Chapter 10

Duffin Creek Water Pollution Control Plant New Outfall

The Regional Municipality of York
The Regional Municipality of Durham

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10.1 Overview

The Duffin Creek Water Pollution Control Plant (WPCP) outfall conveys treated effluent into Lake Ontario. The existing outfall was built in 1978 as part of the original plant construction and consists of a 1,100 m long, 3000 mm inner diameter pipe. The last 180 m of the outfall is equipped with 63 diffuser ports to disperse effluent into the lake. A schematic representation of the existing outfall is shown in Figure 10.1.

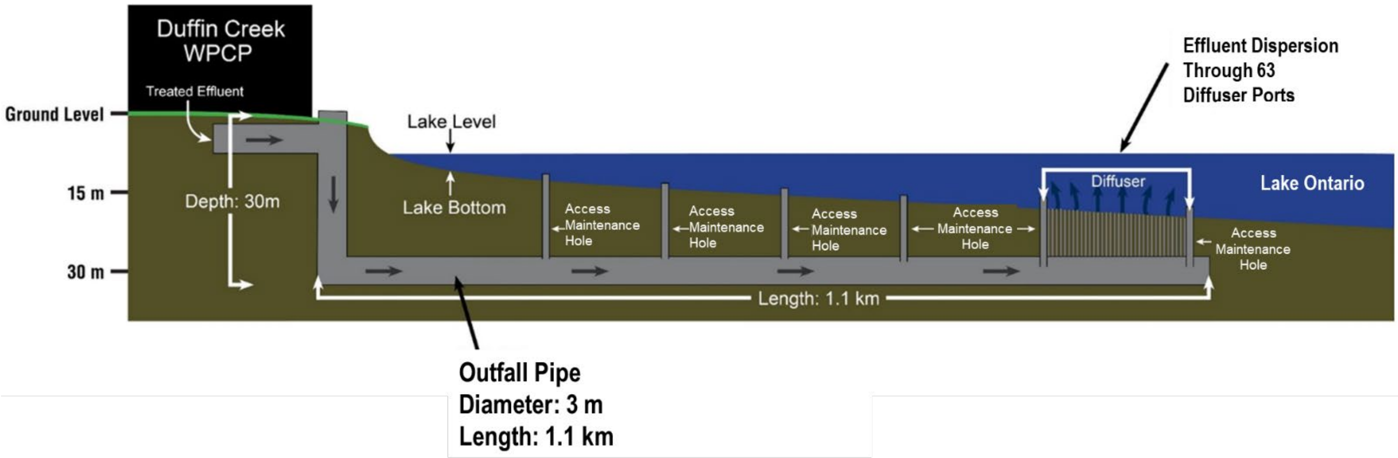

Figure 10.1 Existing Outfall Duffin Creek WPCP

The outfall is an operationally critical asset for the plant, as there is no emergency bypass or redundant outfall. To service the projected sewershed population, a new outfall will be required to handle the estimated wastewater flows. Chapter 10 provides a description of the conceptual design for the proposed outfall.

Chapter 10 includes the following sections:

– Study area
– Existing conditions, including social and built environment, natural environment, and cultural environment
– Receiving water impact assessment
– Outfall configuration
– Hydraulic design of the outfall
– Outfall conceptual design and construction considerations
– Environmental impacts and mitigation strategies.

10.1.1 Study Area

This section summarizes the study area boundary surrounding the existing and proposed outfall in Lake Ontario.

10.1.2 Existing Conditions

This component examines the existing environmental conditions of the project area and establishes a baseline against which the potential impacts are assessed. These different aspects are evaluated through various methods, including scientific studies, surveys and consultation with stakeholders, interested persons and Indigenous Communities. Factors such as air and water quality, land use patterns, wildlife populations, socio-economic conditions, and community resources are evaluated to understand the existing state of the environment as further described in sections 10.1.2.1 to 10.1.2.3.

10.1.2.1 Social and Built Environment

This aspect of the assessment considers the impacts on the social fabric of the community, including human health, quality of life, social well-being and community cohesion, as well as the existing built infrastructure and facilities in the project area. It evaluates factors such as noise, capacity constraints, and changes in land use patterns, recognizing the interplay between social and built elements in the project's environmental impact.

10.1.2.2 Natural Environment

The assessment focuses on the ecological components, such as flora, fauna, ecosystems, and natural resources. It evaluates potential impacts on biodiversity, habitats, water quality, air quality, soil quality, and the overall functioning of natural systems.

10.1.2.3 Cultural Environment

This aspect examines any archaeological resources that may be affected by the proposed project. It considers the potential impacts on cultural identity, traditional knowledge, and the cultural significance of the area.

10.1.3 Receiving Water Impact Assessment

This section describes the lake modelling approach for assessing impacts of the outfall on the natural environment with respect to environmentally sensitive receptors such as beaches, shorelines and water intakes.

10.1.4 Outfall Configuration

This section provides a description of the preliminary preferred concept for the Duffin Creek WPCP New Outfall based on the results from the receiving water impact assessment.

10.1.5 Hydraulic Design

The hydraulic design section provides an overview of the hydraulic criteria, methodology and results to demonstrate that adequate head is available for the outfall configuration presented in section 10.1.4.

10.1.6 Outfall Conceptual Design and Construction Considerations

This section provides a description of the potential construction methods and lining for the shaft and the tunnel.

10.1.7 Environmental Impacts and Mitigation Strategies

This section identifies potential environmental impacts and develops mitigation measures that will inform decision-making processes to promote sustainable development that minimizes negative environmental effects while maximizing positive outcomes.

10.1.8 Capital Cost Estimates and Implementation Plan

These sections discuss the capital cost estimate, future field investigations, and permits and approvals required to design and construct the Duffin Creek WPCP New Outfall project. These components will be further reviewed and refined during the preliminary design phase.

10.2 Study Area

The study area refers to the geographic location where there is potential for cumulative biophysical and socio-economic effects, including lands, communities, and portions of Lake Ontario around the Duffin Creek WPCP site that are considered relevant to assessing any direct and indirect effects of proposed infrastructure. The study area is shown in Figure 10.2.

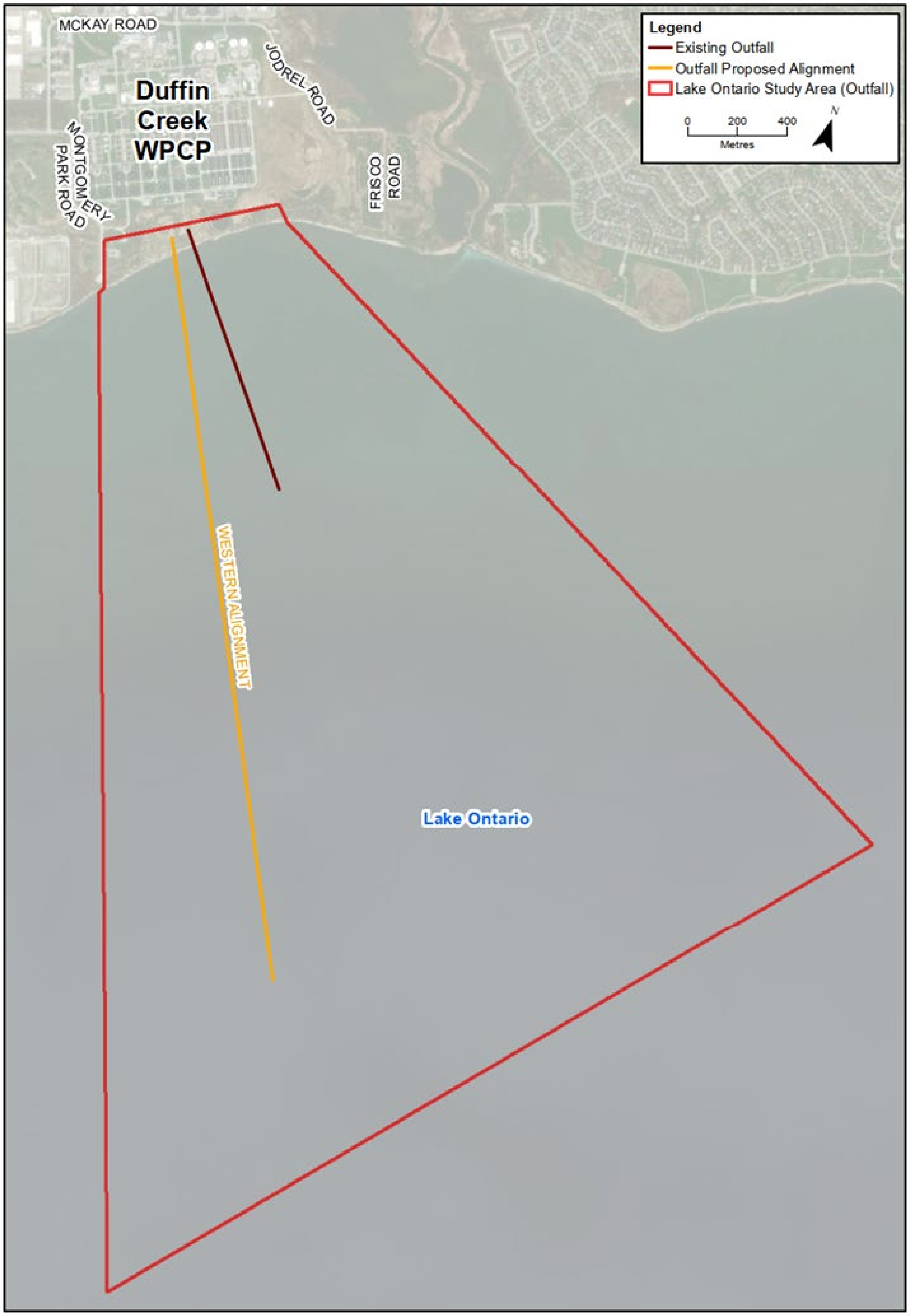

Figure 10.2 Duffin Creek WPCP New Outfall Study Area

10.3 2011 Outfall Environmental Assessment Overview

In December 2010, the Regional Municipality of York (York Region) and the Regional Municipality of Durham (Durham Region) jointly initiated a Schedule C Municipal Class Environmental Assessment (EA) to identify a preferred solution for addressing capacity limitations of the existing outfall at the Duffin Creek WPCP (referred to as the Outfall Class EA). The study was completed in November 2013 and is available at **https://apps.durham.ca/Applications/Works/PublicWorksProjects/studies/prj402/DuffinOutfallESR_MainBody.pdf**.

The Outfall Class EA investigated various alternatives to address outfall capacity limitations, including replacing the existing outfall with a new outfall, upgrading the existing outfall, or implementing tertiary treatment. The construction of a new outfall was identified as a potential solution for meeting future wastewater needs beyond the plant's current capacity of 630 ML/d. For the new outfall alternative, two alignments were evaluated: one that follows the existing outfall alignment (the eastern alignment) and a second (the western alignment) that extends west of the existing outfall farther away from the Ajax Water Supply Plant (WSP) intake. Preliminary CORMIX lake modelling, conducted in support of the Outfall Class EA, demonstrated that an outfall length between 2 and 3 kilometres (km) was required to meet the Ministry of the Environment, Conservation and Parks (MECP) mixing requirements and the Provincial Water Quality Objectives (PWQO) in the near-field region. Other parameters were evaluated, including impacts on the natural environment, geotechnical conditions, land uses, community health and safety, impacts on the Ajax WSP intake, and capital costs. The Outfall Class EA determined that if a new outfall were to be constructed at Duffin Creek WPCP, the preferred alignment would be the western alignment, primarily because of its orientation away from the Ajax WSP intake.

Ultimately, of the various alternatives evaluated in the Outfall Class EA, the recommended solution was to upgrade the existing outfall by installing variable-diameter duckbill check valves on all 63 diffuser ports (Figure 10.3). Upgrades to the outfall were successfully completed over two separate in-water construction periods sequenced in summer/fall 2021 and summer 2022.

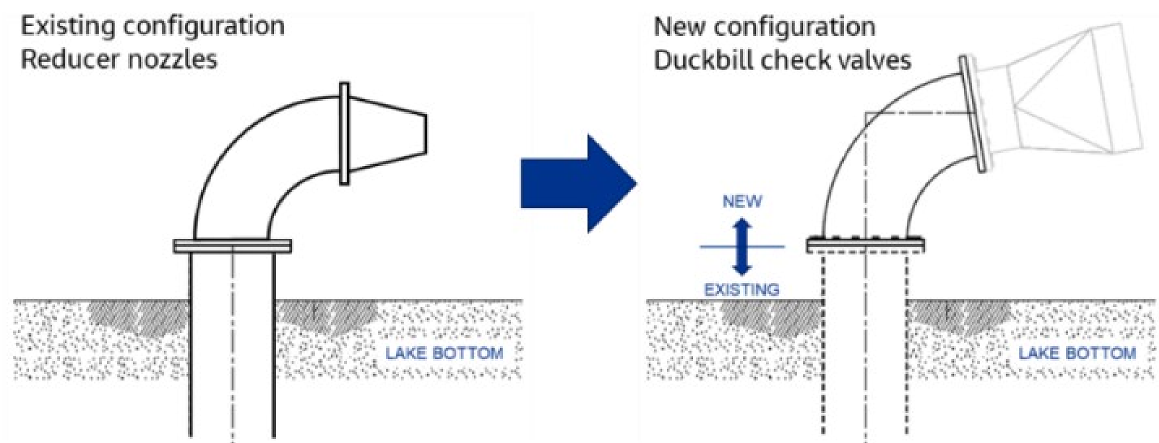

Figure 10.3 Existing and New Diffuser Port Configuration

With these recent upgrades, the upgraded outfall configuration has an instantaneous peak hydraulic capacity of approximately 1,900 ML/d, which is lower than the peak hydraulic capacity of the proposed influent trunk sewer (3,280 ML/d) and the total build-out hydraulic capacity of Stage 1&2 and Stage 3&4 influent sewage pumping stations (SPS) (3,290 ML/d); therefore, a new outfall will be required to handle the ultimate hydraulic capacity planned for the Duffin Creek WPCP.

Investigations and assessments completed in support of the 2011 Outfall Class EA were used as a starting point to develop the outfall design concept presented herein. The design concept is based on the preferred western alignment identified in the Outfall Class EA.

10.4 Existing Conditions

10.4.1 Social and Built Environment

10.4.1.1 Waterfront Trail and Public Parks

The Waterfront Trail is a prominent feature that links the Pickering and Ajax waterfronts. The Waterfront Trail is a multi-use (for example, pedestrian, cycling, and rollerblading) recreational trail that is also used for commuting purposes. In the vicinity of the Duffin Creek WPCP site, the trail can be accessed from Jodrel and Frisco Road in the east and from Montgomery Park Road in the west.

The Rotary Park Beach in Ajax, located to the east of Duffins Creek, is a major focal point on the trail with a long history of beachgoers. It offers a playground and splash pad, and it is also the venue for many Town of Ajax events and charity walks. Visitors also enjoy Canada Day festivities and fireworks here. The park also houses a 3,800-square-foot multi-purpose pavilion, which can be rented for meetings and events.

The Ajax Waterfront Park Beach comprises the longest stretch of undeveloped waterfront in the Greater Toronto Area and provides walking and cycling trails. It connects Paradise Park at Pickering Beach to Rotary Park. Paradise Park is a popular recreational node that incorporates beaches, sports fields, tennis courts, and playgrounds. Recreational swimming is also permitted at Pickering Beach.

Carruthers Marsh is a waterfront marsh protected by the Toronto and Region Conservation Authority (TRCA). It is also the location of a 3,500-square-foot lakefront facility that is available for hosting events. Carruthers Marsh Pavilion is a feature in the Town of Ajax's Waterfront Management Plan.

Beachfront Park rests on the northern shore of Lake Ontario and separates Hydro Marsh to the north from Lake Ontario to the south. The marsh provides great bird watching and fishing opportunities, with common sightings of herons, terns, and shorebirds.

Millennium Square is a large public square adjacent to the Beachfront Park. The Frenchman's Bay Yacht Club is a community yacht club for sail and power boats located west of the Duffin Creek WPCP in Frenchman's Bay in Pickering. Swan's Marina is also located in Frenchman's Bay and offers rental slips, boat sales, and indoor and outdoor storage. Boats from the marina and yacht club use the regional study area for recreational purposes.

10.4.1.2 Aesthetic Conditions along the Shoreline

Since the early 1990s, a resurgence of algal blooms has occurred along the coast of Lake Ontario and other Great Lakes, predominantly Cladophora. Cladophora is a branching, green, filamentous alga found naturally along the coastline of most of the Great Lakes. Cladophora plants attach to the lakebed, forming lush lawns of filaments that are periodically detached by waves and washed ashore as mats. When these mats decay, they result in foul-smelling beaches, which can cause lowered beach use and odours that affect local residents. These types of episodes are transitory and have been observed to occur in both urban and non-urban areas around the Great Lakes.

10.4.1.3 Pickering Nuclear Generation Station

The Pickering Nuclear Generating Station (PNGS) is one of the largest nuclear facilities in the world, consisting of Pickering A and Pickering B Nuclear Generating Stations. PNGS requires a significant volume of water intake from Lake Ontario for use during plant processes. The amount of water used in the PNGS process is highly variable and tends to be an order of magnitude higher than the current rated capacity of the Duffin Creek WPCP. The intake is located approximately 2.5 km west of the Duffin Creek WPCP Outfall near the shoreline.

10.4.1.4 Ajax Water Supply Plant Intake

The Ajax Water Supply Plant (WSP) produces drinking water for approximately 180,000 residents in the Town of Ajax and the City of Pickering. Its intake is located approximately 5 km east of the Duffin Creek WPCP Outfall. The Ajax WSP resides within the Toronto and Region Source Protection Area. The Credit Valley – Toronto and Region – Central Lake Ontario (CTC) Source Protection Committee has issued a Source Protection Plan to eliminate, reduce, or manage threats to drinking water resources by delineating vulnerable areas (Intake Protection Zones [IPZ]) and applying vulnerability scores. The IPZ is the area around a surface water intake that is defined to protect the source water for a municipal residential drinking water system. IPZ-1 has a radius of 1 km around the intake, whereas IPZ-2 represents the area both on land and in water and spans from the PNGS in the west, past Lakeridge Road in Whitby to the east, to Highway 401 along Duffins and Carruthers Creeks to the north and varying from 1 to 4 km into Lake Ontario to the south. As illustrated in Figure 10.4, IPZ-1 is assigned a vulnerability score of 5, which is considered moderate, and IPZ-2 has a vulnerability score of 4.5, which is considered moderate.

