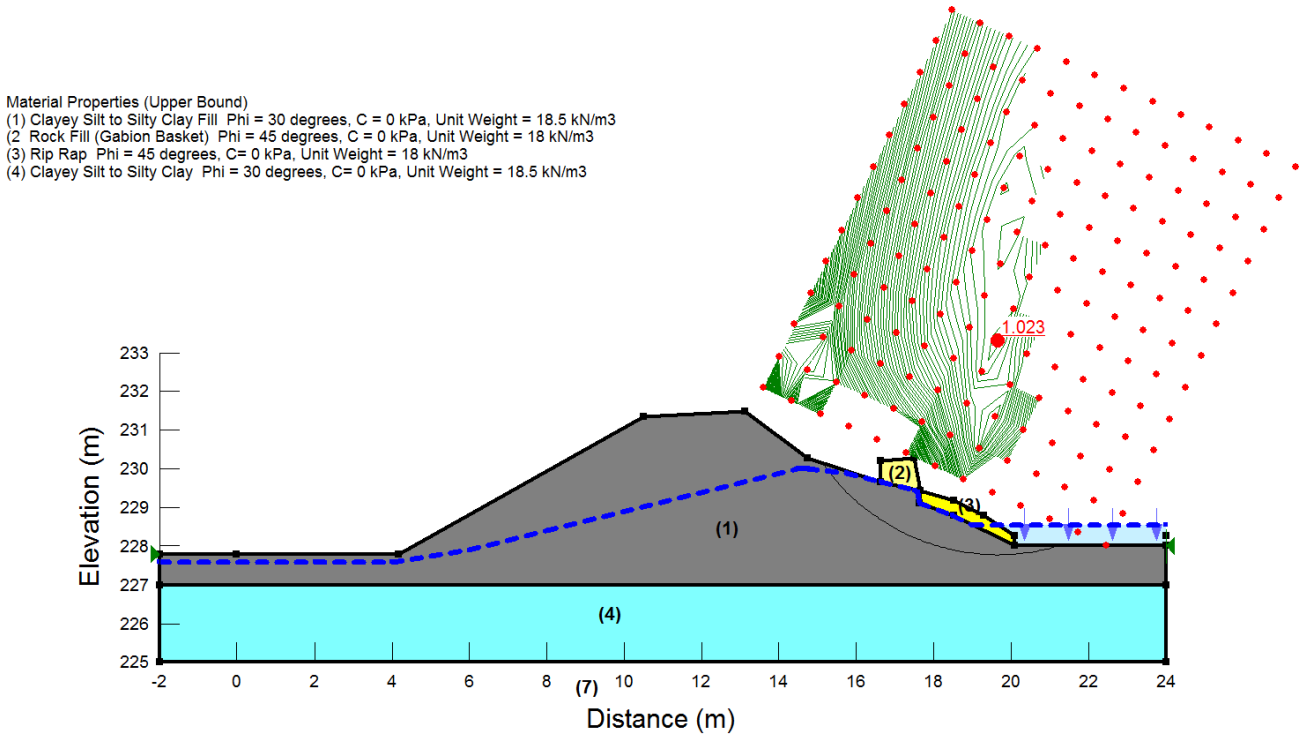


Case II: Extreme Short-term Earthquake Loading (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-2	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

