MV2016L3-0001

FORT PROVIDENCE WATER TREATMENT PLANT OPTIONS ANALYSIS REPORT -FINAL-

Project No. IHFP-S1309

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1 INTRODUCTION

ARKTIS Solutions Inc. (ARKTIS) was commissioned by the Incorporated Hamlet of Fort Providence (the Hamlet) to undertake a condition assessment and performance test of the existing wet well located on the north shore of the Mackenzie River at the Fort Providence water treatment plant (WTP); and to develop options for potential upgrade/replacement of the wet well complete with a list of advantages and disadvantages for each option, including Class 'D' cost estimates.

1.1 Project Background

The Hamlet of Fort Providence is located on the north shore of the Mackenzie River, near Great Slave Lake. The site of the existing water treatment plant (WTP) is located approximately two kilometres southeast of the Hamlet of Fort Providence, on the north bank of the Mackenzie River. The existing WTP facility complex consists of the water treatment plant building, the adjacent storage garage, the wet well structure and associated underground piping all of which are bound by the Mackenzie River to the south and the Fort Providence access road to the north.

According to historical documentation (Reid Crowther, 2000), the wet well was constructed in 1976 and consists of an enclosed concrete structure constructed over and around an existing vertical 760 mm diameter corrugated steel pipe culvert (CSP culvert) serving as the wet well. There were two points of entry provided through the concrete roof, including one manhole directly above the culvert to allow for pump removal and maintenance and another to allow for general access to the structure. The main concrete enclosure structure is covered with a sloping concrete ice cap to protect against and deflect ice impact. At the base of the CSP culvert, three intakes extend downstream approximately 24 m from the culvert into the Mackenzie River and reportedly consist of 100 mm diameter ductile iron pipe with perforated end caps.

Two submersible pumps located at the base of the CSP culvert draw water from the wet well and supply it to the water treatment plant via an original 100 mm diameter buried ductile iron pipe, combined with a 100 mm buried high density polyethylene (HDPE) pipe tied into the original pipe[1] during construction of the 1993 water treatment plant (current facility).

Historical issues with the 1976 wet well have included damage to the intake pipes from ice impact; limited access for maintenance during any given year due to ice cover above the wet well; heavy silting of the intakes and wet well, and reduced water production within the wet well when river water levels become too low to supply sufficient water through the intakes to the wet well (reported November 1999). As a result, a water supply improvement study was undertaken (Reid Crowther, 2000). Following recommendations in that study, a capital project to replace the existing intake system was completed in September 2004. The intent was to replace the existing gravity fed wet well system with a twin intake system (one for process water and one for fire flow) which was set at a lower elevation than the previous wet well (from 1976) gravity fed intake pipes. This was performed to ensure adequate water supply even at low water conditions, which would otherwise render the wet well gravity intake system inoperable. The twin intake system would also be reconfigured using isolation valves to serve as backup to each other if needed.

It has been reported that the 2004 intake system has not been capable of maintaining consistent flow and pressure required for the treatment process and fire process (Hamlet of Fort Providence, 2013). As a result, WTP operators began switching back and forth between the 2004 wet well system and the 1976 wet well system since 2004. As outlined in the project RFP, in October of 2003, divers tried to perform a visual inspection of the new intakes, however the divers were unable to locate the intakes due to extreme river conditions and safety concerns. Instead, one of the two new intakes (250 mm) was located using a

[1] Dillon Consulting Ltd. (1992). Fort Providence Water Treatment Plant, Drawing 101. July.

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submersible video camera from ice on the river, and the intake was found lying on its side. The other new intake (300 mm) was not able to be located with the submersible camera.

Per the Project RFP, the Hamlet had further hired contractors to perform intake investigations in 2008 and 2009 in an attempt to identify the issues and to locate both intakes by placing a camera inside the intakes. However, both investigations were unsuccessful. Also reported in the project RFP, the 2004 intake system has been abandoned because of inadequate flow and pressure for the process train to work properly. The community has since switched back to the old wet well intake system exclusively.

As such, the Hamlet has commissioned a condition assessment and performance review in response to the historical issues which still remain and to determine if the wet well in its current state can meet the water supply needs for the Hamlet (including fire protection). Further the Hamlet seeks to eliminate the issues with the 1976 wet well system, described above, that is currently used to supply water to the community. The condition assessment can be found in ARKTIS (2014), with a summary given in **Section 4** below. This report builds upon the condition assessment to develop conceptual design options to remedy issues with the 1976 wet well system currently in use.