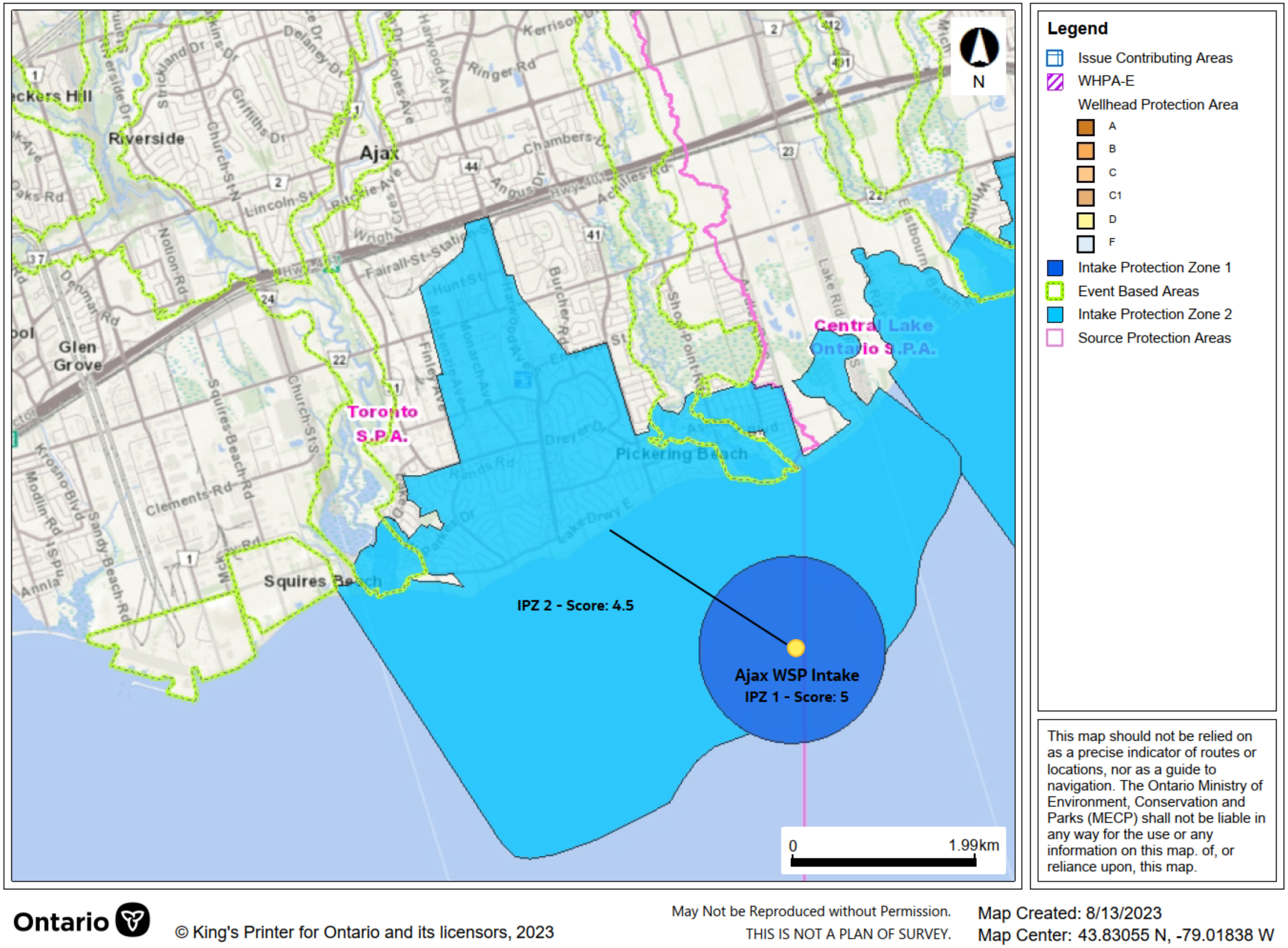

Figure 10.4 Ajax WSP Plant Intake Protection Zones

10.4.2 Natural Environment

The Duffin Creek WPCP New Outfall alignment was assessed to determine the existing conditions and the impact to the natural environment. This assessment included a desktop geotechnical and hydrogeological review and a natural heritage characterization.

10.4.2.1 Geotechnical Desktop Review

A geotechnical desktop review was undertaken by using available background information and researching existing subsurface information on publicly available platforms and databases.

Work was performed by Peto MacCallum Ltd. in 1974 prior to construction of the existing outfall. This work was later supplemented with additional exploratory drilling undertaken by Coffey Geotechnical Inc. between July 7 and August 25, 2010, and between July 27 and August 23, 2011. The boreholes were mainly drilled along the previously proposed eastern alignment for the new outfall roughly parallel to the existing outfall, about 200 to 300 m east of it.

In addition, two marine geophysical surveys consisting of bathymetry and sub-bottom profiling were conducted by ASI Group in 2009 and 2010. The 2010/2011 borehole drilling program completed by Coffey Geotechnics Inc. consisted of extending 11 boreholes to depths ranging between 34 m and 49 m below the lakebed. The 1972/1974 Peto MacCallum work consisted of 34 boreholes.

The rock surface generally slopes from the northwest to the southeast, and the rock is deeply cut with depressions and buried valleys carved out by glaciers. The investigation revealed the presence of buried valleys, which will act as geotechnical constraints for positioning the new outfall and govern the overall vertical tunnel alignment (Coffey Geotechnics Inc. 2012).

These geotechnical investigations also identified two rock formations: (1) the Upper Georgian Bay-Blue Mountain Shale and (2) the Lower and older Cobourg Limestone Formation. Bedrock surface was encountered between elevations 58.6 m and 40.1 m above sea level (masl), according to the information from the Coffey Geotechnics Inc. investigation (2012), and between elevations 46.5 and 67.6 masl, according to the Peto MacCallum (1974a) investigations. Pockets of gas were found in the shale in the past during construction of the existing outfall and during the geotechnical investigation.

According to the available borehole information, the outfall tunnel will advance through light-grey limestone and siltstone with interbedded shale bedrock of the Cobourg Formation for about 2,000 m, after which the bedrock is expected to transition to the dark grey to black shale with interbedded limestone and siltstone bedrock of the Georgian Bay-Blue Mountain (formerly Whitby) Formation. The previously completed investigation by Coffey Geotechnical Inc. (2012) included rock coring to elevations of 16 to 21 masl. The rock cores were photographed and logged, and index properties were recorded, which included the total core recovery, solid core recovery, rock quality designation, fracture index, percent of hard layers and the location and thickness of all hard layers. In addition to the index properties, laboratory tests were completed on recovered rock samples, which included point load index tests, uniaxial compressive strength tests, and hardness tests. The uniaxial compressive strength tests provided the Young’s modulus and Poisson’s ratio parameters for the rock.

The geotechnical and geophysical surveys from 2010 and 2011 found that the bedrock surface was overlain by overburdened soil deposits along the eastern alignment. Bedrock, belonging to Georgian Bay-Blue Mountain and Cobourg Formations, was encountered at elevations between 58.6 and 40.1 masl.

The Georgian Bay-Blue Mountain Formation is typically weak to medium in strength, brownish grey to black in colour, and brittle and moderately fissile, and it has a fine- to very fine-grained texture. This rock formation is approximately 70% to 90% shale interbedded with limestone and is frequently bituminous, containing organic gases. The Cobourg Formation is typically fine-grained, fossiliferous, and massively bedded with thin shale interbeds, and it contains pockets of gas.

The Georgian Bay-Blue Mountain Formation, which occurs at a higher elevation than the Cobourg Formation, is quite weathered (fractured) at the transition zone between soil and rock. According to the information gathered from the preliminary geotechnical investigations, which relied primarily on data from boreholes along the eastern alignment, the rock conditions at the likely outfall tunnel elevation are expected to be fair to good.

The thickness of the overburden that overlies the bedrock ranged from 0 to 16.4 m, with the thickest deposits found in the areas of the two buried valleys. The composition of the overburden material varied from very loose or soft organic silts or clay to very dense glacial tills, with the weakest or organic soil found in the buried rock valleys (Coffey Geotechnics Inc. 2012).

Previous investigations by Peto MacCallum Ltd. (1974) and Coffey Geotechnics Inc. (2010 and 2011) also identified two buried valleys (possibly three in the western alignment) crossing the eastern and western alignments, which will act as geotechnical constraints for positioning the new outfall and govern the overall vertical tunnel alignment (Coffey Geotechnics Inc. 2012). In addition, sediments in these areas will be studied to evaluate potential unstable conditions for construction or other risks to the structures.

10.4.2.2 Hydrogeology Desktop Review

A hydrogeological desktop review was undertaken using water wells records from the MECP Oak Ridges Moraine Groundwater Program (ORMGP), Ontario Geological Survey Geotechnical Borehole Database, and historical reports. Data were analyzed within 500 m of the outfall and drop shaft work area.

The available borehole logs, monitoring well logs, and water well information system (WWIS) records report the following:

- Topsoil has a thickness of 0.3 m.
- A clay layer was observed from the surface or below the topsoil, up to 12.6 metres below ground surface (mbgs). Cobbles were occasionally noted within the clay layer.
- Fill material was not logged in any of the well locations; however, because the well records are relatively old (1959 to 1966), some fill materials may be present in the onshore work area as a result of construction activities and earthworks at the site.
- Overburden in the area is clay material occasionally underlain by a gravel layer.
- A gravel layer was identified below the clay layer in one WWIS record. The water well was constructed in this gravel layer. The thickness of the layer was not explored.
- Encountered bedrock was noted as shale.

According to the geological descriptions provided by ORMGP and WWIS, the site hydrogeology includes the following major units (Figure 10.5):

- Undifferentiated Upper Sediments (Aquifer)
- Lower Oak Ridges Moraine Aquifer Complex (Aquifer)
- Lower Newmarket Till (Aquitard)
- Thorncliffe Formation (Aquifer)
- Sunnybrook Formation (Aquitard)
- Scarborough Formation (Aquifer)
- Bedrock – Georgian Bay-Blue Mountain Formation (Aquitard)
- Bedrock – Cobourg Formation (Aquiclude).

The depth of the shaft construction excavation is expected to be up to 70 mbgs. At the potential shaft locations, hydrostratigraphic formations likely to be encountered include the following:

– Undifferentiated Upper Sediments (Aquifer)
– Lower Newmarket Till (Aquitard)
– Thorncliffe Formation (Aquifer)
– Scarborough Formation (Aquifer)
– Georgian Bay-Blue Mountain Formation (Bedrock)
– Cobourg Formation (Bedrock).

The outfall tunnel will be excavated through the Cobourg Formation first and then advanced through the Georgian Bay-Blue Mountain Formation.

The hydrogeological characteristics of each of these units should be assessed during site investigations to understand groundwater control and dewatering requirements during construction.

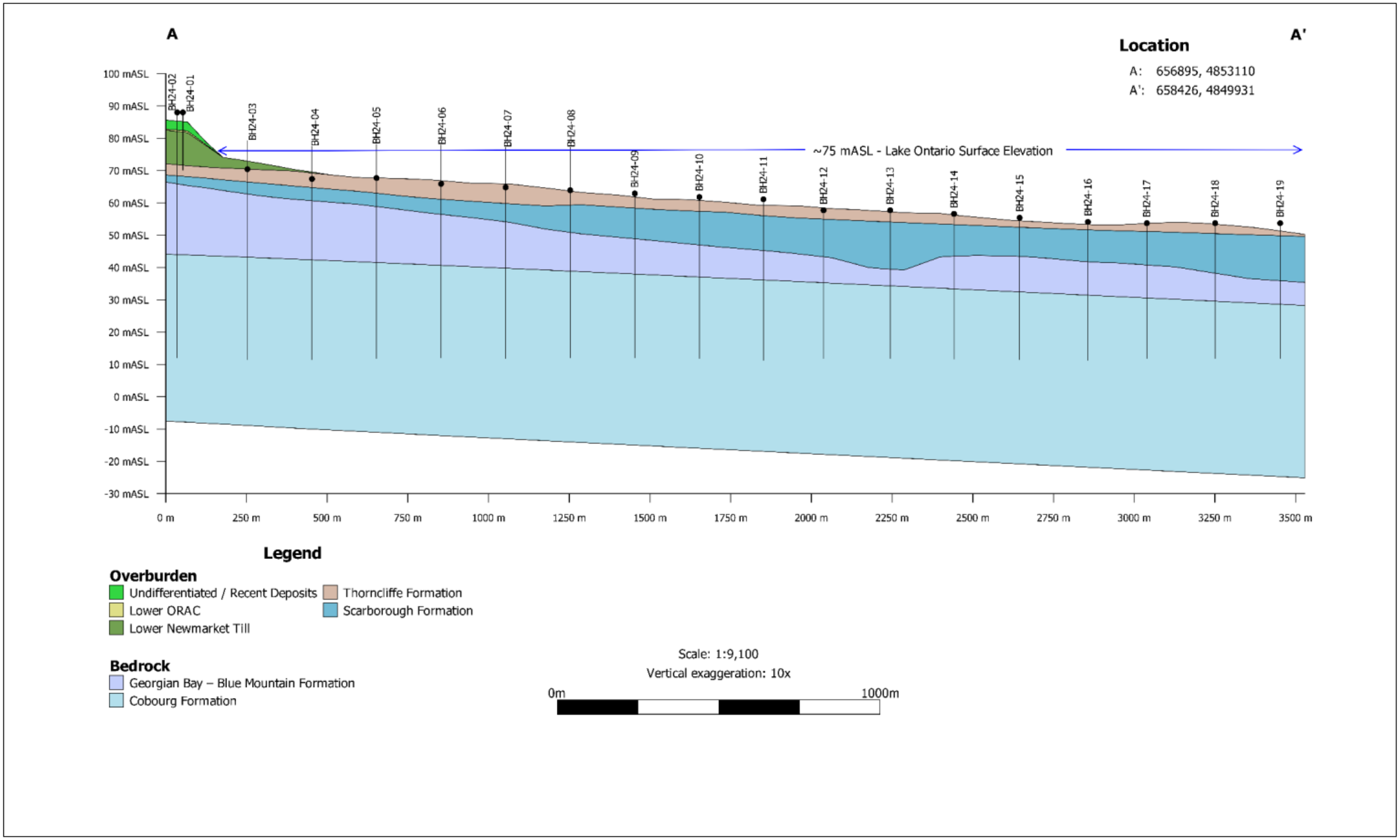

Figure 10.5 **Geological Stratigraphy Profile at the Duffin Creek WPCP Outfall**

10.4.2.2.1 Ground Elevation and Quality

Static water levels associated with WWIS wells within 500 m of the work area were reviewed. Static water levels noted in well records ranged between 4.6 m and 15.2 mbgs.

No groundwater quality data for the bedrock were available for review, and data for the overburden within the work area were limited; therefore, the proposed subsurface investigation described in Section 11 will include groundwater samples collected from the monitoring wells constructed in the overburden and bedrock at the drop shaft.

10.4.2.2.2 Preliminary Assessment of Hydraulic Gradients

The ORMGP water table contour mapping shows a regional groundwater gradient primarily from northwest to southeast of the Oak Ridges Moraine toward Lake Ontario.

Groundwater flow direction is consistent with the surface topography of the work area and surrounding area, which slopes downward from northwest to southeast. Interpretation of an ORMGP mapping suggests the water table elevation near the work area is approximately 75 to 80 masl.

According to ORMGP mapping, the work area and the immediate area are characterized by downward hydraulic gradients between the water table and potentiometric surface. The subsurface investigation will assess the vertical gradient within the Newmarket Till aquitard, Thorncliffe Formation, and Scarborough Formation by using nested groundwater monitoring wells screened within each of the hydrostratigraphic units. This analysis is required to understand groundwater flow and associated pressure between the units and to assess potential effects on construction and dewatering requirements.

10.4.2.3 Natural Heritage Characterization

The natural environment within the study area (shown in Figure 10.6) was assessed based on a background review of secondary source information and subsequent field investigations of the shoreline and tableland areas adjacent to the plant and the aquatic environment within Lake Ontario.

The study area is characterized by the following two main zones:

1. The littoral zone, which is shallow, brighter, warmer, more oxygenated and more vegetated, and therefore could provide habitat and resources for a wide range of aquatic species.
2. The pelagic zone, further out into the lake, is characterized by colder temperatures, less light penetration, fewer food sources, less oxygen, and reduced biodiversity.

A one-day terrestrial environment survey was conducted in June 2023 to determine the geographical extent, composition, structure and function of vegetation communities and to document wildlife species and available wildlife habitat within the survey area. The natural environment characterization of the terrestrial area is described in Chapter 9 of the Project Report.

During the 2011 Outfall Class EA (described in Section 10.3), a natural sciences assessment was conducted by LGL Limited. This assessment included an aquatic survey to characterize substrates in the nearshore (approximately <15 m water depth) and offshore zones of Lake Ontario, as defined by the Great Lakes Fishery Commission (GLFC, 2017)

The aquatic survey included the collection of sidescan sonar imagery and depth data along two proposed outfall alignments. Fish observations were documented as three general categories, including individual large fish (such as rainbow trout), groups of large fish and schools of bait fish (such as alewife).

The results of the 2013 sidescan and sonar imagery confirmed the lack of availability of specialized habitat within the lakebed along the surveyed alignments. It was found that substrates varied from sand to mixtures of sand and cobble, to boulder and exposed bedrock from nearshore to offshore, respectively.

Given that the aquatic component of the local study area includes an exposed shoreline that provides little in the way of refuge for fish and an offshore zone of exposed bedrock and shale, fish habitat in the study area was deemed as providing a forage and passage function.

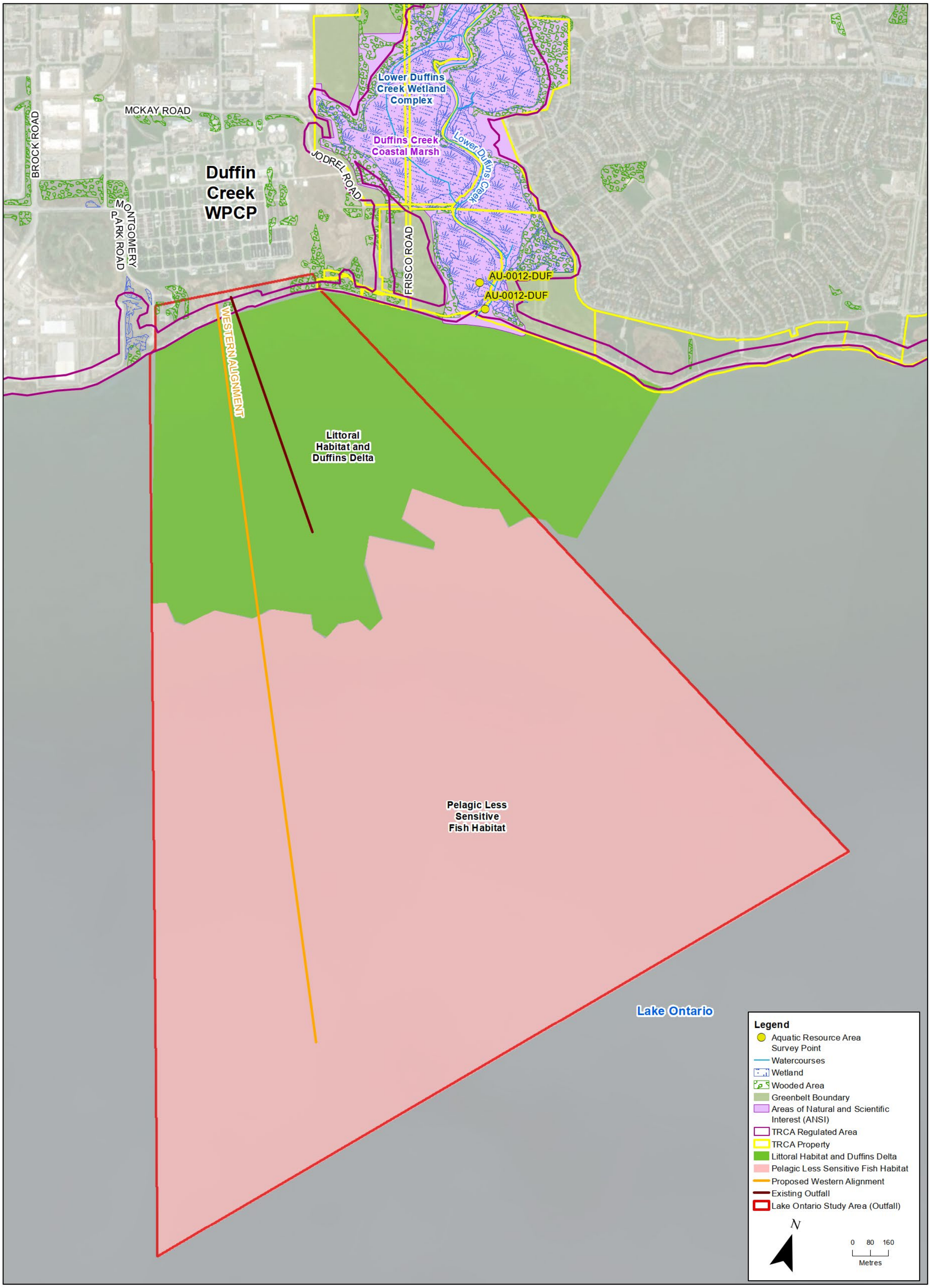

Figure 10.6 **Natural Heritage Boundary Land Information Ontario**

10.4.3 Cultural Environment

10.4.3.1 Marine Archaeological Resources

Archaeological Research Associates was retained in June 2023 to conduct the marine archaeological background research (equivalent of land-based Stage 1 archaeological assessment) for the proposed outfall study area. The local study area extends approximately 4 km into Lake Ontario, and the regional study area extends from Frenchman Bay to the west, east of Paradise Beach on the east, and about 8 km from the base of the WPCP.