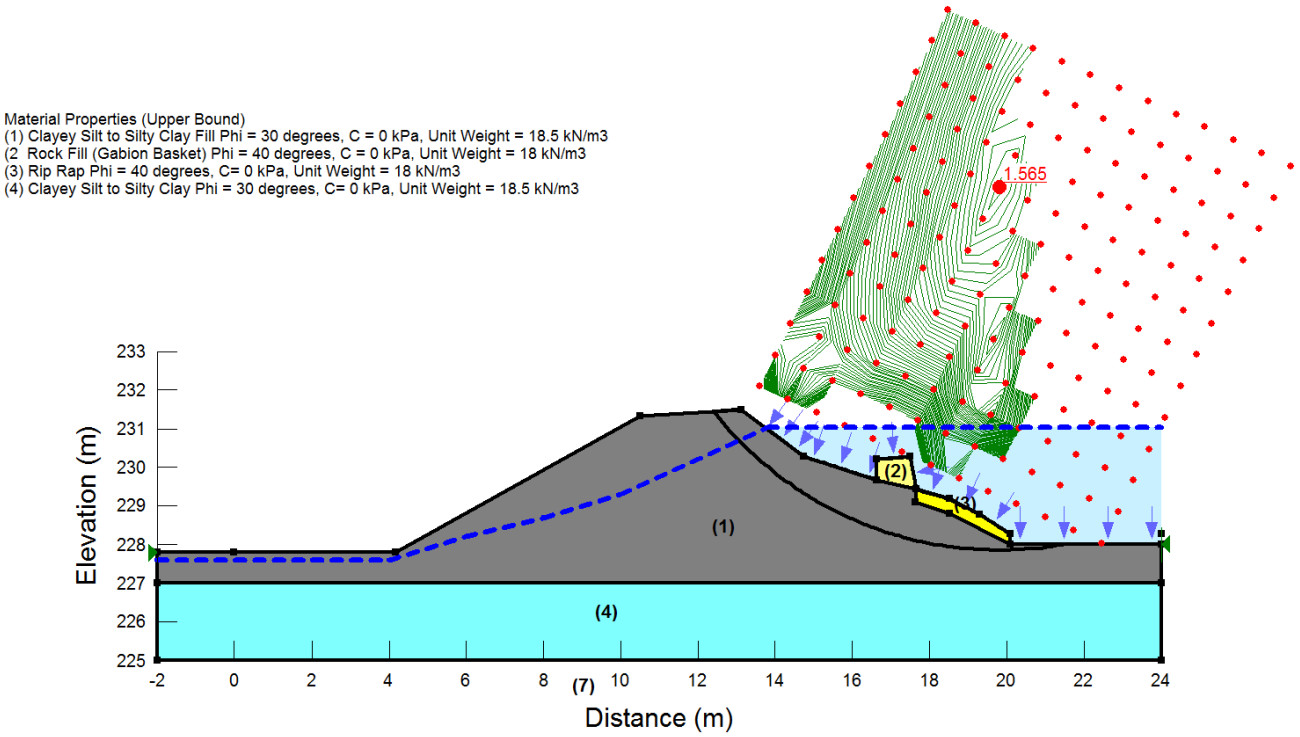


Case III: Short Rapid Drawdown Condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-3	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