2 SCOPE OF WORK

This options analysis report is in partial fulfillment of the project scope of work. More specifically, the purpose of this report is to:

- Provide a general site description;
- Describe a background of the existing water treatment plant
- Outline issues with the existing water treatment plant and required upgrades/ criteria set forth by the Hamlet;
- Outline previous historic studies pertaining to the existing WTP, including observations and recommendations made during the condition assessment portion of this contract undertaken by ARKTIS;
- Outline and describe options to upgrade the existing WTP to achieve criteria set forth by the Hamlet including:
    - 25 year life for the intake;
    - Year round access to the pump;
    - Year round raw water supply capability;
    - Provide appropriate fire flow requirements; and
    - Meet applicable regulations.
- Make recommendations based on described advantages and disadvantages of each option.

As stated in ARKTIS' proposal for this project, details regarding specific mechanical equipment associated with the improvements are beyond the scope of this report, however commentary is given in the report to discuss the feasibility of each design option from an electrical and mechanical perspective.

3 SITE DESCRIPTION

3.1 General

The Hamlet of Fort Providence is located on the north shore of the Mackenzie River, near Great Slave Lake. The site of the existing water treatment plant (WTP) is located approximately two kilometres southeast of the Hamlet of Fort Providence, on the east bank of the Mackenzie River. The existing WTP facility complex consists of the water treatment plant building, the adjacent storage garage, the wet well structure and associated underground piping all of which are bound by the Mackenzie River to the south and the Fort Providence access road to the north.

The water supply system in Fort Providence is comprised of a primary water treatment plant system and several subsystems. The water to the community is trucked from the WTP. The subsystems include:

- Raw water supply to the WTP, which is comprised of three gravity intakes to a wet well. The wet well has two submersible pumps controlled by the level gauges in the clear wells. The pumps feed the WTP;
- The water treatment primary unit is a Neptune Microfloc plant (Water Boy). This is a coagulation and multi-media filtration package plant. Upstream of the plant are chemical injectors for alum and polymers injection. On the supply line to the plant is also an on-line filter and flash mixer;
- Treated water from the Water Boy is stored in two underground concrete reservoirs (clear wells). The reservoir levels are controlled by two ultra-sonic level sensors;
- The truck fill sub-system consists of two submersible pumps in the clear wells. The pumps are controlled by the truck fill control panel located on the outside wall of the building on the truck fill arm and inside the truck fill panel. The water is chlorinated in the truck fill piping prior to discharge to the trucks.

Of note, the current water licence for the community is based on the 2004 inclined shaft intake system and not the 1976 intake system that is being used to supply water to the community presently.

3.2 Population and Water Demand

Reid Crowther (2000) performed water supply estimates to the year 2020 in order to assess the suitability of the 1976 intake system to supply adequate water volumes to the community, and to also assess contingencies including sufficient fire flow. The deficiencies of the 1976 intake system are well documented in that report, and as such are not presented herein since the same deficiencies in adequate volumes are still applicable to present date and beyond for the 1976 intake system.

However for conceptual planning purposes of intake improvement conceptual design options, population projections were taken from Government of the Northwest Territories (GNWT) Bureau of Statistics for a 25 year design life of the water treatment facility, assuming upgrades to the system are performed in 2015. It should be noted that the GNWT Bureau of Statistics provides conflicting data on their website as follows:

- Census 2011 data shows a population of 734 in 2011 for Fort Providence;
- GNWT population projections show a population of 778 for Fort Providence; and
- The statistical profile for Fort Providence on the GNWT Bureau of Statistics website states a 2012 population of 788.

For conservatism, the population projections used the higher number presented by the GNWT Bureau of Statistics website, and applied a 1% population growth rate, which is a conservative assumption based on the average for Canada posted by the GNWT Bureau of Statistics. ARKTIS then performed a calculation to predict the water demand for total community water use with this population based on standards and guidelines (GNWT, 1993). Per GNWT (1993), a calculation for water demand was performed for a community with a population between 0 and 2,000 people, with a trucked water and sewage delivery system, as in place in Fort Providence. The calculation is as follows:

$$\textit{Volume (per capita)} = RWU*(1.0 + (0.00023*\textit{Population}))$$

Where RWU is the residential water use; 90 litres per capita per day (lpcd) as determined by Section 6.1.1 for trucked water and sewage.

Table 1 below provides the results using the above noted formula specific to the Hamlet of Fort Providence with a 2012 population of 788.

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Table 1: Projected Water Use in Fort Providence