Background research indicated that six registered archaeological sites occur within 1 km of the study area, of which only one, AkGs-49, relates to a marine environment or infrastructure.

Several land archaeological assessments were conducted on adjacent lands (Archeoworks 2004, 2005, 2013). Scarlett Janusas Archaeological and Heritage Consulting and Education conducted two marine archaeological assessments at Frenchman's Bay (SJAHCE 2012a, 2012b). These reports indicated no lakebed archaeological artifacts along preliminary outfall alignments.

In support of the Stage 1 marine archaeological assessment, a site visit was conducted in April 2023. It was noted that the westernmost stretches of the shoreline were heavily eroded with steep banks and cutaways, sometimes in excess of 3 to 4 m. The eastern end of the local study area has also been subject to erosion, but the banks are less steep, ranging from between 0.5 to 1.5 m. It is evident, however, that along the entire coastline, erosion has destroyed any remnants of the original shoreline and buried it under sand.

According to the background research and property visit, limited marine archaeological potential exists for the proposed alignment area. Notably, marine archaeological potential for the remainder of the local and regional study areas does exist, as evidenced by the pilings noted along the shoreline in the regional study area, the use of Duffins Creek as a navigable waterway and resource (fish, for example), the use of Frenchman's Bay as a safe harbour, the geotechnical evidence of two ancient river valleys (deeply buried), and the possibility of associated Indigenous remains located along original shorelines.

Work can proceed within the designated area based on this conclusion. If work is to occur outside of this area, an additional review needs to be held by a licensed marine archaeologist.

10.5 Receiving Water Impact Assessment

10.5.1 Introduction

The Receiving Water Impact Assessment (RWIA) is a study that evaluates the assimilative capacity of a waterbody to incorporate substances such that the water quality does not degrade below a predetermined level. This section summarizes the results of a RWIA conducted by Baird & Associates, dated September 14, 2023 (Baird, 2023) to assess the performance of the outfall configurations with the proposed flows for the Duffin Creek WPCP treatment expansion.

10.5.2 Regulatory Framework

Two key MECP documents apply to the derivation of effluent requirements for outfalls. The ambient conditions and the new effluent discharge are evaluated relative to MECP's Policies for Surface Water Quality Management (MECP 1994a) and the MECP Guidelines for Deriving Effluent Requirements (MECP 1994b).

10.5.2.1 Provincial Water Quality Objectives

Effluent requirements for surface water discharges are outlined in the MECP publication, Policies, Guidelines and Provincial Water Quality Objectives of the Ministry of Environment Conservation and Parks (1994a). With regard to surface water quality, the goal is to verify that the water quality is satisfactory for aquatic life and recreation. MECP sets the PWQO at a level of water quality that is protective of all forms of aquatic life and all aspects of the aquatic life cycles during indefinite exposure to the water. The objectives for protecting recreational water use are based on public health and aesthetic considerations.

The water quality parameters that were the focus of the RWIA for the Duffin Creek WPCP include Total Phosphorus (TP), Un-ionized Ammonia (UIA), Total Ammonia Nitrogen (TAN), Escherichia coli (E. Coli), pH and thermal impacts. These parameters are summarized in Table 10.1, along with the PWQO.

Table 10.1 Summary of Duffin Creek WPCP RWIA Parameters of Interest with their Provincial Water Quality Objectives (MECP, 1994a)

Receiving water parameter	PWQO / GLWQA
Escherichia Coli (E. Coli)	100 counts/100 mL (geometric mean of at least five samples) at a designated beach.
Total Phosphorus (TP)	Concentrations should not exceed 0.02 mg/L for the ice-free period to avoid nuisance concentrations of algae.
Un-ionized Ammonia (UIA)	0.02 mg/L at edge of near-field mixing zone, calculated using equations 1 and 2.
Temperature	The temperature at the edge of the mixing zone shall not be more than 10°C above the natural ambient water temperature.
Total Ammonia Nitrogen (TAN)	0.5 mg/L – Based on the Great Lakes Water Quality Agreement (GLWQA) objective
pH	Maintained within range of 6.5 to 8.5

UIA, which can be toxic above certain levels, can be calculated from the formula provided in MECP 1994a, adapted from Emerson et al. 1975, as in equations 1 and 2:

$$f = \frac{1}{10^{pka - pH} + 1} \quad [1]$$

$$pKa = 0.09018 + \frac{2729.92}{T} \quad [2]$$

Where:

– f is the fraction of UIA (NH_3) in total ammonia (NH_3+NH_4^+), and T is the ambient water temperature in Kelvins (K = °C + 273.16).

10.5.2.2 Mixing Zone

The MECP publication titled Deriving Receiving-Water Based, Point-Source Effluent Requirements for Ontario Waters (1994b) provides guidance about the requirements for point-source discharges and the procedures for determining effluent requirements for an Environmental Compliance Approval (ECA). The process outlined for establishing effluent requirements includes the following:

– Site-specific receiving water assessments are to be carried out to determine the effluent requirements based on the assimilative capacity of the receiving water.
– Effluent requirements will be compared with regulatory guidance for effluent discharges.
– Effluent dilution requirements must consider background concentrations in the receiving water body, as defined by 75th percentile values of measured data (25th percentile for dissolved oxygen). Seasonal or diurnal changes may need to be considered.

The RWIA focused on the performance of the proposed outfall concept and its ability to meet effluent mixing requirements and minimize the potential impacts on environmental endpoints, such as recreational uses, aquatic communities, and drinking water treatment plant intakes.

MECP (1994b) defines a mixing zone as an area of water contiguous to a point source or definable nonpoint source where water quality does not comply with one or more of the PWQO. Terms and conditions related to a mixing zone are designated on a case-by-case basis and may be specified in the ECA. Specific requirements of the mixing zone are as follows:

– Mixing zone is to be designed to be as small as possible.
– In the Great Lakes, initial mixing for discharge diffusers must have a minimum near-field (initial mixing) ratio of 20:1.
– Mixing zones cannot interfere with other water uses, such as drinking water supply or recreation.
– MECP terms of reference developed for previous wastewater treatment plant outfall receiving water impact assessments on Lake Ontario state that the PWQO should be met at the edge of the mixing zone and that, as a minimum requirement, the mixing zone should be limited to half the distance between the offshore length of the outfall and the nearest shore, also referred to as the “half-pipe distance”.

The definition of a mixing zone is open to interpretation. For this study, the mixing zone was defined using the far-field model MIKE3. Model predictions were also used to evaluate effluent impacts at other critical locations, such as drinking water intakes, beaches, and selected shoreline locations.

10.5.3 Lake Ontario Background Conditions

The data used to define the background concentrations and physical characteristics in Lake Ontario are shown in Figure 10.7 and summarized in Table 10.2. Water quality data were collected from the TRCA nearshore sampling program, MECP’s Drinking Water Surveillance Program, Environment Canada’s Great Lakes Surveillance Program (GLSP), and raw water samples from adjacent drinking water treatment plants. Lake currents and water temperature data were collected from one of MECP’s Acoustic Doppler Current Profilers, which was deployed in 2012, and TRCA’s temperature string data collected between 2015 and 2018. The data collected were used to define background conditions in the lake offshore of Duffin Creek WPCP.

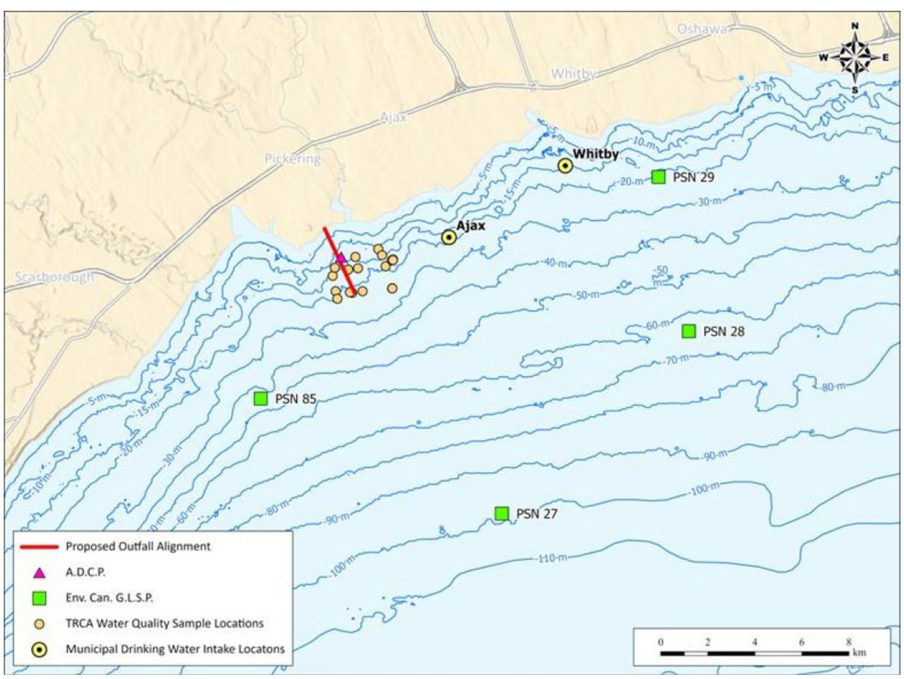

Figure 10.7 Location of Data Sources Used to Define Background Conditions in Lake Ontario

The background conditions in the lake were summarized for the following key parameters: total phosphorus (TP), total ammonia nitrogen (TAN), un-ionized ammonia (UIA), *E. coli*, pH, and water temperature. The modelling approach used requires consideration of background concentrations in the receiving water body, defined by 75th percentile values of measured data, and 25th percentile for depth-averaged current speed.

Table 10.2 Summary of Seasonal Ambient Conditions in Lake Ontario Used for Modelling

Parameter	Parameter	PWQO	All data	Apr 1 to Aug 31	May 1 to Oct 31	Nov 1 to Apr 30
75th percentile	UIA (mg/L)[1]	0.02	0.005	0.008	0.006	0.001
	TAN (mg/L)[2]	0.5[3]	0.040	0.038	0.048	0.013
	TP (mg/L)[2]	0.02	0.009	0.009	0.010	0.008
	pH[2]	-	8.3	8.4	8.4	8.2
	Temp[4]	-	12.0	14.0	17.0	6.0
25th percentile	Current Speed (m/s)	-	0.04	0.03	0.04	0.03
E. coli (#/100 mL)[5]	E. coli (#/100 mL)[5]	100	2	2	2	2

Notes:
m/s = metre(s) per second
mg/L = milligram(s) per litre

Overall, the water quality conditions in the lake are below PWQO with respect to UIA, TP, and *E. coli*, making the lake a Policy 1 receiver; MECP, therefore, requires “water quality shall be maintained at or above the Objectives”. Historically, higher concentrations tend to occur during the warmer months but are still lower than PWQO.

10.5.4 Outfall Discharge Characterization

This section provides the characterization of treated effluent flow and quality from the Duffin Creek WPCP for use in lake modelling.

10.5.4.1 Effluent Flow

The current rated ADF capacity of the Duffin Creek WPCP is 630 ML/d. The rated ADF capacity will be increased to the ultimate site hydraulic capacity of 940 ML/d to accommodate future growth in York Region and Durham Region. The proposed outfall design considered in this RWIA study was developed using a design flow of 940 ML/d.

10.5.4.2 Effluent Quality

The effluent quality presented as 75th percentile of the operating conditions from 2018 to 2022 is summarized in Table 10.3. Calculation of the 75th percentile was based on daily effluent samples collected between 2018 and 2022. The geometric mean was used to define *E. coli* concentrations.

[1] Calculated
[2] GLSP/TRCA
[3] GLWQA objective
[4] Ajax WTP/TRCA/GLSP
[5] TRCA (Geomean)

Table 10.4 shows the current and proposed ECA compliance limits for TP and TAN. The proposed concentration limits for future plant expansion are consistent with those outlined in the current ECA. While the plant is designed to meet stricter effluent objectives (effluent TP concentration of 0.35 mg/L and effluent TAN concentration of 5.0 mg/L), as a conservative measure, the proposed effluent limits (effluent TP concentration of 0.45 mg/L and effluent TAN concentration of 6.0 to 10.0 mg/L) were used in lake modelling to complete the RWIA for Duffin Creek WPCP when it is operating at 940 ML/d.

Table 10.3 Summary of Duffin Creek WPCP Effluent Quality 2018 to 2022

Water quality parameter	Annually (All data)	Apr 1 to Aug 31	May 1 to Oct 31	Nov 1 to Apr 30
UIA (mg/L)[a]	0.0019	0.0020	0.0018	0.0019
TAN (mg/L)	0.60	0.74	0.58	0.64
TP (mg/L)	0.37	0.38	0.39	0.34
pH	7.1	7.1	7.1	7.1
Effluent Temperature (°C)	19.4	20.9	21.1	15.7
E. coli Geometric Mean (counts/100 mL)	34	33	30	39

[a] Calculated value using TAN, water temperature, and pH.

Table 10.4 Summary of Duffin Creek WPCP Effluent Criteria

Compliance limits	Annually (All data)	Apr 1 to Aug 31	May 1 to Oct 31	Nov 1 to Apr 30
Current compliance limits for TP[a] (mg/L)	-	-	0.8 (monthly average)	0.8 (monthly average)
Current compliance limits for TAN (mg/L)	-	-	6.0 (monthly average)	10.0 (monthly average)
Proposed compliance limits for TP at 940 ML/d[a] (mg/L)	0.45 (annual average)	0.45 (seasonal average)	-	-
Proposed compliance limits for TAN at 940 ML/d (mg/L)	-	-	6.0 (monthly average)	10.0 (monthly average)

[a] Per the current ECA, the Duffin Creek WPCP is currently required to achieve an effluent TP concentration limit of 0.8 mg/L (monthly average). Upon completion of upgrades in the liquid and solid treatment trains, the concentration limit will decrease to 0.45 mg/L (annual average and seasonal average from April to August).

10.5.4.3 Required Effluent Dilution

MECP terms of reference for recent receiving water impact assessments on Lake Ontario (for example, the G.E. Booth [formerly Lakeview] Wastewater Treatment Plant [WWTP] and Clarkson WWTP) stated that the PWQO should be met at the edge of the near-field mixing zone, and that as a minimum requirement, the extent of the near-field mixing zone should be limited to half the distance between the offshore length of the outfall and shore. The definition of a mixing zone is open to interpretation, but this half-pipe distance is generally accepted as a starting point for evaluation by MECP; for the purpose of this study, the mixing zone is defined based on the far-field model results discussed in Section 6.7.2 of this report. The half-pipe distance was used in the near-field analysis to evaluate the performance of the outfall system in a region closer to the diffuser where rapid dilution occurs. An understanding of the dilutions required to achieve PWQO is needed to support this assessment; this was achieved using equation 3:

$$Dilution\ Ratio = \frac{C_{Eff} - C_{Amb}}{C_{PWQO} - C_{Amb}} \quad [3]$$

Where:

– C_{Eff} = Duffin Creek WPCP effluent concentration (mg/L)

– C_{Amb} = Lake ambient concentration (mg/L)

– C_{WQO} = Water quality objective concentration (mg/L).

The required target dilutions were determined using this formula considering the proposed effluent compliance criteria for TP and TAN (Table 10.4), 75th percentile ambient lake concentrations (Table 10.2), and the PWQO (Table 10.2). The results are presented in Table 10.5. The water quality parameter with the highest dilution requirement is the governing constituent, and the diffuser system must achieve this level of dilution at a minimum. The results showed that TP is the governing constituent. Dilution requirements to meet the PWQO for TP were the greatest at 40:1; therefore, this minimum dilution target was used in the conceptual design of the outfall diffuser system.

Table 10.5 Seasonal Dilution Targets for Key Water Quality Parameters (Based on Proposed Compliance Limits for TAN and TP)

Water quality parameter	Annually (All data)	Apr 1 to Aug 31	May 1 to Oct 31	Nov 1 to Apr 30
UIA (mg/L)	<1:1	<1:1	<1:1	<1:1
TAN (mg/L)	NA	NA	13:1	21:1
TP (mg/L)	40:1	40:1	NA	NA
pH	2:1	2:1	2:1	2:1
Temperature[6]	4:1	4:1	2:1	5:1

Notes:

< = less than

NA = not applicable

10.5.5 Outfall and Diffuser Configurations

Two options were evaluated and modelled for this conceptual planning study. The proposed outfall options include a 500 m long diffuser starting 2,000 m from shore (Option 1) and 2,500 m from shore (Option 2). The average water depths over the length of the diffuser for Options 1 and 2 are 17 m and 20 m, respectively. There are 34 risers over the entire length of the diffuser with a port diameter of 0.7 m. The diffuser sections for both options discharge treated effluent into the pelagic zone of Lake Ontario, which is further out into the lake and is characterized by colder temperatures, less light penetration, less food sources, less oxygen and reduced biodiversity.

10.5.6 Lake Modelling Approach

The numerical modelling approach used both near-field and far-field modelling techniques. The two models used in this study were CORMIX and MIKE3.

The CORMIX model predicts dilutions of the effluent in the near-field region, which is the mixing where the momentum and buoyancy characteristics of the effluent jet dominate the mixing process.

The half-pipe distance was used as a metric to support the near-field modelling activities; that is, PWQO were assessed against predicted concentrations from the CORMIX model at select distances away from the point of discharge up to the half-pipe distance (150 m, 300 m, and 600 m). As shown in Figure 10.8, the half-pipe distance for Option 1 is 1,060 m away from the proposed diffuser concept, and for Option 2, it is 1,310 m away. Note that initial dilution was used as a surrogate for concentration as part of the near-field analysis. The results from CORMIX were

[6] Assumed thermal criteria that ambient discharge temperatures cannot exceed 2 degrees Celsius (°C) above ambient.

used to assist in designing the proposed diffuser by evaluating its performance with respect to generating the required mixing and dilution to achieve PWQO.