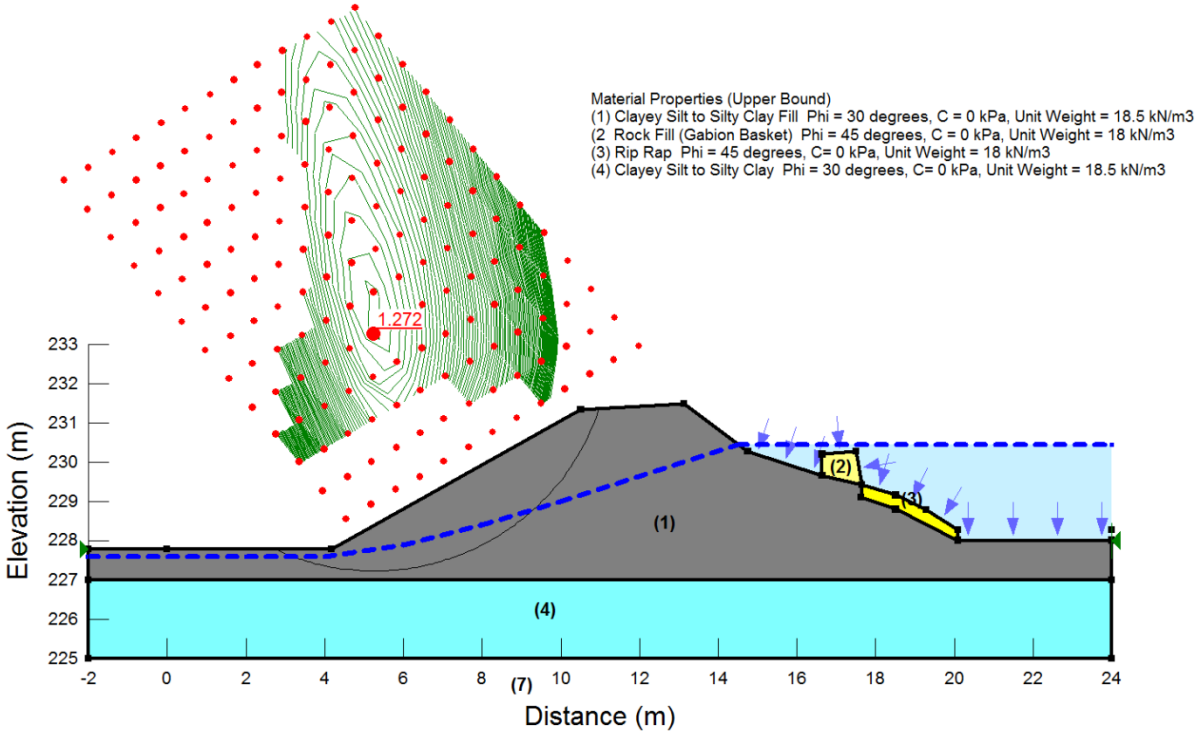


Case IV: Short term IDF condition (Upstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-4	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

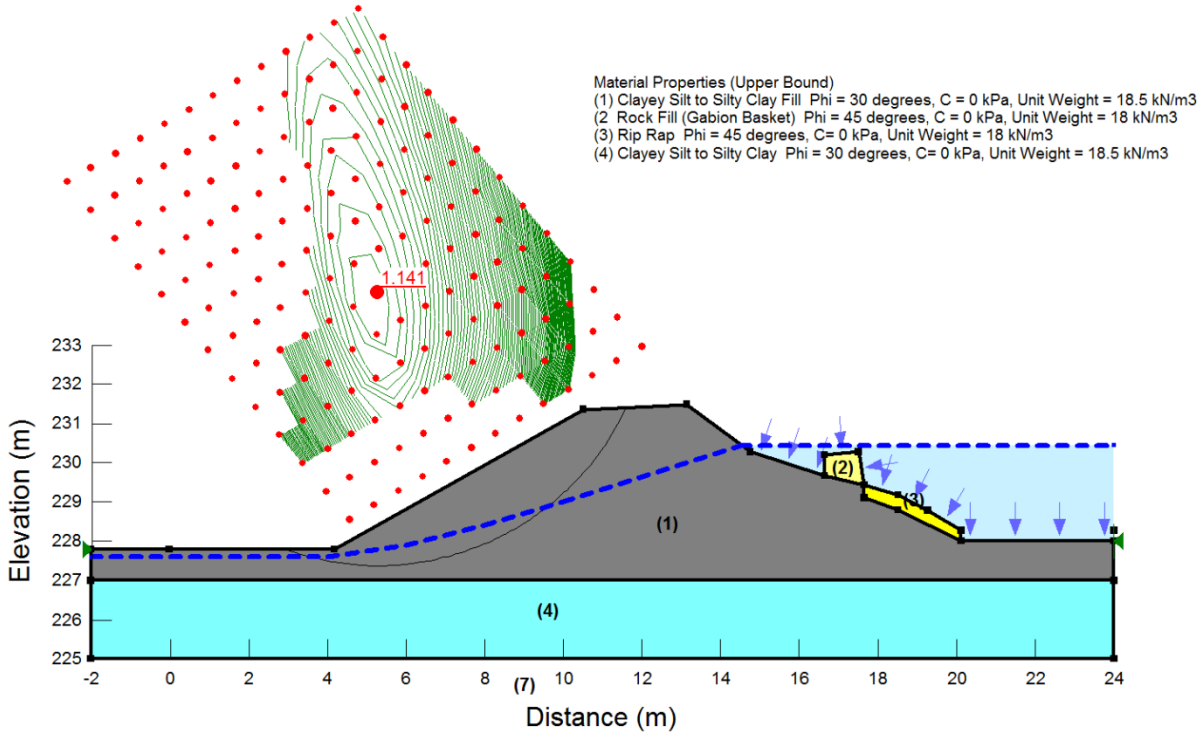


Case I: Long-term Normal Operating Condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-5	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties