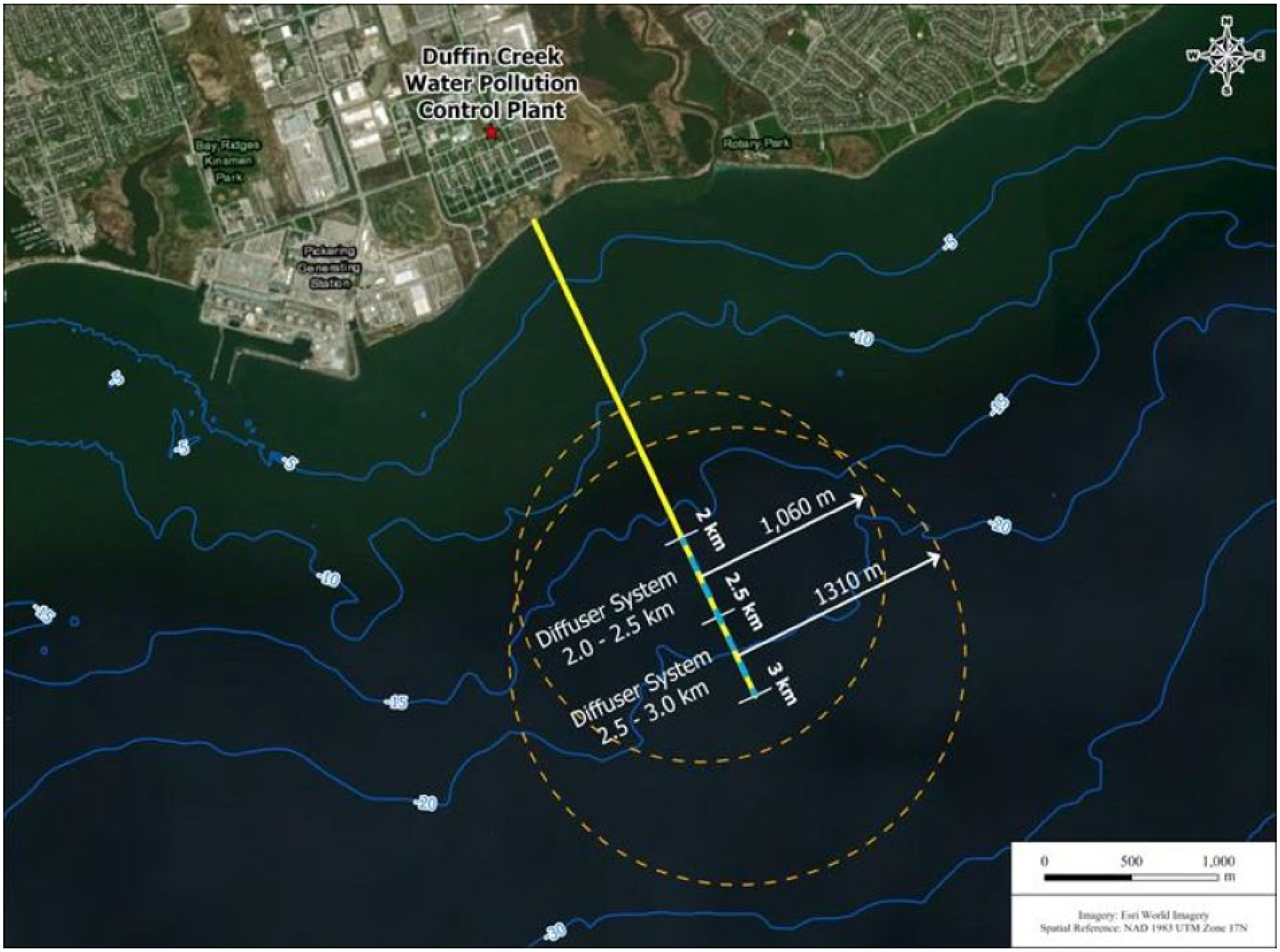

Figure 10.8 **Half-pipe Distance for the Proposed Diffuser Concepts at Duffin Creek WPCP**

The MIKE3 model predicts far-field mixing, where mixing is associated with ambient lake processes, and tends to occur at a greatly reduced rate. The MIKE3 model was used to predict the movement of the mixing zone and its impact on beneficial uses (that is, water intakes, beaches and aquatic habitat).

10.5.7 Lake Modelling Results

10.5.7.1 Near-field Assessment

The CORMIX model was set up to predict initial dilutions for both diffuser options using the seasonal compliance criteria proposed for TP and TAN (that is, 0.45 mg TP/L, 6 mg TAN/L for period between May 1 to October 31, and 10 mg TAN/L for period between November 1 to April 30).

Output from the CORMIX model includes centreline dilution predictions at various distances away from the diffuser. Four distances were selected to evaluate the results, including the half-pipe length of 1,060 m for Option 1 and 1,310 m for Option 2. The initial dilutions are based on low current speeds (25th percentile) and high effluent concentrations (compliance limits or 75th percentile values). The dilution predictions from CORMIX for the two diffuser options are shown in Table 10.6 and Table 10.7.

Table 10.6 CORMIX Initial Dilution Predictions for Diffuser Option 1 (2,000 m Outfall + 500 m Diffuser)

ECA seasons	Target dilution – TP	Target dilution – TAN	Predicted dilution from diffuser 150 m	Predicted dilution from diffuser 300 m	Predicted dilution from diffuser 600 m	Predicted dilution from diffuser 1,060 m	Distance to NFR
Annually	40:1	N/A	33:1	40:1	45:1	50:1	173
Apr 1 to Aug 31	40:1	N/A	21:1	30:1	38:1	43:1	281
May 1 to Oct 31	N/A	13:1	33:1	39:1	44:1	49:1	173
Nov 1 to Apr 30	N/A	21:1	21:1	30:1	38:1	42:1	281

Note:
NFR = near-field region

Table 10.7 CORMIX Initial Dilution Predictions for Diffuser Option 2 (2,500 m Outfall + 500 m Diffuser)

ECA seasons	Target dilution – TP	Target dilution – TAN	Predicted dilution from diffuser 150 m	Predicted dilution from diffuser 300 m	Predicted dilution from diffuser 600 m	Predicted dilution from diffuser 1,060 m	Distance to NFR
Annually	40:1	N/A	40:1	46:1	52:1	60:1	162
Apr 1 to Aug 31	40:1	N/A	26:1	36:1	44:1	51:1	249
May 1 to Oct 31	N/A	13:1	40:1	45:1	51:1	59:1	162
Nov 1 to Apr 30	N/A	21:1	26:1	36:1	43:1	50:1	249

As shown in Table 10.6 and Table 10.7, the near-field region terminates within 300 m of the diffuser. The governing constituent (TP) dilution (40:1), on an average annual condition, is achieved within 300 m for Option 1 and within 150 m for Option 2. During the period of April 1 to August 31, target dilutions are met within 800 m for Option 1 and 400 m for Option 2.

10.5.7.2 Far-field Assessment

The 3D finite difference model MIKE3 FD was developed for this study and used to define the mixing zones for key constituents (TP, UIA) and to predict the impact at key locations near shore and at adjacent drinking water intakes (TP, TAN).

The 75th percentile values for ambient background TP and TAN (averaged over all seasons), shown in Table 10.2, were used to define the initial receiving water quality conditions in the lake.

Output from the MIKE3 model was used to predict water quality concentrations at shoreline locations and at the Ajax WSP intake, as shown in Figure 10.9.

Figure 10.9 Receiving Water Impact Assessment Fiduciary Markers

10.5.7.3 Effluent Impacts at Shore

Table 10.8 summarizes the maximum instantaneous, the peak (average) daily, and the average concentration for TP at key locations under the annual seasonal period.

Table 10.8 Predicted TP Concentrations at Shore and Drinking Water Intake (Annually)

Fiducial markers	Option 1 – Instantaneous Max (mg/L)	Option 1 – Peak day (mg/L)	Option 1 – Average (mg/L)	Option 2 – Instantaneous Max (mg/L)	Option 2 – Peak day (mg/L)	Option 2 – Average (mg/L)
Ajax water supply plant Intake	0.016	0.014	0.010	0.015	0.013	0.010
Petticoat Creek CA	0.015	0.013	0.009	0.012	0.012	0.009
Frenchman's Bay	0.014	0.012	0.009	0.012	0.012	0.009
Beachfront Park	0.014	0.013	0.009	0.012	0.012	0.009
Pickering Nuclear Generation Station	0.013	0.012	0.009	0.012	0.012	0.009
Rotary Park	0.013	0.013	0.009	0.012	0.012	0.009
Ajax Waterfront Park	0.013	0.012	0.009	0.012	0.012	0.009
Carruthers Marsh	0.013	0.012	0.009	0.011	0.011	0.009

The model predicted that the peak daily TP concentration did not exceed PWQO at any of the locations under the future flow condition. Furthermore, the maximum instantaneous value was also less than PWQO at all fiducial locations. The average TP concentration over the simulation period remained at or slightly greater than background concentrations (0.009 mg/L) at all locations.

It is worth noting that if background concentration of TP at the shoreline locations is higher due to other inputs of phosphorus from the shore (for example, tributary watercourses; stormwater), it is possible that the relative effect of discharge from the proposed new outfall would be even less discernible at the shoreline locations.

10.5.7.4 Effluent Impacts at Drinking Water Intakes

TAN concentrations were assessed at the Ajax WSP intake (at lakebed) and other shoreline locations shown on Figure 10.9. TAN was used as a surrogate measure of the conditions at the water filtration plant intakes and an indicator of effluent impacts on intake quality.

Table 10.9 and Table 10.10 summarize the maximum instantaneous, the peak (average) daily, and the average concentration for TAN at all locations over the 1-year simulation based on the proposed compliance periods (that is, May 1 to October 31, and November 1 to April 30).

Table 10.9 Predicted TAN Concentrations at Shore and Drinking Water Intake (May 1 to Oct 31)

Fiducial markers	Option 1 – Instantaneous max (mg/L)	Option 1 – Peak day (mg/L)	Option 1 – Average (mg/L)	Option 2 – Instantaneous max (mg/L)	Option 2 – Peak day (mg/L)	Option 2 – Average (mg/L)
Ajax water supply plant Intake	0.13	0.10	0.05	0.12	0.09	0.05
Petticoat Creek CA	0.12	0.09	0.05	0.09	0.09	0.04
Frenchman's Bay	0.11	0.08	0.05	0.08	0.08	0.04
Beachfront Park	0.10	0.09	0.05	0.10	0.09	0.04
Pickering Nuclear Generation Station	0.10	0.08	0.05	0.08	0.07	0.05
Rotary Park	0.10	0.09	0.05	0.09	0.08	0.05
Ajax Waterfront Park	0.10	0.09	0.05	0.09	0.08	0.04
Carruthers Marsh	0.11	0.09	0.05	0.08	0.07	0.04

Table 10.10 Predicted TAN Concentrations at Shore and Drinking Water Intake (Nov 1 to April 30)

Fiducial markers	Option 1 – Instantaneous max (mg/L)	Option 1 – Peak day (mg/L)	Option 1 – Average (mg/L)	Option 2 – Instantaneous max (mg/L)	Option 2 – Peak day (mg/L)	Option 2 – Average (mg/L)
Ajax water supply plant Intake	0.20	0.13	0.06	0.11	0.09	0.05
Petticoat Creek CA	0.10	0.10	0.05	0.11	0.10	0.05
Frenchman's Bay	0.10	0.09	0.05	0.11	0.10	0.05
Beachfront Park	0.10	0.10	0.05	0.11	0.10	0.05
Pickering Nuclear Generation Station	0.10	0.09	0.05	0.11	0.10	0.05
Rotary Park	0.09	0.08	0.05	0.08	0.07	0.04
Ajax Waterfront Park	0.09	0.08	0.05	0.08	0.07	0.04
Carruthers Marsh	0.08	0.07	0.05	0.07	0.06	0.04

The findings showed that predicted TAN concentrations remained less than the GLWQA source water protection objective of 0.5 mg/L at all locations and for all statistical metrics. The impact at other drinking water intakes situated farther away from the diffuser would be minimal, based on the findings at the Ajax WSP.

10.5.7.5 Mixing Zone for TP

Figure 10.10 and Figure 10.11 show the predicted far-field mixing zones for TP based on two ECA compliance seasons (annual and April 1 to August 31). The mixing zones are represented by the red area. The red area indicates those grid points where PWQO was exceeded at least 10% of the time.

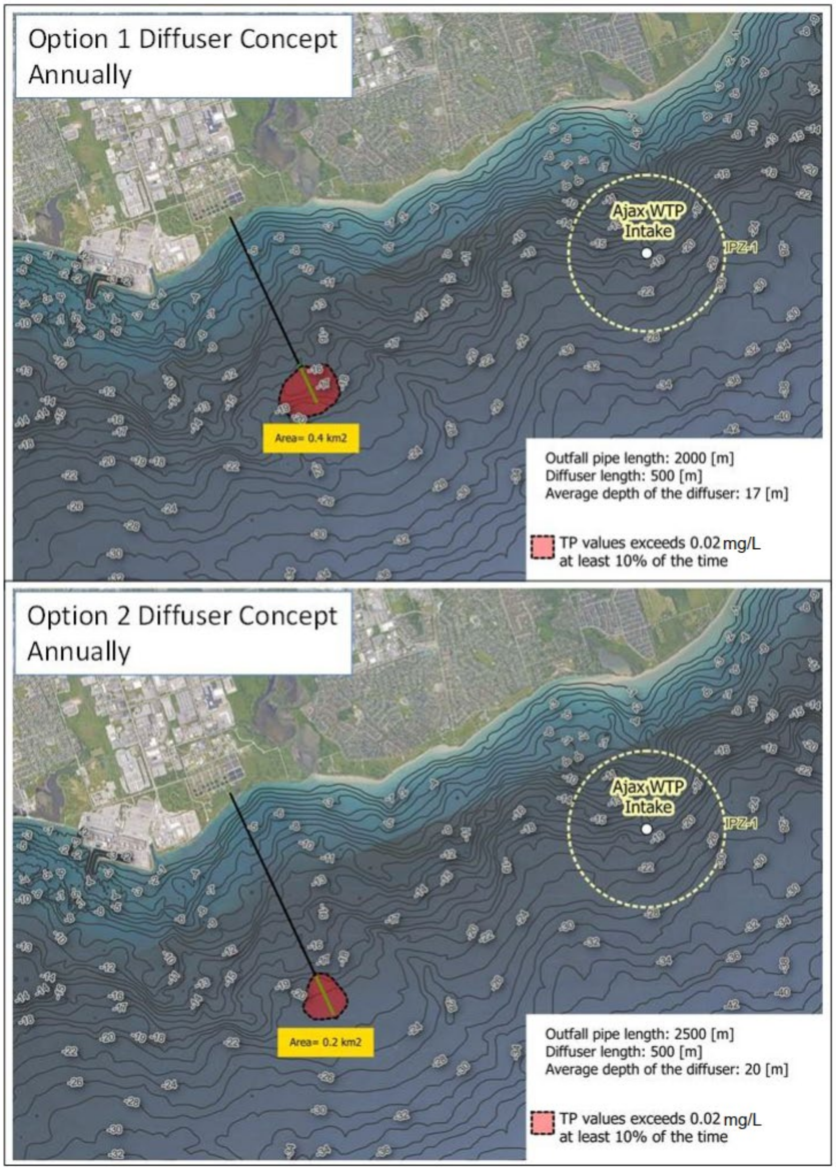

Figure 10.10 Predicted Mixing Zone for TP for the Annual Period

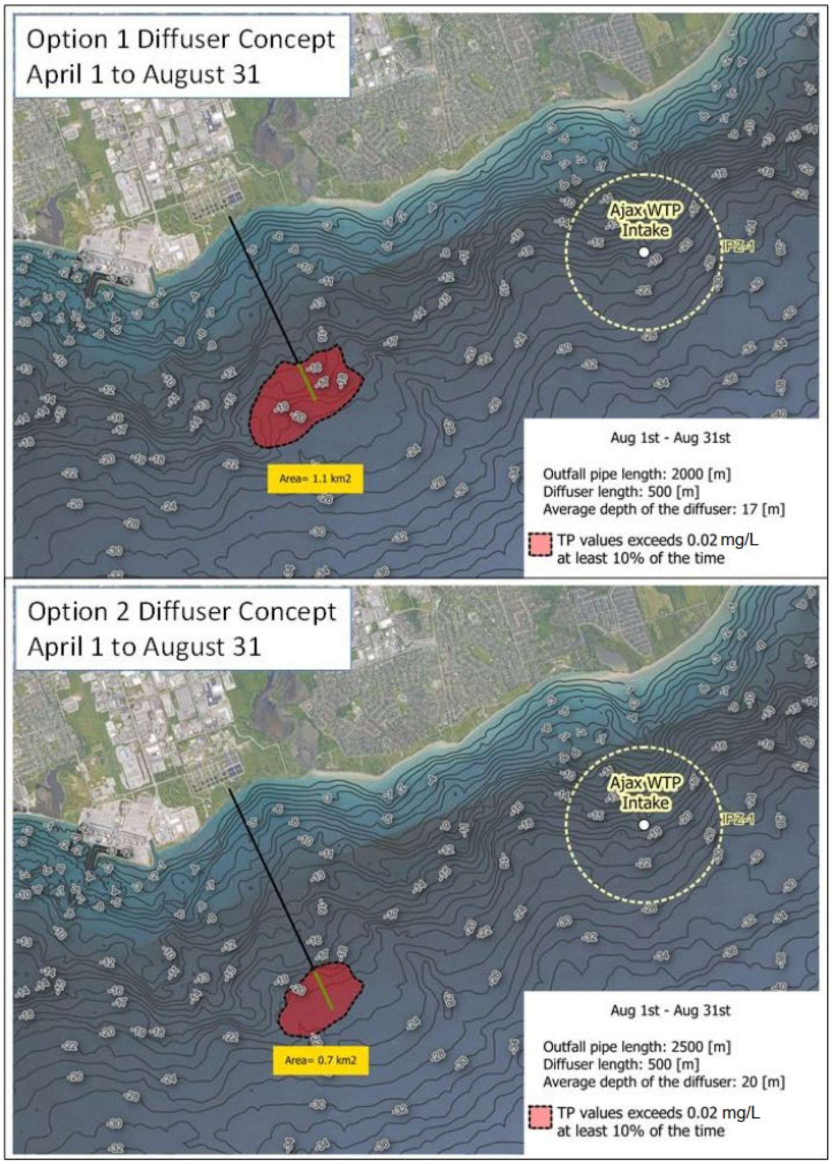

Figure 10.11 Predicted Mixing Zone for TP from April 1 to August 31 Period

The size of the mixing zone for TP ranged from 0.4 square kilometre (km^2) to 0.2 km^2, based on the annual period for diffuser Options 1 and 2, respectively. The footprint for Option 2 is smaller than Option 1, as the outfall is located farther offshore in deeper water. During the algal growth season, which is represented by the April 1 to August 31 period, the mixing zone increased in size because of weaker current conditions and limited mixing caused by stratification in the water column. The areas of the mixing zones for diffuser Options 1 and 2 for this seasonal period were determined to be 1.1 km^2 and 0.7 km^2, respectively.

10.5.7.6 Conclusions

The findings from the near-field analysis showed that the target dilution of 40:1 is achieved within 300 m for Option 1 and within 150 m for Option 2, based on average annual conditions. During the period of April 1 to August 31, target dilutions are met within 800 m for Option 1 and 400 m for Option 2. Both options meet the PWQO at their respective half-pipe distances.

For both outfall configurations, predicted that TP concentrations remained lower than the PWQO at all shoreline locations assessed, including the Ajax WSP intake on a maximum instantaneous basis and on a peak daily average concentration basis. The average TP concentration remained at or slightly above background concentrations at all locations.

Similar trends were observed for TAN, as predicted concentrations remained lower than the GLWQA source water protection objective of 0.5 mg/L at all locations and for all statistical metrics. Maximum and peak daily average TAN concentrations at the Ajax WSP intake during the November 1 to April 30 period are slightly lower for Option 2 compared with Option 1.

The size of the mixing zone for TP ranged from 0.4 km^2 to 0.2 km^2 for the annual period and 1.1 km^2 to 0.7 km^2 for the April 1 to August 31 period for Option 1 and Option 2, respectively.

The lake near-field and far-field model results show that both options meet the provincial regulatory requirements defined in the MECP's Policies for Surface Water Quality Management (MECP, 1994a) and the MECP Guidelines for Deriving Effluent Requirements (MECP, 1994b) and therefore can be considered for implementation at the Duffin Creek WPCP.

Additional lake modelling will be conducted during preliminary design to define the optimal outfall configuration, including length, number of diffusers, port opening diameter/type, and diffuser length.

10.6 Outfall Configuration

The results of the RWIA assessment demonstrate that both outfall configuration options meet the MECP regulatory requirements and therefore can be considered for implementation. The preferred outfall configuration will be defined in the subsequent design phase to account for results on the future geotechnical investigations and optimization of the outfall configuration, including length, number of diffusers, port opening diameter/type, and diffuser length.

For the purpose of this assessment, of the two outfall configurations modelled, Option 2 was selected to be carried forward for further evaluation. The design concept for the Duffin Creek WPCP New Outfall (based on Option 2) consists of the following major components:

- **Onshore work area**: Along the shore of Lake Ontario, at Duffin Creek WPCP, approximately 40,000 square metres (m^2).
- **Drop shaft**: 12 m internal diameter (ID) launch onshore shaft.
- **Tunnel**: Single 5.5 m ID tunnel constructed in bedrock, extending approximately 3,000 m straight out from the launch shaft beneath Lake Ontario.
- **Diffuser**: 34 risers (1 m ID) with diffuser ports (730 millimetres [mm] ID), constructed inline with the tunnel at equal spacing along a 500 m length diffuser section (starting 2,500 m offshore), extending vertically from the tunnel to the lakebed.

10.7 Hydraulic Design of the Outfall

10.7.1 Hydraulic Analysis Methodology

10.7.1.1 Vertical Survey Reference Datum

Hydraulic calculations were based on the plant datum (PD) elevations; however, to avoid confusion about lake water levels that are taken from public sources, the commonly used 1985 International Great Lakes Datum (IGLD85) is also shown. IGLD85 is 0.26 m lower than PD.

10.7.1.2 Lake Ontario Design Lake Level

Water levels of Lake Ontario have been measured since 1918 and are referenced to the IGLD85. For the Stage 3 expansion project and the 2011 Outfall Class EA, a design high lake level of 76.02 m PD (75.76 m IGLD85) was used. Lake Ontario's level exceeded this historical design lake level in 2017 and 2019. In June 2019, the high lake level of 76.17 m PD (75.92 m IGLD85) was 0.72 m above the 10-year average water level of 75.45 m PD (75.19 m IGLD85) and 10 centimetres higher than the previous record set in June 2017.

The selection of a new design high lake level is critically important because this level will determine the available hydraulic head to convey effluent into the lake. In addition, since the same conditions that lead to high lake levels can also contribute to elevated flow to the plant, it is likely that the peak flow to the plant may coincide with a future high lake level event. In this scenario (high flow and high lake level), the available hydraulic head is at its lowest (worse-case scenario).

Given that lake water levels are influenced by many factors (precipitation, snowmelt runoff, drought, evaporation rates, and withdrawing rates), and that the future outfall will be designed to provide a nominal service life of approximately 100 years, the outfall design should provide resilience for managing extreme variability in wastewater flow rates and variations in lake water levels associated with climate change.

To account for the uncertainty in lake water levels along the outfall lifespan, a new design high lake level of 77.18 m PD (76.92 m IGLD85) was selected based on the highest measured lake level that occurred in 2019 (75.92 m IGLD85) and a contingency factor of 1 m.

10.7.1.3 Available Hydraulic Head

The available hydraulic head to drive flow through the outfall and diffusers is dictated by the water level in Lake Ontario and water level at the outfall drop shaft. If the head required to drive effluent water through the outfall is greater than the difference in these elevations, flooding will occur in the plant.

For this planning phase, the chlorine contact tank weirs were used as the first hydraulic control point upstream of the outfall drop shaft, allowing for a maximum water level at the drop shaft of 80.77 m PD. Therefore, the plant's available head is 3.6 m, based on the design lake level (76.18 m PD) and the maximum water level established at the drop shaft.

10.7.1.4 Design Effluent Flow Rates

The design flows considered in this analysis are presented in Table 10.11. The hydraulic assessment was based on the proposed rated peak flow capacity for the Duffin Creek WPCP Expansion. The proposed rated ADF capacity and the ADF forecasted by the time the outfall is in operation were reviewed to assess tunnel velocities and potential for solids settling in the outfall.

Table 10.11 Design Flows for the Duffin Creek WPCP Outfall

Design criteria	Value	Basis
Proposed Rated Peak Flow Capacity, ML/day	3,290	Based on full build-out firm pumping capacity of Stages 1&2 and Stages 3&4 influent pumping stations
Proposed Rated Plant Capacity, ML/day	940	Based on a design wet weather peaking factor of 3.5 and total installed firm pumping capacity
ADF forecasted by 2036 ML/day	612	The ADF forecasted by the time the outfall is in operation

10.7.1.5 Outfall Hydraulic Model Setup

The hydraulic analysis for the outfall components was set up in an Excel-based model using first principle calculations. The outfall was divided into the following sections:

– **The first 2,500 m of tunnel starting from the outfall shaft**: Headloss is mainly associated with friction loss and also includes the entrance loss from the shaft into the tunnel.
– **The last 500 m of the tunnel (diffuser section) starting 2,500 m offshore**: Headloss is associated with friction loss (from the tunnel and riser pipes) and minor losses (riser pipes and diffuser ports), as illustrated on Figure 10.12.
– Note that for the diffuser conduit section, headloss is governed by the lake water level. In other words, the headloss through each of the 34 riser/port assemblies is the same, which equals the difference between the water level in the shaft and the lake water level, minus the friction loss in the 2,500 m tunnel section.

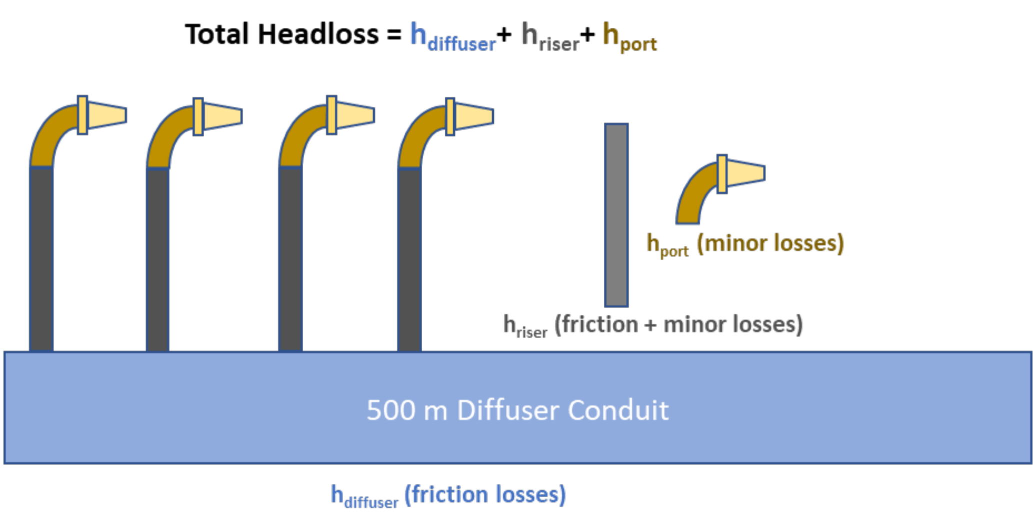

Figure 10.12 Illustration of Headloss Components for the Diffuser Section of the Outfall

The friction headloss in the drop shaft, outfall and risers was calculated using the Hazen-Williams equation, equation 4 and minor headlosses were calculated using equation 5:

The Hazen-Williams equation is expressed as follows:

$$h_f = \frac{10.67\ L\ \ Q^{1.85}}{C^{1.85}\ \ d^{4.87}} \quad [4]$$

Where:

– h_f if the headloss is due to friction (m)
– L is the length of the pipe segment (m)
– Q is the flow through the pipe (cubic metres per second)
– C is the friction factor
– d is the pipe diameter (m)

Minor losses were calculated using the following equation:

$$h = K\left(\frac{V^2}{2g}\right) \quad [5]$$

Where:

– h is the minor headloss (m)
– K is the headloss factor associated with the fitting for those conditions
– V is the water velocity (m/s)
– g is the gravitational constant of 9.806 m/s^2

The headloss factor and minor loss coefficients are presented in Table 10.12 and illustrated on Figure 10.13.

Table 10.12 Hydraulic Design Headloss Factors for Outfall Pipe

Design criteria	Coefficient Type	Value	Notes	Reference
Friction factor – (Hazen-Williams)	C for concrete	100	Range is 100 to 140: – 100 (poor condition) – 140 (new pipe)	*Handbook of Hydraulics*, Seventh Edition, p. 6.29
Minor loss coefficient	Exit loss and reducer (at diffuser port) (k1 and k2)	1.18	– Exit loss k1 = 1 – Reducer loss k2 = 0.18 (1.0 m riser to 0.73 m diffuser port, contraction d2/d1 = 1.4, average velocity in port 3.0 m/s)	*Handbook of Hydraulics*, Seventh Edition, p. 6.34 to 6.36
	Elbow loss at top of riser (k3)	0.2	Elbow with flanged long-radius fitting	*Handbook of Hydraulics*, Seventh Edition, p. 6.37
	Sudden contraction (into riser) (k4)	0.48	Contraction d2/d1 = 5.5, from 5.5 m ID diffuser pipe into 1.0 m ID risers, average velocity in riser 1.6 m/s	*Handbook of Hydraulics*, Seventh Edition, p. 6.38, Table 6.7
	Tee branch (into riser) (k5)	0.9	Tee, threaded, dividing line flow	Water-Resources Engineering, Third Edition, p. 47, Figure 2.7
	Elbow and contraction (into the outfall pipe) (k6)	0.9	– Contraction d2/d1 = 2.1, from 12 m ID shaft into 5.5 m tunnel. – Elbow loss is 0.5	*Handbook of Hydraulics*, Seventh Edition, p. 6.38, Table 6.7

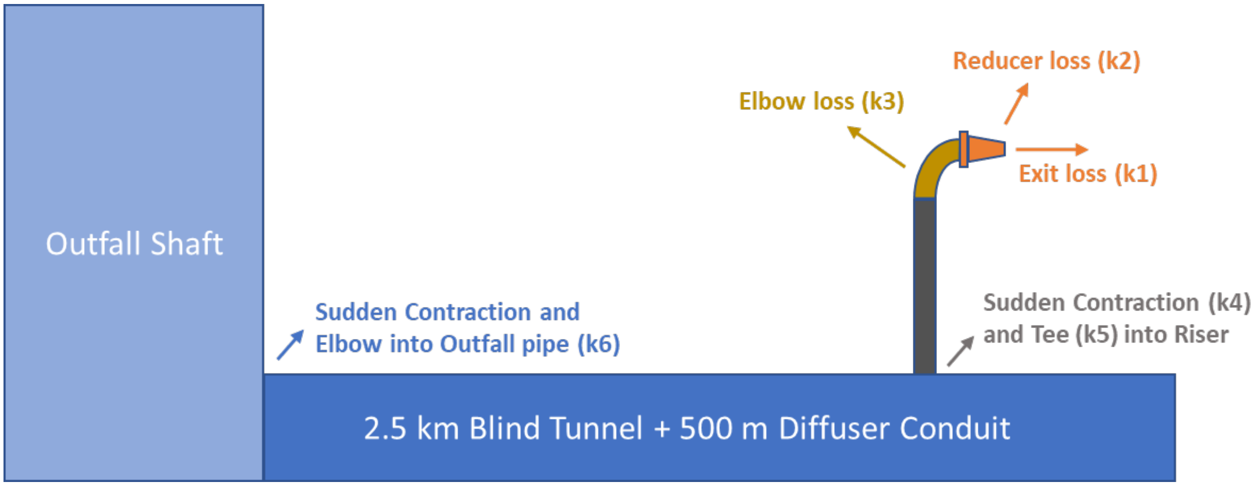

Figure 10.13 Minor Loss K Factors for Outfall Tunnel, Risers, and Diffuser

10.7.2 Hydraulic Analysis Results

The calculated headloss with a range of Hazen-Williams friction factors (C-value, 100 [worst case] and 140 [best case]) for each key component of the outfall (at peak flow of 3,290 ML/d) is presented in Table 10.13.

The design C-value selected is applicable to both cast-in-place (CIP) concrete and precast concrete tunnel lining (PCTL) segments. Typical C-value is in the range of 130 to 140 for new CIP concrete and 120 to 130 for new PCTL segments. Over time, concrete materials will deteriorate and result in a lower C-value. The design C-value of 100 was selected to represent the worst-case scenario for all concrete materials.

Table 10.13 Impact of C-Factor on Calculated Outfall Headloss

Component	Headloss based on C-value = 100 (Friction + minors losses) (m)	HGL elevation based on C-value = 100 (m PD)	Headloss based on C-value = 140 (Friction + minor losses) (m)	HGL elevation based on C-value = 140 (m PD)
Lake Ontario (High Lake Level)		77.180		77.180
Diffuser conduits (Tunnel + Riser + Ports)	0.945	78.125	0.915	78.095
Blind tunnel (2,500 m)	1.230	79.352	0.713	78.808
Outfall shaft	0.0007	79.352	0.0004	78.808
Total headloss	**2.172**		**1.628**	

Note:
HGL = hydraulic grade line

The total headloss under the worst-case scenario (that is, peak flow of 3,290 ML/d, C-value of 100 for concrete at poor conditions) is estimated to be 2.17 m from the top of the shaft through the outfall. This headloss does not exceed the available hydraulic head between the chlorine contact tank weirs and the lake level (3.6 m, summarized in Section 10.7.1.3). Therefore, the water level in the outfall shaft will not exceed 80.77 m PD and no flooding will occur at the plant under these design conditions.

The hydraulic profiles under the peak and average flow conditions with a range of Hazen-Williams friction factors (C-value, 100 [worst case] to 140 [best case]) are presented on Figure 10.14.

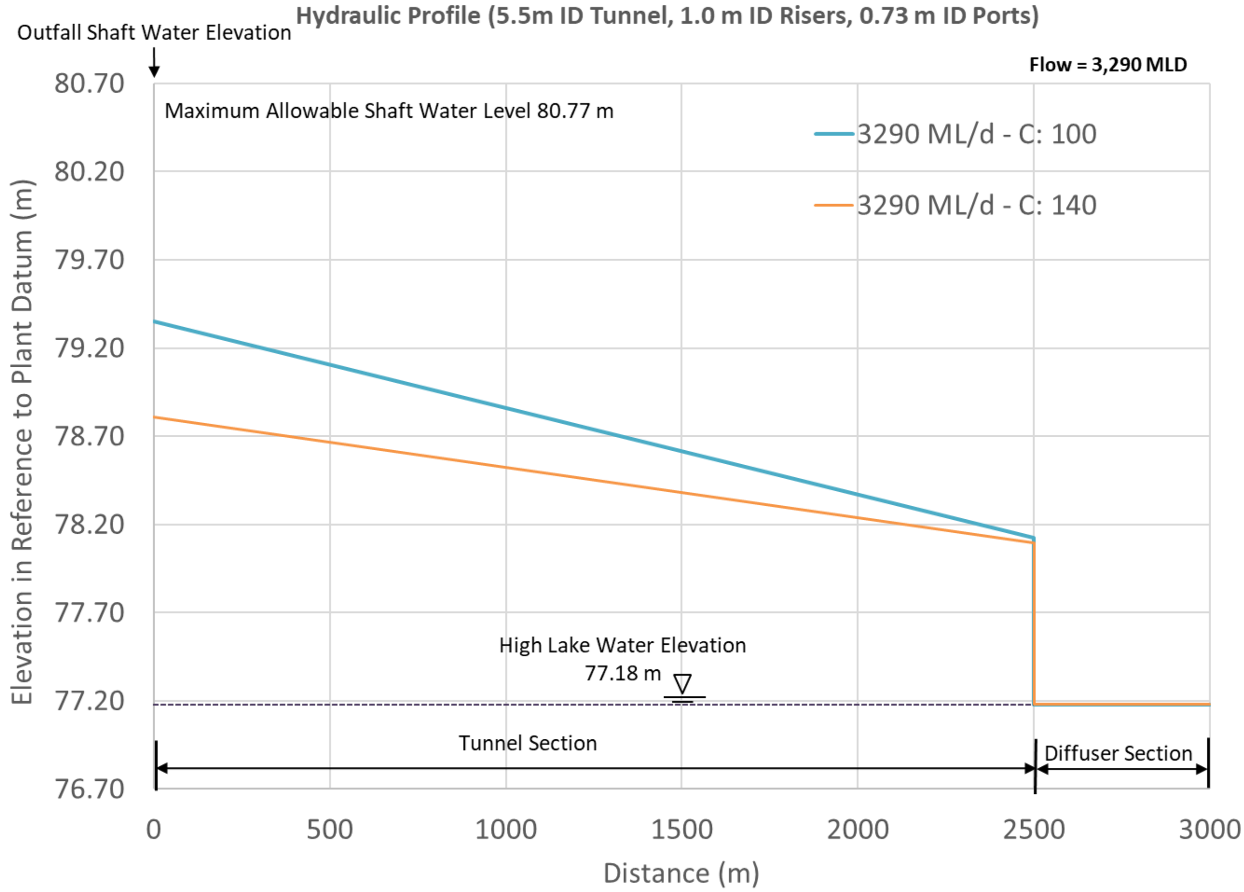

Figure 10.14 **Outfall Hydraulic Profile under C-100 and C-140 Hazen-William Friction Factors**

10.8 Outfall Conceptual Design and Construction Considerations

10.8.1 Effluent Channels Design Concept

The existing Stage 1, 2, and 3 and new Stage 4 effluent channels will be connected to the proposed drop shaft located west of the existing outfall. For this planning level, the conceptual connection plan shown on Figure 10.15 was developed. Alternative connections will be further investigated during preliminary design.

The conceptual connection plan illustrated on Figure 10.15 includes the following:

- **Stage 4 flows**: An effluent channel extending from the eastern side from the Stage 4 chlorine contact tanks, 4 m wide and 3.6 m deep (internal dimensions).
- **Stage 3&4 flows**: An effluent channel built parallel along the existing Stage 3 effluent channel, combining Stage 3 and Stage 4 flows , 4.0 m wide and 3.6 m deep (internal dimensions).
- **Stage 1&2 flows**: An effluent channel extending from the existing Stage 1&2 effluent channel and connecting to the “all flows” effluent channel, 4.0 m wide and 3.6 m deep (internal dimensions).
- **All flows**: An effluent channel, combining all flows, 5.0 m wide and 3.6 m deep (internal dimensions).

The existing effluent channels from Stages 1&2 and Stage 3 will be rerouted to the new drop shaft, and a new effluent channel extending from the Stage 4 chlorine contact tanks will be connected to the drop shaft.