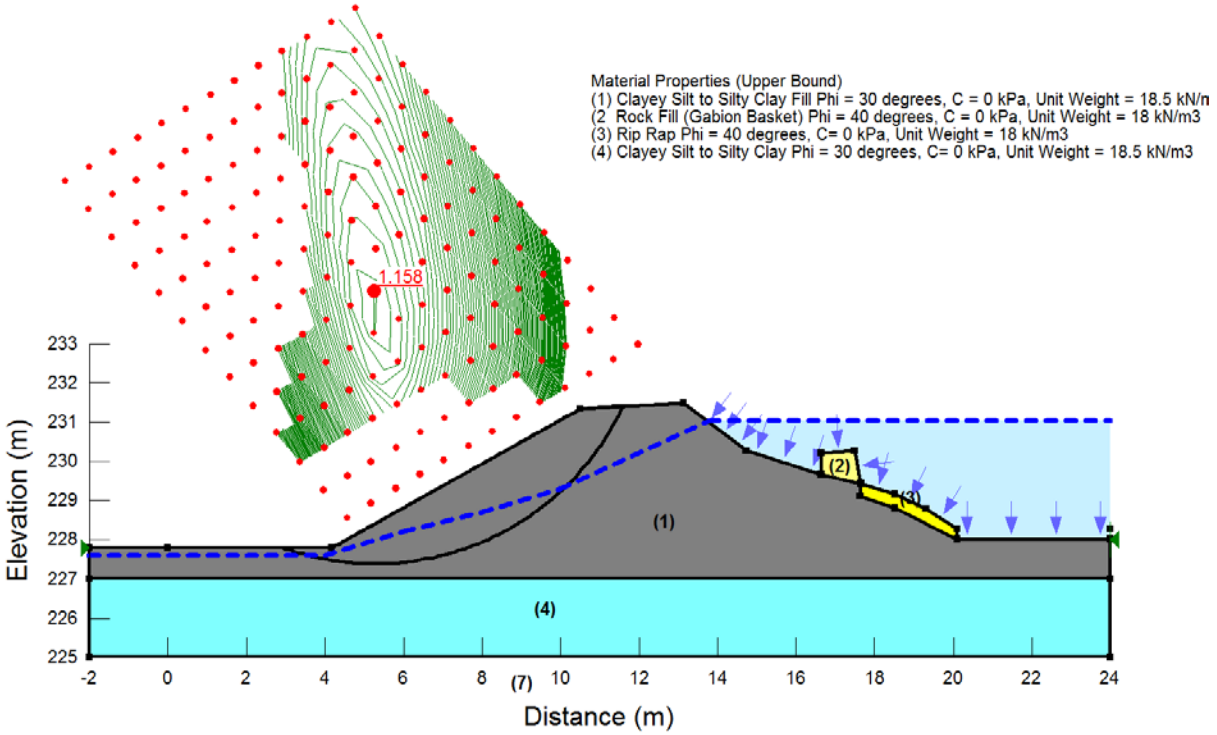


Case II: Extreme Short-term Earthquake Loading (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-6	

Talbot Dam and Lock 38 – Southeast Earthfill Embankment

Slope Stability Analyses- Upper Bound Material Properties



Case IV: Short term IDF condition (Downstream Slope)

KGS GROUP	Public Works and Government Services Canada / Travaux publics et Services gouvernementaux: Canada	
TALBOT DAM AND LOCK 38 STABILITY ANALYSIS		
SOUTHEAST EARTHFILL EMBANKMENT		
OCTOBER 2014	FIGURE G-7	