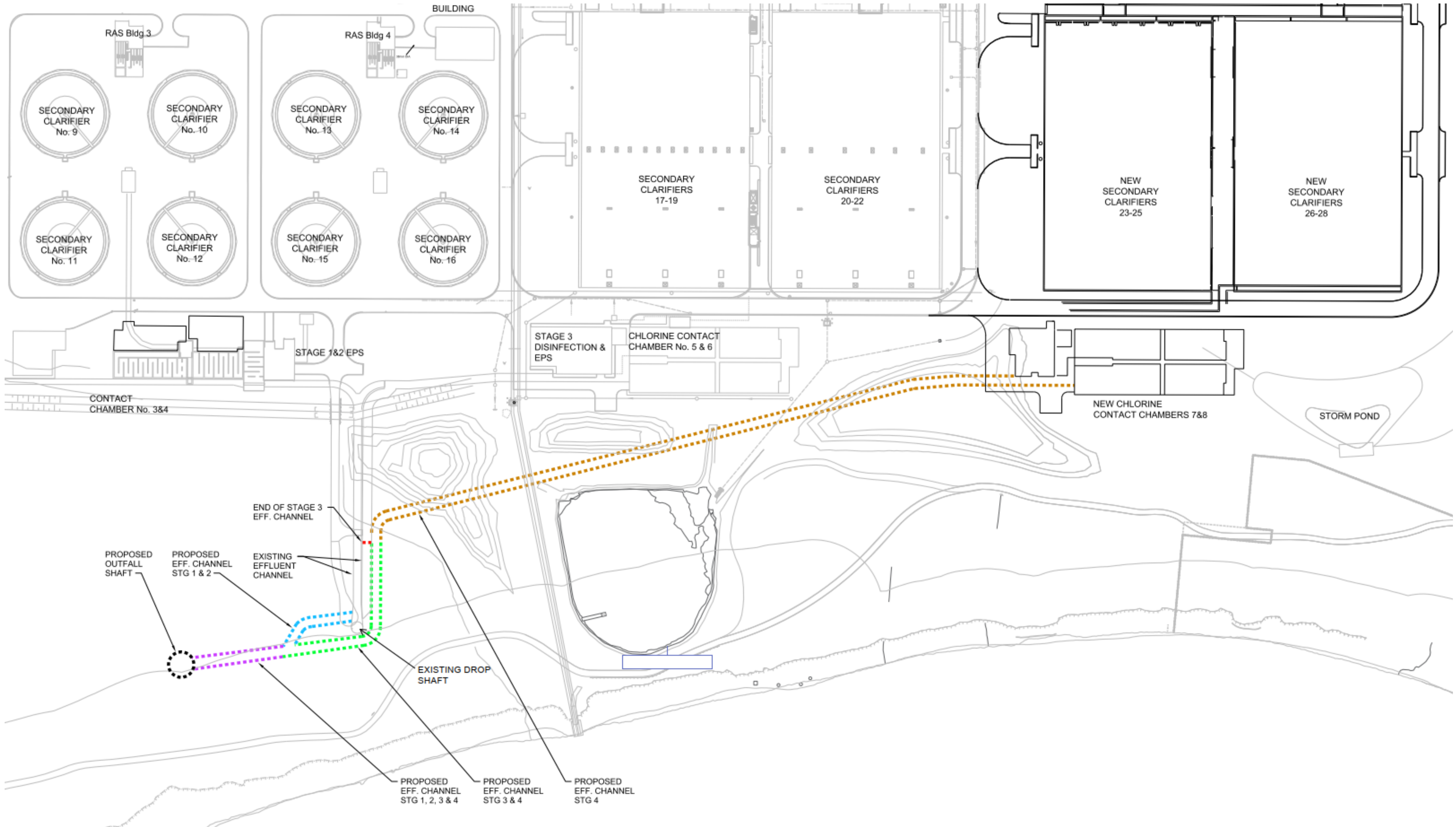

Figure 10.15 **Conceptual Plan of Effluent Channels Connections to Drop Shaft**

10.8.1.1 Effluent Channels Construction

Construction sequencing for the proposed effluent channels will be planned to avoid shutdown of the existing effluent channels. Cofferdams will be used to locally dewater and provide dry work areas within the existing channel in order to install bulkheads and demolish walls to connect and transfer existing flows into the new effluent channels. The use of cofferdams will narrow the channel and locally increase the hydraulic grade line. Work will have to be timed and executed to align with lower flow periods at the plant to prevent overall effects to plant operations.

The following describes the conceptual construction sequence of work to connect and divert flow from the existing Stage 3 effluent channel into the proposed Stage 4 effluent channel shown on Figure 10.15. The sequence of work for connecting the proposed Stage 1&2 effluent channel to the existing effluent channel will be similar.

1. Excavate and construct the new Stage 4 effluent channel starting at the Stage 4 chlorine contact tanks, all the way to the eastern side of the existing Stage 3 effluent channel (shown in yellow), then south and around the existing drop shaft (shown in green) and then west to connect with the new shaft (shown in purple). A new wall opening will be constructed within the new Stage 4 effluent channel at the connection point to the existing Stage 3 effluent channel. The local roof slab above this area will not be constructed so that work in this area can continue. At this time, no demolition will have been done in the existing Stage 3 channel, so effluent flow will continue per normal operations.
2. Leakage test the new Stage 4 effluent channel.
3. Locally remove the full width of the roof slab above the existing Stage 3 effluent channel at the connection point. The contractor is to use pre-installed lifting hooks and other means to remove slab and limit debris into the channel below.
4. Using cofferdams, install wall brackets on both walls of the existing Stage 3 effluent channel downstream of the connection point, which will allow for the insertion of a bulkhead once the connection work is complete.
5. Using cofferdams, locally demolish a portion of the eastern wall of the existing Stage 3 effluent channel at the connection point and install a stop log frame. Insert stop logs to temporarily stop flows from entering the new channel.
6. Construct the new roof slab on the existing and new channel side. The roof slab will have corresponding openings in it to allow for access and installation of bulkheads and stoplogs as necessary.
7. Remove stop logs and insert bulkhead to divert flow into the new Stage 4 effluent channel.

If the existing outfall is planned to be used as backup, a construction strategy to use the existing channels will be defined during preliminary design. Any potential for the use of the existing outfall as a backup would be considered a multi-day major maintenance activity for inspection or maintenance purposes for the new outfall. Because the existing outfall has a peak hydraulic capacity of approximately 1,900 ML/d, and the forecasted flows will be greater when the new outfall is constructed (up to 3,290 ML/d), the use of the existing outfall as a backup would be limited and may require careful timing to correspond with low-flow periods.

10.8.2 Drop Shaft Design Concept

The proposed new drop shaft will serve as the main tunnel mining shaft during construction and as an access shaft thereafter. The completed shaft will convey treated effluent flows from the Duffin Creek WPCP to the outfall tunnel.

During the 2010 Outfall Class EA (detailed in section 10.3), three alternative shaft locations within the onshore area were evaluated. The evaluation criteria included ease of implementation, ability to use existing infrastructure, impacts on terrestrial environment, impacts on surrounding Species at Risk and Significant Habitats, geotechnical impacts, impacts to tourism and recreational uses of lakefront, risks to impact the community health and safety, worker health and safety and capital costs. The relative impacts for each alternative location were quantified. The location situated on the western side, as shown in Figure 10.15, was selected as the preferred shaft location due to its relative ease of implementation, most available space, and its low potential to impact terrestrial wildlife and habitat during construction.

The planned shaft is approximately 65 to 70 m deep, and the inside diameter will be 12 m. The shape of the shaft contributes to optimizing support requirements. Circular excavations require less support because the redistribution of stresses around a cylinder creates compression in the support that is easier to handle.

The shaft design and construction will need to be compatible with the anticipated ground conditions. The shaft will be excavated through approximately 14 m of fill and overburden soils and 40 to 45 m of bedrock. The overburden consists primarily of glacial till. The groundwater conditions will need to be further explored, but the groundwater table is expected to be relatively shallow because of the proximity of Lake Ontario. The bedrock is anticipated at a depth of about 14 mbgs and is expected to consist of two different formations, namely the Georgian Bay-Blue Mountain Formation and the Cobourg Formation.

10.8.2.1 Drop Shaft Construction

Because of the proximity of the shaft to the lake, it is expected that the excavation support of the shaft in the overburden will consist of secant piles. Secant piles are interlocked concrete piles that create a continuous concrete wall and therefore limit groundwater infiltration into the excavation. It is also more rigid than steel sheet piles, limiting ground deformation on the surface caused by the excavations.

Figure 10.16 shows a typical drill rig for auguring of the secant piles and what an arranged configuration within a shaft of secant pile system looks like. The secant piles overlap, creating a continuous concrete support that allows excavation to proceed without active dewatering.

Once a shaft excavation advances to bedrock, the excavation support systems typically used in Ontario are rock dowels, welded wire mesh, and shotcrete, as shown on Figure 10.16. The length of rock dowels depends on the size of the excavation, but also on the discontinuity systems or family of discontinuities that are found in the rock mass. The dowels increase the friction between the potential sliding planes and reduce the probability of falling rock blocks.

Figure 10.16 Secant Pile Drill Rig (left) and Installed Secant Piles (right)

Figure 10.17 advanced in the soil and weathered rock to create a water-tight seal where groundwater infiltration is a concern. The primary piles are unreinforced, and the secondary piles are drilled through the primary piles to create an overlap. The secondary piles have steel reinforcement placed in them for additional support. Within the fresh rock, the excavation support can consist of dowels, welded wire mesh and shotcrete.

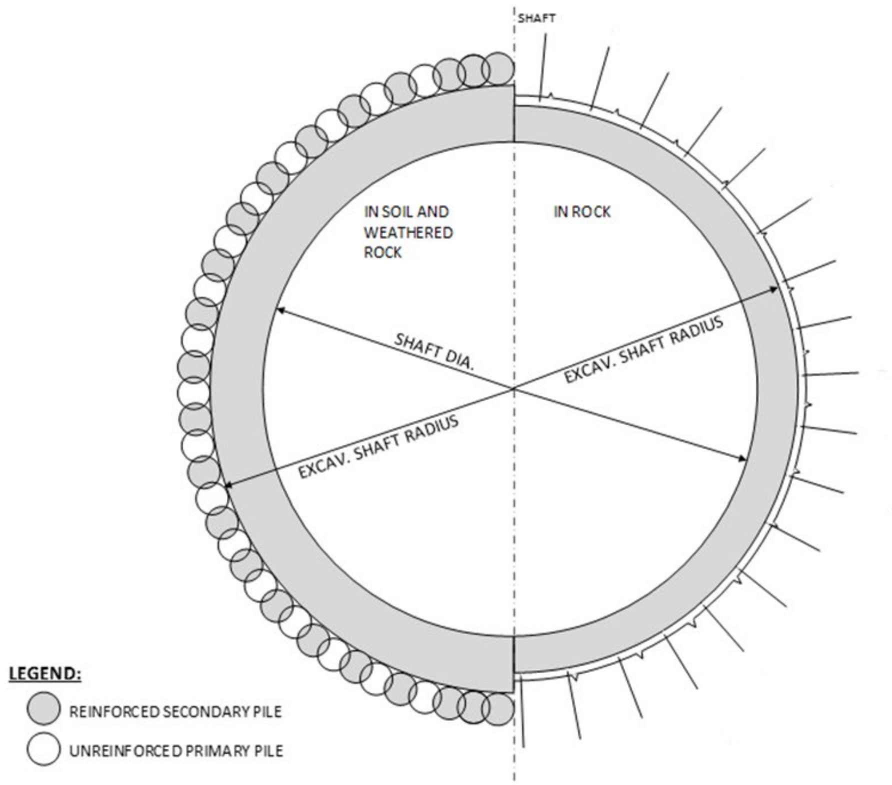

Figure 10.17 Shaft Excavation Support Types

The permanent lining of the shaft will be finished with a 12 m ID, steel-reinforced, CIP concrete lining over its entire depth. The final lining can be cast after the tunnel is completed to remove the risk of damaging the final lining during the outfall tunnel construction. The design of the final shaft liner and cap, including provisions for shaft access, should be determined during detailed design.

10.8.3 Tunnel Design Concept

Multiple factors were considered in selecting the preferred tunnel configuration, such as tunnel diameter, length, horizontal alignment, depth, and permanent lining. Some of these are constraints of the overall system requirements, some depend on the known conditions, and some are based on preventing potential risks from becoming realized.

There is some flexibility in selecting the tunnel depth. The ground conditions along the horizontal alignment will inform the depth of the tunnel, based on the associated risks and construction considerations. A summary of the preferred tunnel concept is provided in Table 10.14. Further detailed descriptions are in the sections that follow. The information gathered during detailed design will support the final selection of the outfall tunnel criteria.

Table 10.14 Tunnel Outfall Design Concepts

Parameter	Design concept
Tunnel depth below ground surface on shore	65 to 70 mbgs
Tunnel length	3,000 m
Tunnel inside diameter	5.5 m
Ground conditions	Rock consisting of limestone, siltstone and shale
Tunnel excavation method	Rock TBM, single or double shield
Excavation support and permanent lining	Precast concrete segmental tunnel liner

10.8.3.1 Tunnel Diameter and Length

Based on the outfall configuration "Option 2", an internal outfall diameter of 5.5 m is needed to meet the hydraulic requirements. The tunnel excavation diameter will be the outfall ID plus the width of the final tunnel lining and any over-excavation mining tolerance during tunnelling. Assuming a final tunnel lining width of about 250 mm and an over-excavation of about 50 mm, the tunnel excavation diameter is assumed to be about 6.1 m. The actual mined diameter will be determined by the contractor within limits set in the contract documents.

The tunnel length depends on the required location of the diffusers into Lake Ontario, the required number of risers, the required spacing, and the overall length of the diffuser system. Based on the outfall configuration "Option 2", the diffuser section will be located approximately 2,500 to 3,000 m offshore in Lake Ontario and will include 34 risers, evenly spaced along the diffuser length (500 m). The tunnel length will need to advance just past the final riser for a total length of about 3,000 m.

Note that more risks are associated with longer tunnel drives. For example, longer tunnel drives will have higher probability of encountering rock valleys, and a longer alignment would extend farther into the dark grey to black shale of the Georgian Bay-Blue Mountain Formation, increasing the risk of encountering gases. A risk evaluation is required during detailed design.

10.8.3.2 Tunnel Alignment

As described in Section 10.3, the western alignment is preferred over the eastern alignment because of the orientation away from the Ajax WSP intake.

The geotechnical conditions along the western alignment have not been explored as much as the conditions along the eastern alignment. However, some assumptions can be made about the western alignment based on the information of the ground conditions along the eastern alignment. For example, two buried valleys have been identified crossing the eastern alignment where the surface of the bedrock extends deeper. These buried valleys are assumed to also cross the western alignment, which will need to be confirmed.

The selected depth of the tunnel must consider the ground conditions along the alignment. It is preferred to have a tunnel with sufficient depth to allow a rock cover above the tunnel of a minimum of 2.5 times the tunnel excavation diameter. This rock cover is taken from the crown of the tunnel to the top of bedrock, where the bedrock at its lowest point along the entire alignment is considered. For this reason, the vertical profile of the tunnel can change based on the results of future investigations and confirmation of the most recent bedrock surface along the preferred horizontal alignment. The minimum rock cover is important for constructability and safety reasons, as the tunnel is being constructed below Lake Ontario. There is also a risk of discontinuities in the rock hydraulically connecting the tunnel to the lakebed sediment, which would cause uncontrolled flow of water from the lake into the tunnel and possibly lead to injuries or fatalities. Contingency plans for mitigating this risk will need to be incorporated in the detailed design and contract documents.

The preferred tunnel profile will have a rock cover of about 14 m at the point of the lowest buried valley, based on the assumed tunnel excavation diameter. Considering the bedrock information that is known at this time, the conceptual depth is in the range of 65 to 70 mbgs. The conceptual profile of the outfall tunnel is provided in Figure 10.18.

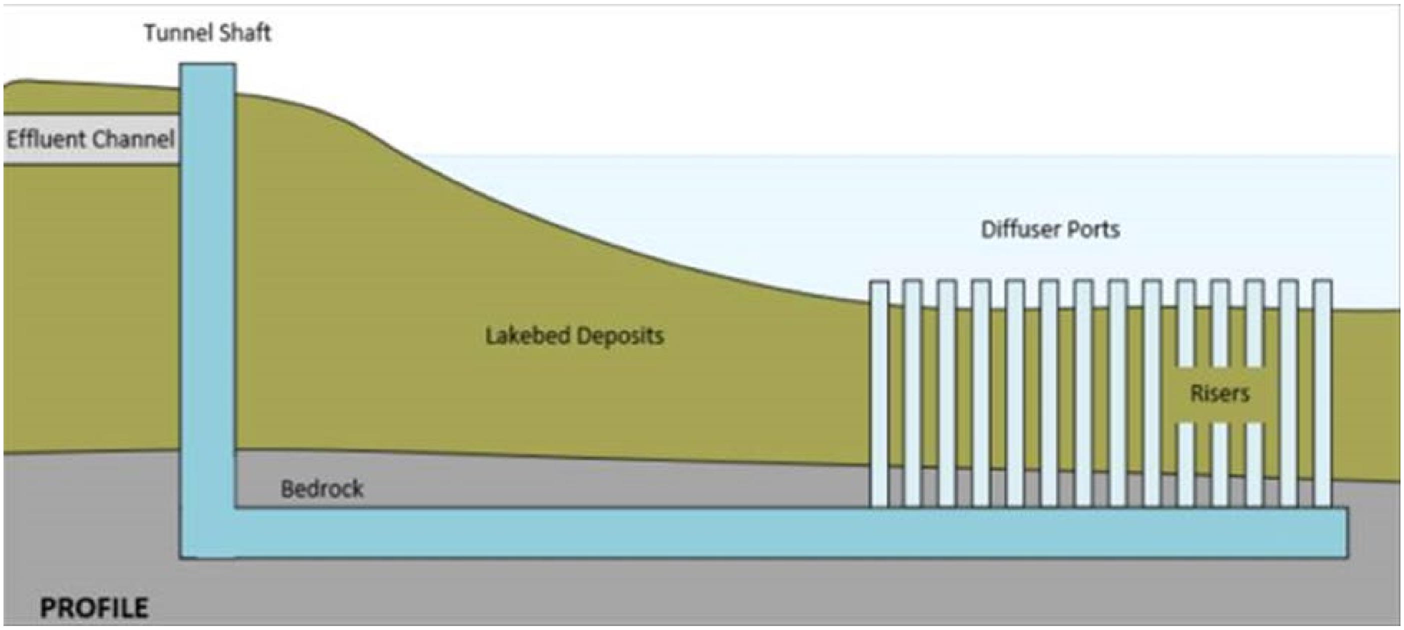

Figure 10.18 Conceptual Profile of Outfall Tunnel

The tunnel could be sloped downwards toward the lake or sloped upwards toward the lake. There are advantages and disadvantages to each approach that must be considered during construction.

If the tunnel slope is upwards toward the lake, any water infiltration will migrate toward the shaft and away from the tunnel face (end of the diffuser section). This will allow water accumulating within the shaft to be pumped up toward the surface. However, should the water within the shaft rise above the tunnel crown (highest point of the arched cross section) in a significant flooding event, the workers within the tunnel would be trapped until the flood waters are lowered sufficiently.

Alternatively, the tunnel can be sloped downwards toward the lake so that water infiltration into the tunnel accumulates at the tunnel face. This slope design would require the water to be pumped at the tunnel face and back toward the shaft and then upwards to the surface. It will require a greater cost to pump the water the length of the tunnel rather than allowing the water to flow toward the shaft, but in the event a significant flood occurs, the workers will be able to safely move away from the accumulating water.

Based on these construction considerations, it is preferred that the outfall tunnel be sloped down toward the lake. Although this alternative will require more energy and costs to pump the water from the tunnel face back toward the shaft, it is a safer alternative during construction. The tunnel slope will be further reviewed during preliminary design.

10.8.3.3 Tunnel Construction

The tunnelling methods to be considered have to incorporate the safety aspect of excavating a tunnel under the lake where no practical access or intervention will be feasible from the lake for equipment repairs or rescue. This implies using mechanized methods that can be repaired from inside the tunnel and installing robust excavation support.

10.8.3.3.1 Tunnel Boring Machines

A tunnel boring machine (TBM) excavates the full face of the tunnel cross-section. The TBM will have a cutterhead at the front of the machine equipped with rock-cutting discs that transfer the thrust of the machine onto the rock, creating fractures and rock chips to break away from the tunnel face (Figure 10.19). Minimum performance-type specifications should be incorporated into the contract documents for the TBM so that the TBM selected by the contractor is compatible with the expected ground conditions and the initial support system.

Figure 10.19 Photo of Rock TBM Facing Cutterhead

Typical rock TBM technologies are main-beam TBMs, or fully shielded TBMs with a single-shield or double-shield arrangement. A summary table of the rock TBM types and brief description is presented in Table 10.15

A main-beam TBM is equipped with gripper shoes that push on the sidewalls of the tunnel to thrust forward. A main-beam TBM requires the rock to be competent and hard because the machine is open to the surrounding rock while the tunnel lining is installed. According to the existing geotechnical information about the bedrock, it is expected that a main-beam TBM might not be compatible with the lower quality and softer rock.

A single-shield TBM provides a shield that protects workers from falling rock until the tunnel lining can be safely installed. The body of the machine is enclosed in a shield that is smaller than the tunnel diameter. As the cutterhead turns, a ring of hydraulic cylinders provides forward thrust through shoes that push against the tunnel lining. The lining consists of PCTL segments that are erected to form a supporting ring that becomes the final lining once the tunnel is finished. While the machine is excavating, the last assembled ring leaves the tail shield, and the annulus is grouted. After each push, a segmental ring is erected, and no excavation can be conducted.

A double-shield TBM has an arrangement of a smaller diameter inner shield that slides within a larger outer tail shield. Behind the forward shield are the grippers that push against the tunnel walls as the main propel cylinders push the cutterhead forward. In this type of machine, the precast concrete segments can be erected within the tail shield while the machine continues to advance forward. If the ground becomes too weak to withstand the gripper shoe pressure, the required thrust is transferred to the tunnel lining by cylinder jacks similar to a single-shield TBM arrangement.

Table 10.15 Rock Tunnel Boring Machine Types

TBM type	Description	Pros	Cons
Main beam	TBM is “open” and equipped with gripper shoes that push on the sidewalls of the tunnel to thrust forward.	Fast production rate.	Risky where low-quality rock is encountered, as there is no protection for workers until the tunnel support is installed.
Single shield	TBM has a shield that protects workers from falling rock until the tunnel lining can be safely installed.	Safer for workers, as they are protected while the tunnel lining is installed.	Slower production rate, tunnel lining is placed sequentially, and TBM cannot advance while lining is placed.
Double shield	TBM has an arrangement of a smaller diameter inner shield that slides within a larger outer shield.	Dual mode operation available, can install tunnel lining while the TBM pushes forward.	Relatively longer shield is more susceptible to squeezing or blocky rock.

In addition to rock TBMs, there are also pressurized face TBMs, such as a slurry TBM or earth pressure balance tunnel boring machine (EPBM, that develop pressure at the face of excavation to support the ground). However, these TBMs are more suitable for tunnelling in soft ground and are usually considered for mixed-face excavations where the tunnel will advance through very soft and extremely weathered rock. However, although not ideal, these TBM types could still be used for the Duffin Creek WPCP outfall, based on the available information regarding the expected ground conditions. The EPBM can excavate rock in non-pressurized mode, and when required, the chamber or plenum behind the cutting head can be pressurized to contain the external pressure. The slurry tunnel boring machine can only operate in pressure mode but could still excavate the shale rock. A more recent TBM development is the multi-mode TBM, which is project-designed to excavate variable ground conditions using pressurized mode in soft ground and open mode in rock more efficiently than an EPBM.

Should there be any risk of a rock TBM crossing a fault or shear zone, or lower quality rock with increased discontinuities that could convey a significant amount of groundwater into the tunnel, a grouting program could be implemented. Grouting from within the TBM would not allow the tunnel to advance forward while grouting. The grout would be placed around the tunnel horizon ahead of the excavation, similar to Figure 10.20.

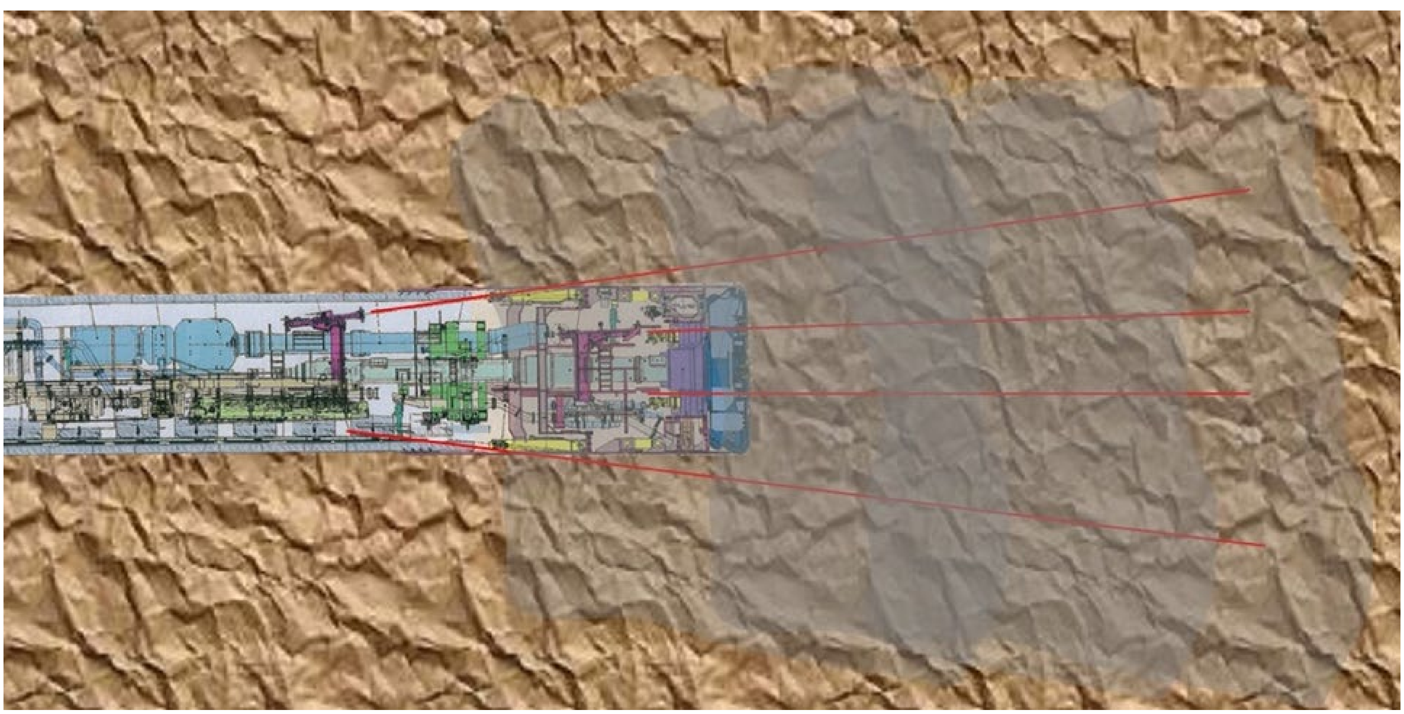

Figure 10.20 Grouting from within the TBM

10.8.3.3.2 Tunnel Lining System

The initial tunnel excavation support depends on the rock quality and ground conditions that are expected to be encountered. The tunnel excavation support needs to be installed immediately behind the TBM so that rock in the crown is not given more opportunity than necessary for uncontrolled overbreak to occur, which creates a safety concern for workers. The final tunnel lining can be installed as CIP concrete against the excavation support after the tunnel excavation is finished, which is referred to as a two-pass system. Alternatively, and as mentioned before, when the excavation support becomes the final tunnel lining, it is referred to as a one-pass system. A brief description of each of these systems is provided as follows.

One-Pass Systems

The term one-pass system refers to the excavation support installed during excavation of the tunnel that becomes the final lining during operation. For tunnel diameters in the range of what is being considered for this project, that is between 5 and 6 m ID, the final lining will be the PCTL. The segments are usually between 1 and 2 m wide, depending on the tunnel diameter and alignment, and are installed in the back of the TBM forming a ring; several rings side by side form the tunnel lining. The annulus between the excavated surface and the segmental rings is grouted immediately as the ring is ejected out of the trailing shield of the TBM. Figure 10.21 provides sample images of Precast Concrete Tunnel Lining.

Figure 10.21 Precast Concrete Tunnel Lining

One-pass has proven to be successful in water conveyance projects and was used in the Ashbridge's Bay Outfall (Hatch/Jacobs, 2017). PCTL is an excellent alternative when high-quality control is maintained during manufacturing and installation and is recommended for the Duffin Creek WPCP Outfall.

Two-Pass Systems

A two-pass system involves installing an initial ground support system, such as shotcrete, rock dowels, and welded wire mesh, immediately behind the TBM to stabilize and support the tunnel excavation. The initial support is followed by a final lining that is installed typically when the excavation is finished. The final lining for the diameter that is being considered for this project is often a CIP concrete. After concrete placement, holes are drilled into the crown to conduct contact grouting through. The contact grout fills any remaining voids behind the lining and assures even load transfer around the full lining support.

10.8.4 Diffusers Design Concept

Based on Option 2, the outfall diffuser section will start approximately 2,500 m offshore in Lake Ontario and will include 34 risers, in line with the tunnel, evenly spaced along the 500 m diffuser length. The diffuser system consists of the tunnel risers, tunnel-riser connections, carrier pipes, casings, ports, annulus grout, and lakebed scour protection/armouring.

The configuration will consist of 1 m ID stainless steel riser pipes, which connect the outfall tunnel to above the lakebed. A stainless steel reducer connects the pipe above the lakebed to the diffuser port. Diffuser ports can be manufactured from many different materials, including fibreglass reinforced polymer, high-density polyethylene, steel, and so on. Material selection should be made during preliminary design. However, recent similar projects have used fibreglass reinforced polymer and high-density polyethylene diffuser ports because these materials are relatively inexpensive to replace if damaged and do not corrode.

The riser height, the depth of overburden, and the depth of rock vary as the riser's progress offshore. Table 10.16 summarizes the range of general riser configurations.

Table 10.16 Diffuser Riser Design Concepts

Parameter	Design concept
Riser height	25 to 30 m
Riser excavation diameter	1.5 m
Lake water depth to lakebed	20 to 25 m
Depth of overburden to rock	3 to 6 m
Depth of rock to tunnel crown	12 to 17 m

The range is provided for conceptual considerations only. The depth of the water along the diffuser section, along with the depth of overburden and rock, will need to be confirmed during preliminary design after the subsurface investigation is complete. The elevation, heights, and other dimensions of individual riser pipes will be established in preliminary design.

10.8.4.1 Diffusers Construction

Riser shafts will be excavated from a barge fixed to the lake bottom, and the drill rig will advance through the lakebed sediments, the underlying bedrock, and into the tunnel horizon. Once complete, the bored holes for the tunnel risers will be backfilled to about 1 m above the planned tunnel crown with granular material. A steel casing will be set on the granular material and grouted in place. This casing will be closed at each end and fitted with two valves.

The main sequence for safe installation of the drilled casings, risers, and ports is as follows:

- Install casing through overburden and embed into bedrock.
- Install riser within the casing and grout the annular space around the riser.
- Pressure test the riser and grout seal.
- Excavate the 5.5 m ID outfall tunnel below the risers.
- Mine the tunnel to riser connection adit from within the outfall tunnel.
- Drain the riser until empty and close the top valves.
- Remove bottom riser valves, install a plug, and line the riser adit section.
- Flood the tunnel, open the top riser valves, install ports and armouring.

Upon placement and installation of risers in the drilled holes, the risers must be grouted along their entire depth in place. The bottom valves and the grout pipes will be protected against damage. Leak tests must be performed to confirm the effectiveness of the grout seal. During detailed design, an annulus grouting performance specification will be developed and will require the contractor to submit a grouting methods statement.

10.9 Environmental and Community Impacts and Mitigation

The Duffin Creek WPCP Expansion project will potentially have impacts on the social, built, natural and cultural environments. Desktop studies were done to determine the possible extent of these impacts and to propose mitigation measures that would reduce the likelihood and extent of the impacts should they occur. Table 10.17, Table 10.18 and Table 10.19 summarize the potential impacts on the social and built environment, the natural environment and the cultural environment and recommend mitigation measures that can be adopted during design, construction and operations.

Table 10.17 Social and Built Environment – Potential Effects and Mitigation

Item #	Criteria	Indicators	Potential Effects (Positive/Negative)	Avoidance/Mitigation/Compensation
SB-1	Effect of noise on sensitive receptors – construction	Number of sensitive receptors affected and extent and duration of adverse effects during construction. Impacts as a result of construction	– Environmental noise may cause sleep disturbance and general annoyance. The magnitude of the noise disturbance depends on the number of pieces of equipment, their proximity to each other and to sensitive receptors, construction methods, equipment deployed, construction hours, and duration of exposure to sensitive receptors.	Construction noise impact mitigation measures include, but are not limited to, the following to meet applicable noise criteria: – Site construction staging to reduce adverse impacts to sensitive receptors where possible. – Use construction equipment compliant with noise level specifications in MECP guidelines NPC-115 and NPC-118. – Keep equipment in good working order and operate with effective muffling devices where possible. – Use acoustic enclosures for equipment such as generators and compressors. – Use localized, movable noise barriers/screens for specific equipment and operations. – Minimize simultaneous operation of equipment where possible, particularly noisy sources. – Implement a no idling policy onsite (unless necessary for equipment operation). – Restrict construction hours where possible: • Perform construction during daytime hours when possible. If nighttime construction is necessary, high-noise activities should be restricted to daytime when possible. – Consider operational duration limits for construction. – Inform local residents before construction of the type of construction and expected duration if occurring outside of typical daytime hours (7 a.m. to 7 p.m.). – Limit the number of heavy trucks onsite to the minimum required. where possible – Stage construction vehicles away from noise-sensitive locations, where possible – When construction location and design are better known, establish and apply project-specific construction noise criteria/exposure limits. – Undertake noise monitoring throughout the construction phase. Where noise level limits are exceeded, additional noise mitigation measures will be considered. – Consider developing a communications protocol that includes timely resolution of complaints. – Consider additional mitigation measures not listed herein as construction progresses.
	Effect of noise on sensitive receptors – Operations	Number of sensitive receptors affected and extent and duration of adverse effects during construction. Impacts as a result of construction	– None anticipated	– None planned

Item #	Criteria	Indicators	Potential Effects (Positive/Negative)	Avoidance/Mitigation/Compensation
SB-2	Effect of air quality on sensitive receptors – Construction	Number of sensitive receptors affected and extent and duration of adverse effects during construction	– Potential air quality impacts caused by dust and odour from diesel combustion and particulate emissions. – Exhaust emissions from construction vehicles may contribute to increased levels of criteria air contaminants. – Some construction activities are likely to have higher dust emissions, which include earthworks activities, demolition activities, travel on dusty or unpaved surfaces with heavy equipment travel, and erosion from uncovered soil storage piles.	– Site construction vehicle activity will be managed to control emissions of odorous contaminants and diesel exhaust. Mitigation measures that can be followed include the following: • All vehicles are fuel efficient • Proactive identification of emission sources • Equipment maintenance program • Water application to roads during dry periods. – Mitigation measures consistent with ECCC's Best Practices for the Reduction of Air Emissions from Construction and Demolition Activities (Cheminfo Services Inc., 2005), and MECP's Technical Bulletin Management Approaches for Industrial Fugitive Dust Sources, will be followed. – The following mitigation measures can be considered in the Air Quality Management Plan: • All equipment complies with Canadian engine emissions standards. • All equipment visually inspected prior to use and properly maintained in accordance with the manufacturer's manual. • Landscaping materials ordered close to time of use to reduce onsite storage. • Minimize drop height of materials onsite. • Covering surface area of hauled bulk material. • Methods and equipment for cleanup of accidental spill of dusty materials. • Implement a no idling policy onsite (unless necessary for equipment operation). • Use of electricity from the grid over diesel generators wherever possible. • Retrofitting of combustion engines with specific exhaust emission control measures, such as particulate traps. • Application of soil stabilizers or dust control polymers where feasible. • Daily removal of accumulated mud, dirt and debris deposits onsite, and regular truck washing. • Paved and unpaved roadway cleaning, watering or application of acceptable dust suppressants. • Complete earthwork grading within 10 days of ceased active construction. • Temporary seeding or mulching of bare soil and storage piles. • Compression or clodding of soil surfaces and storage piles to reduce erosion. • Confine storage pile activity to downwind side of piles. • Reduction of activities during high wind conditions. • Full or partial enclosure of demolition activities. • Wind screens or barriers where possible or necessary. • Offsite construction of certain structures or parts of structures to minimize air emission due to interference with the normal flow of traffic. • Scheduling certain construction activities (i.e., site preparation and earth works activities, demolition activities, unpaved surfaces with heavy equipment travel, and uncovered soil storage piles) to periods of time when exposure to dust is expected to be limited (for example, avoid scheduling activities during dry, windy weather conditions). • Limit travel speeds onsite to a maximum of 20 km per hour. • Visually monitor for dust during construction. • With a suitable instrument, monitor for fine particulate when construction boundary is within 15 m of a residence. • If disruption of contaminated soils is anticipated at any time, consult with the construction manager to prevent release of harmful or volatile contaminants. • Consider developing a communications protocol which includes timely resolution of complaints.

Item #	Criteria	Indicators	Potential Effects (Positive/Negative)	Avoidance/Mitigation/Compensation
	Effect of air quality on sensitive receptors – Operations	Number of sensitive receptors affected and extent and duration of adverse effects during operations	– None anticipated	– None planned
SB-3	Effect of odours on sensitive receptors – Construction	Number of sensitive receptors affected and extent and duration of adverse effects during construction	– Limited odours emissions of volatile organic compounds from diesel fuel combustion – Overall odour emissions are not expected to be significant as a result of Project construction activities.	– Employ portable odour control device as necessary, such as a misting device or portable activated carbon control unit. Choice to be determined during the design stage based on details of the potential odour emissions.
	Effect of odours on sensitive receptors – Operations	Number of sensitive receptors affected and extent and duration of adverse effects during operation	– None anticipated	– None planned

Table 10.18 Natural Environment – Effects and Mitigation

Item #	Criteria	Indicators	Potential Effects (Positive/Negative)	Avoidance/Mitigation/Compensation
N-1	Effect on groundwater	Temporary changes in groundwater quantity and quality	– The lowering of the shallow groundwater level due to construction dewatering could potentially reduce the groundwater input into nearby groundwater dependent features. – Dewatering discharge that may be directed to nearby tributaries could potentially alter the physical, chemical and thermal regimes of the receiving streams. – Potential for ground settlement resulting from construction dewatering where the estimated drawdown is significant and compressible soils are located within the zone of influence of the dewatering. – Changes in groundwater-surface water interaction (reversal of vertical hydraulic gradient) results in impact to terrestrial and aquatic habitat and associated SAR (where applicable) – reduction in baseflow. – Potential effects on groundwater water quality as a result of potential mobilization of contaminated water where active dewatering/depressurization is required. – Reduction in groundwater quality from spills or the mismanagement of fuel/chemical in work areas. – Sump and Excess Process (SEP) water and dewatering discharges at the drop shaft impacts surface water quality and quantity.	– Where dewatering is anticipated, an assessment of the potential for settlement will be required. – Monitoring and contingency plans are required to be prepared as part of the hydrogeological field investigation to identify, minimize, and mitigate potential impacts to nearby potential receptors, including Lower Duffins Creek Wetland Complex and Duffins Creek Coastal Marsh. – Discharge to the natural environment may require approval by MECP, MNRF, TRCA and/or others depending on the location and proximity to TRCA-regulated areas.
N-2	Effect on soils	– Area of erosion and sedimentation during construction – Area of contaminated soils	– Dust and sediment can be created during construction of staging areas and access roads. – Contaminated soils may be encountered during construction.	– Install sediment traps to deal with storm runoff during construction, where appropriate. – Install silt fences along the perimeters of the construction staging areas where appropriate to manage erosion by retaining soil within disturbed land. Watering will also be considered. – Cover exposed excavated material to prevent erosion by rain/wind. – Drop-in filter bags should be utilized in catchbasins during construction to prevent migration of sediments to receiving watercourses and from entering the storm sewer system, where necessary. – Remove sediment from paved roads and access points. – Tarp, monitor, and clean trucks transporting soil, waste, or granular material. – Test soils to determine the type of contaminant. Discharge contaminated soils at designated locations. – Re-integrate uncontaminated excess soils (berms) into the project when possible.

Item #	Criteria	Indicators	Potential Effects (Positive/Negative)	Avoidance/Mitigation/Compensation
N-3	Effect on surface water	Temporary change in surface water	– Changes to surface water bodies are expected to be minimal, as tunnelling is the recommended construction method. – Erosion and sedimentation due to run-off in construction areas. – Decrease of depletion of surface water due to active dewatering or water migration through ground into excavations. – Excavations close to surface water bodies have the potential to cause adverse impacts to aquatic ecosystems. – Water may enter the shaft during construction due to dewatering activities in proximity to waterbodies. – Dewatering may also be required due to groundwater intrusion depending on the depth to water table. – Change in surface water temperature from groundwater taking and/or discharge to surface water features. – Changes to stream morphology resulting from the release of groundwater dewatering water. The potential reduction in baseflow due to water taking in a lower confined aquifer due to increased downward hydraulic gradients across the aquitard separating the stream and the confined aquifer. – The potential reduction in baseflow from a stream reach that intersects an aquifer in which the water taking is occurring. – Sump and Excess Process (SEP) water and dewatering discharges at shaft sites impacts surface water quality and quantity	– Install preventative straw bales crossing the water flow downstream of the tunnel to contain any spill. This could cause some retention of water in which case the straw bales can be keep on site to be installed only in case of a spill. – Pump the slurry from within the watercourse using a vac-truck on site before it spread along the river. – Install proper erosion and sedimentation measures, such as silt fences, silt socks, and implement Best Management Practices. To limit suspended solids and mitigate/avoid potential impacts. – Avoid active dewatering, specify sealed excavation support systems for the shafts to minimize impacts on water bodies.
N-4	Effects on Aquatic habitat – Shorelines	Temporary or permanent loss of aquatic features or categorical loss of functions by type, including wetlands, watercourses by sensitivity type, and others – Shoreline	– Construction may cause shoreline disturbance or erosion.	– Construction activities will maintain the buffers established during the design phase to reduce potential negative impacts. Shorelines or banks disturbed by construction activities will be immediately stabilized by any activity associated with the Project to prevent erosion and sedimentation through revegetation with native species suitable for the site. – In-water construction timing windows and other mitigation measures as required by Fisheries and Oceans Canada (DFO) and TRCA need to be adhered to. – Onsite inspection will confirm implementation of the mitigation measures. Corrective actions, if required, may include additional site maintenance or altering site activities to reduce impact.

Table 10.19 Cultural Environment – Potential Effects and Mitigation

Item #	Criteria	Indicators	Potential Effects (Positive/Negative)	Avoidance/Mitigation/Compensation
C-1	Effects on known or potential significant archaeological resources	– Number and type of known archaeological site affected. – Extent of affected area within potential archaeological sites.	– Potential impacts on archaeological resources during construction.	– Stage 1 Archaeological assessment would be required should works extend beyond the current study area.

10.10 Capital Cost Estimate

The cost estimate methodology and the estimate basis are from the Association for the Advancement of Cost Engineering International (AACEI) methodology and represent a Class 5 cost estimate with an accuracy of -50% to +100%. The estimate reflects the probable cost obtained for the Greater Toronto Area and is a determination of fair market value for the proposed scope of work. Allowances and markups were also included in the estimate for additional items, such as design contingency, construction contingency, and future investigations.

10.10.1 Scope of Work

The design concept for the Duffin Creek WPCP outfall consists of the following major components:

– **Drop Shaft**: 12 m ID launch onshore shaft.

– **Tunnel**: Single 5.5 m ID tunnel constructed in bedrock, extending approximately 3,000 m straight out from the launch shaft beneath Lake Ontario.

– **Diffuser**: 34 risers (1 m ID) with diffuser ports (730-millimetre ID), constructed inline with the tunnel at equal spacing along a 500 m length diffuser section (starting 2,500 m offshore), extending vertically from the tunnel to the lakebed.

10.10.2 Cost Assumptions

The estimate is based on the assumptions that the work will be tendered on a competitive bid basis and that the construction contractor will have a reasonable amount of time to complete the work. Other assumptions include the following:

– Because of limited information at this conceptual design stage, the prices used are based on similar projects and/or conceptual drawings.

– Some materials are based on vendor quotes or historical data from past or recently tendered similar projects, with allowances for installation based on ratios of the material cost.

– Quotes from vendors are budgetary only.

– There is no allowance for rock excavation, which is included in the excavation costs of the shaft.

– An allowance of 15% design contingency is considered to cover design and pricing unknowns in the preparation of this estimate. The allowance is not meant to cover additional scope of work or quality modifications, but rather to provide some flexibility as the design develops. The design allowance typically decreases as the design progresses and is a nominal percentage at the pretender stage.

– An allowance of 10% construction contingency is considered to cover the unexpected increase in costs or unforeseen site conditions resulting from design modifications during the construction phase.

– An allowance of 2% is considered for the cost of future investigations.

The cost estimate excludes the following costs:

1. Market contingency
2. Non-construction costs for the following items:
    a. Design
    b. Services during construction
    c. Legal
    d. Owner administration costs
3. Any unforeseen significant increase in material prices
4. Unavailability of materials and skilled labour
5. Accelerated or delayed schedule

6. Overtime premium
7. Applicable taxes
8. Escalation
9. Permits and approvals (summarized in section 10.11.2)

10.10.3 Cost Estimate

Table 10.20 summarizes the overall opinion of probable construction cost for the Duffin Creek WPCP outfall project. The estimated construction cost is $259 million (M), with a probable cost ranging from $130 M (-50%) to $518 M (+100%).

Table 10.20 Estimated Construction Costs

Low range (-50%) ($, excluding HST)	Estimated costs ($, excluding HST)	High range (+100%) ($, excluding HST)
129,500,000	259,015,000	518,000,000

Table 10.21 summarizes the total project cost including construction, design contingency, construction contingency, geotechnical investigations, and allowance for future investigations (summarized in section 10.11.1). The total project cost is estimated at $318 M.

Table 10.21 Estimated Capital Costs

Item	Description	Amount
1	Outfall and risers cost	182,850,000
2	Site restoration cost	2,100,000
3	Miscellaneous cost[7]	22,739,000
4	Construction contingency (10% of items 1-3) and cash allowance	21,500,000
5	Design contingency (15% of items 1-3)	29,826,000
6	Total construction cost estimate	259,015,000
7	Engineering (15%)	38,852,250
8	Future investigations 2% (subsurface utility engineering, and other surveys)	5,180,300
9	Geotechnical investigations	14,900,000
10	Total project cost estimate (rounded off to nearest million)	317,948,000

[7] Miscellaneous work include mobilization, demobilization, surveyor, pre-construction assessment, erosion and sedimentation control, traffic management, soil disposal, records drawing, and so on.

10.11 Implementation Plan

10.11.1 Field and Desktop Investigations

The conceptual planning of the Duffin Creek WPCP was based on desktop review of available information. Field investigations are required prior and during the design stage to ascertain factual data required for detailed design, which would either confirm or modify design concepts. Table 10.22 lists what is anticipated to be the required future investigations.

Table 10.22 Proposed Field and Desktop Investigations

Field Investigation	Comments
Geotechnical	Subsurface investigations are required to characterize the geotechnical conditions for the onshore drop shaft and offshore portion of the outfall between the shoreline and the diffuser: – 17 offshore boreholes drilled with about 200 m intervals. – 1 onshore deep borehole with one monitoring well. – 1 onshore shallow borehole with three monitoring well screened in the Lower Oak Ridges Moraine Aquifer Complex, Thorncliffe, and Scarborough Hydrostratigraphic units (aquifers). – Additional borehole locations may be required at the interface of two rock formations or where dubious conditions are identified.
Hydrogeological	To confirm initial findings and provide hydrogeological information for design: – Install four nested monitoring wells near the drop shaft to confirm the hydrogeologic characteristics of the subsurface and assess potential dewatering requirements and design considerations. – Packer testing to characterize the bedrock hydraulic conductivity. – All groundwater samples will be analyzed and compared against York/Durham Region's Storm and Sewer Use By-law, Ontario's PWQO, and O. Reg. 153/04 (as amended) criteria to determine options for dewatering discharge.
Environmental Investigation	To provide due diligence subsurface and preliminary soil and rock quality information for management of excess soil during shaft and tunnel construction: – Soil bulk samples will be analyzed for the following parameters: • Metals and inorganics • Polycyclic aromatic hydrocarbons • Polychlorinated biphenyls • pH • Petroleum hydrocarbons F1 through F4, including benzene, toluene, ethylbenzene, and xylene • Sodium adsorption ratio and electrical conductivity • Volatile organic compounds – Leachate analysis will be completed on at least 10% of the soil samples for certain contaminants in accordance with O. Reg. 406/19 requirements.
Stage 1 Archaeology	– Stage 1 Marine and Terrestial Archaeological Assessment completed in June 2023.
Stage 2 Archaeology	– Not required.
Tree Inventory and Natural Environment Studies	Arborist inventory and Arborist Report to determine required tree removal and compensation plantings. – Additional breeding bird surveys and amphibian night surveys. – Ecosystem compensation for vegetation removal in TRCA-regulated areas in accordance with TRCA's Ecosystem Compensation Guidelines.

Field Investigation	Comments
Receiving Water Impact Assessment	– Conduct additional lake modelling to optimize the final riser/diffuser configuration required to meet regulatory requirements. – Additional lake modelling during preliminary/detailed design can consider either the effluent concentration objectives or limits, as appropriate, to provide additional information on water quality impacts and inform the final outfall design.
Hydraulic Modelling and Analysis	– Update hydraulic model and analysis to align with the optimized outfall configuration based on the results of the receiving water impact assessment. – Optimize tunnel diameter. – Conduct a Computational Fluid Dynamic model to assess solids transportation and settlement within the effluent channels and the outfall components.
Shoreline Hazard Study	– A qualified coastal engineer will be retained to carry out a shoreline hazard study to accurately delineate the shoreline hazard limit. The approach is to be informed by TRCA The Living City Policies.

Note:

O.Reg. = Ontario regulation

10.11.2 Permits and Approvals

Various federal and provincial legislations and policies and municipal by-laws govern the planning, design, construction, and operation of the Duffin Creek WPCP. A number of permits and regulatory approvals will be required for the new outfall to conform with engineering design standards, health and safety best practices, and environmental regulations.

Table 10.23 and Table 10.24 identify the agencies and municipalities to be consulted and the permits and approvals that may be required in support of the onshore and offshore subsurface investigations and the outfall construction project. The anticipated permits are based on a concept level of the proposed upgrades and will need to be confirmed as part of the detailed design and preconstruction stages.

10.11.2.1 Summary of Permits and Approvals Schedule

A summary of milestones for all the required permits and approvals are presented in Figure 10.22 The majority of the permits and approvals will be obtained prior to tendering, except for those that need to be applied by the Outfall Contractor prior to construction.

In addition, the permits and approvals that are only applicable to the marine works will be obtained prior to in-water construction, but consultation with the agencies will be initiated during detailed design.

Table 10.23 Approvals, Permits and Consultation for Subsurface Investigations

Regulatory agency	Permit/Approval/Notification	Assumed minimal approval timeline	Applicability (on-shore/off-shore)
Toronto and Region Conservation Authority	Development, Interference with Wetlands and Alterations to Shorelines and Watercourses Permit	1 month	On-shore
City of Pickering	Sanitary/Storm Sewer Discharge Permit	2-4 months	On-shore
	Road Occupancy Permit	0.5 month	On-shore
	Noise Exemption Permit	1-month	On-shore
	Tree Cutting Permit	1-2-month	On-shore
Utility Authorities	Utility Service Clearances	To be determined	On-shore

Regulatory agency	Permit/Approval/Notification	Assumed minimal approval timeline	Applicability (on-shore/off-shore)
Ministry of Environment, Climate and Parks	Environmental Activity and Sector Registry or Permit to Take Water	6-12 months	On-shore and off-shore
	SAR Consultation and Species at Risk Permit	12 months	On-shore and off-shore
Ministry of Nature, Resources and Forestry	In-Water Construction Timing Window	1-3 months	
	LOA or Public Land Permit	2-3 months	Off-shore
	Blanket Drilling License	1-3 months	Off-shore
Ministry of Labour	Notice of Project	To be determined	On-shore and off-shore
Ministry of Citizenship and Multiculturalism	Archaeological Assessment Clearance Letter	To be determined	On-shore and off-shore
Department of Fisheries and Ocean	Project Authorization Fisheries Act Authorization and/or SARA permit	2-5 months	Off-shore
Transport Canada (Navigate Water)	NPP Approval	1-2 months	Off-shore
Environment and Climate Change Canada	Canadian Wildlife Service Approval	1-2 months	On-shore

Table 10.24 Summary of Approvals, Permits and Consultation for Design and Construction

Regulatory agency	Permit/Approval/Notification	Assumed minimal approval timeline
Toronto and Region Conservation Authority	Development, Interference with Wetlands and Alterations to Shorelines and Watercourses Permit	1 month
City of Pickering	Sanitary/Storm Sewage Discharge Agreement	2 - 4 months
	Site Plan Approval Building Permit	2-month
	Noise Exemption Permit	1 month
	Tree Cutting Permit	1-2 month
	Road Occupancy Permit	0.5 month
Utilities Authority	Utility Service Clearances	To be determined
Ministry of Environment, Climate and Parks	Environmental Compliance Approval – Sewage	6-12 months
	Environmental Activity and Sector Registry or Permit to Take Water	6-12 months
	SAR Consultation and Species at Risk Permit	12 months
	Excess Soil Regulation Consultation	To be determined

Regulatory agency	Permit/Approval/Notification	Assumed minimal approval timeline
Ministry of Nature, Resources and Forestry	LOA or Public Land Permit	2-3 months
	Crown Land Authorization	2-5 months
	Blanket Drilling License	1-3 months
Ministry of Citizenship and Multiculturalism	Archaeological Assessment Clearance Letter	To be determined
Ministry of Labour	Notice of Project	To be determined
	Notice of Diving Operations	To be determined
	Notice of Tunnels, Shafts, Caissons and Cofferdams	To be determined
	Confined Space Entry Permit	To be determined
Department of Fisheries and Ocean	Project Authorization	2-5 months
	SARA Permit	3 months
	In-Water Construction Authorization	1-2 months
Environment and Climate Change Canada	Migratory Bird Timing Window	To be determined
	Mitigation Measures for Deleterious Substances Permit	1-2 months
	Species at Risk Act (SARA) Permit	3 months
Transport Canada (Navigable Water)	NPP Approval	1-2 months

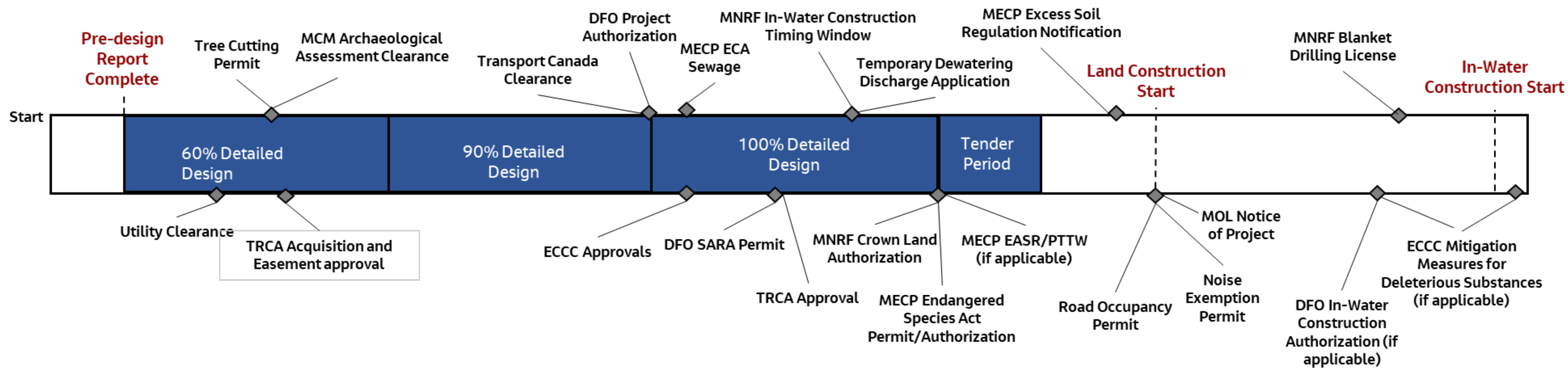

Figure 10.22 Summary of Schedule for Various Permits and Approvals for Outfall Construction and Operation

10.12 Schedule

The implementation plan for the outfall is defined by when the new outfall must be in service. The timing of when the existing outfall will reach its hydraulic capacity (1,900 ML/d) depends on the peak instantaneous flow through the plant.

The current rated ADF capacity (630 ML/d) is forecast to be reached by 2036. The expanded Duffin Creek WPCP will provide a rated ADF capacity of 940 ML/d and a peak flow capacity of 3,290 ML/d. The peak flow capacity is based on the ultimate peak hydraulic capacity of the Stages 1&2 and Stages 3&4 influent pumping stations (that is, a peak instantaneous flow [PIF] peak factor of 3.5).

Table 10.25 summarizes the timing for the next outfall based on several PIF scenarios through the plant.

Table 10.25 Sensitivity Analysis for Future Outfall Requirement

PIF factor	Equivalent ADF capacity of outfall, ML/d[8]	Year when outfall capacity is reached
3.0	630	2038
3.5	540	2031
4.0	473	2030

A peak flow statistical analysis based on historical flow (2016 to 2022), shows that a peak factor of 3.0 represents the 99.96 percentile of PIF data. This means that peak flows exceeding a peaking factor of 3.0 occurred for a theoretical total of 4 hours per year.

It is worth noting that historical and future climate trends for York Region show an increase trend on rainfall intensity, duration and total precipitation (Fausto et al. 2015). Significant peak flow events have been observed at the plant (for example, a 50-year rainfall on January 11, 2020) resulting in PIF peak factor of 3.99.

The new primary trunk, expected to be in operation by 2031, will provide a nominal hydraulic capacity of 3,280 ML/d. Considerations for flow management strategies will be developed during preliminary design of the primary trunk project, which is further described in Chapter 8 of the Project Report.

For this planning phase, the implementation plan was based on the estimated flow projections and a peak flow peak factor of 3.0, meaning that a new outfall will be needed by 2038, which aligns with the Stage 4 expansion requirement (that is, 2036).

The design of the new outfall and shaft will be completed following these sequential tasks:

- Subsurface investigations
- Outfall preliminary design
- Outfall detailed design
- Outfall construction permits and approvals
- Outfall tender period
- Outfall construction.

Durations and sequencing are presented in a Gantt chart on construction schedule was based on experience from recent related projects and also considering the MNRF and DFO in-water construction periods. As shown in Table 10.26, design and construction of the outfall is anticipated to be completed over a 13-year period. If the outfall is required by 2038, subsurface investigation should start by 2025. The symbol X in Table 10.26 denotes the project stage duration.

[8] Based on outfall hydraulic capacity 1,900 ML/d.

Table 10.26 **Outfall Project Implementation Schedule**

Activity	Duration (years)	1	2	3	4	5	6	7	8	9	10	11	12	13
Subsurface Investigation	5	x	x	x	x	x								
Preliminary Design	1						x							
Detailed Design	1							x						
Outfall Tender Period	1								x					
Permits and Approvals	2							x	x					
Outfall Construction	5									x	x	x	x	x

10.13 References

Baird & Associates. (2023, September 14). Duffin Creek WPCP Receiving Water Impact Assessment.

Brater , E. F., King, H. W., Lindell, J. E., & Wei, C. Y. (1996). *Handbook of Hydraulics.*

Chin, D. A. (n.d.). *Water-Resources Engineering.* (T. Edition, Ed.) Pearson.

MECP. (1994a). *Water management: policies, guidelines, provincial water quality objectives.* Ontario: Ministry of the Environment, Conservation and Parks.

MECP. (1994b). *B-1-5 Deriving Receiving Water Based Point Source Effluent Requirements for Ontario Waters.*

Stewart, T. J., Todd, A., & LaPan, S. (2017). Fish community objectives for Lake Ontario. Retrieved from http://www.glfc.org/pubs/FisheryMgmtDocs/Fmd17-01.pdf

ghd.com

jacobs.com