Facility Year	Calendar Year	Population	Growth Rate	Total Water Use Per Capita	Total Water Use Per Day	Total Water Use Per Year
		(persons)		(lpcd)	(litres/day)	(litres/year)
-	2012	788	0%	106.3	83,774	30,598,286
-	2013	796	1%	106.5	84,741	30,951,686
-	2014	804	1%	106.6	85,721	31,309,573
0	2015	812	1%	106.8	86,713	31,672,011
1	2016	820	1%	107.0	87,718	32,039,065
2	2017	828	1%	107.1	88,736	32,410,801
3	2018	836	1%	107.3	89,767	32,787,287
4	2019	845	1%	107.5	90,811	33,168,591
5	2020	853	1%	107.7	91,868	33,554,782
6	2021	862	1%	107.8	92,939	33,945,930
7	2022	870	1%	108.0	94,024	34,342,107
8	2023	879	1%	108.2	95,122	34,743,385
9	2024	888	1%	108.4	96,235	35,149,840
10	2025	897	1%	108.6	97,362	35,561,546
11	2026	906	1%	108.7	98,504	35,978,578
12	2027	915	1%	108.9	99,661	36,401,016
13	2028	924	1%	109.1	100,832	36,828,937
14	2029	933	1%	109.3	102,019	37,262,422
15	2030	943	1%	109.5	103,221	37,701,552
16	2031	952	1%	109.7	104,439	38,146,410
17	2032	962	1%	109.9	105,673	38,597,081
18	2033	971	1%	110.1	106,923	39,053,649
19	2034	981	1%	110.3	108,189	39,516,202
20	2035	991	1%	110.5	109,472	39,984,828
21	2036	1,001	1%	110.7	110,772	40,459,617
22	2037	1,011	1%	110.9	112,089	40,940,660
23	2038	1,021	1%	111.1	113,424	41,428,051
24	2039	1,031	1%	111.3	114,776	41,921,882
25	2040	1,041	1%	111.6	116,146	42,422,251

3.3 Mackenzie River Water Level Assessment

The maximum and minimum daily water levels recorded at the Fort Providence Water Survey of Canada Station (WSC Station #10FB001) for the period of 1979 to 2012 (some gaps in the data were noted) were used to determine probable maximum and minimum water levels to aid in the recommendation of conceptual options for a new water intake system.

Hydroconsult EN3 Services (Hydroconsult) performed a similar frequency analysis for the 2004 intake design with water level data up to 1999 (Dillon, 2002 and Reid Crowther, 2000). The Hydroconsult study presented a methodology to transfer water levels from the WSC gauging station (#10FB001) to the intake site that included a statistical analysis correlating gauging station water levels with water levels measured at the intake site for the period of 1992 to 1999. Since no other available water level data at the intake site was available, ARKTIS limited its efforts to a review of the technique and found it to be appropriate. As such, the reported derived water level was adopted for the statistical analysis presented herein as:

*Intake Site Water Level = 2.568 + 0.944 * (WSC Water Level)*

The minimum and maximum annual water levels at the WSC gauging station were then determined from the daily data, and was transferred to the intake site, as given in **Table 2** below. A statistical distribution of this data was then performed using a Log-Pearson III distribution to determine probable maximum and minimum water levels for various return intervals.

Table 2: Estimated Annual Minimum and Maximum Water Levels at the Fort Providence Intake Site

Year	Minimum WSC Water Level (m)	Maximum WSC Water Level (m)	Minimum Level Converted to Intake (m)	Maximum Level Converted to Intake (m)
1979	150.777	153.661	144.9	147.62
1980	150.671	151.869	144.8	145.93
1981	151.088	152.136	145.2	146.18
1982	151.384	153.491	145.47	147.46
1983	151.406	152.074	145.5	146.13
1984	150.968	153.261	145.08	147.25
1985	151.697	152.84	145.77	146.85
1986	151.46	152.843	145.55	146.85
1987	151.706	152.497	145.78	146.53
1988	150.821	153.532	144.94	147.5
1989	151.455	152.388	145.54	146.42
1990	150.968	153.735	145.08	147.69
1991	150.858	152.569	144.98	146.59
1992	150.911	153.742	145.03	147.7
1993	149.306	152.811	143.51	146.82
1994	150.045	152.276	144.21	146.32
1995	148.995	152.125	143.22	146.17
1996	-	-	-	-
1997	-	-	-	-
1998	150.366	153.96	144.51	147.91
1999	-	-	-	-
2000	-	-	-	-
2001	-	-	-	-
2002	-	-	-	-
2003	-	-	-	-
2004	-	-	-	-
2005	-	-	-	-
2006	-	-	-	-
2007	149.532	153.521	143.73	147.49
2008	149.369	151.656	143.57	145.73
2009	149.748	151.774	143.93	145.84
2010	149.297	152.215	143.5	146.26
2011	149.938	152.109	144.11	146.16
2012	149.837	152.027	144.01	146.08

The Log-Pearson Type III distribution is a statistical technique for fitting frequency distribution data to predict the design flood for a river at any given site. Once the statistical information is calculated for the river site, a frequency distribution can be constructed. The advantage of this particular technique is that extrapolation can be made of the values for events with return periods well beyond the observed flood events. This technique is the standard technique used by Federal Agencies in the United States (Bedient et. al., 2002). The Log-Pearson Type III distribution gives the likely values of discharges to expect in the river at recurrence intervals based on available historical records taken from the WSC. The minimum and maximum water levels for various return periods calculated using this statistical analyses are given below in **Table 3**.

Table 3: Probability Analysis Results for Maximum and Minimum Water Levels

Return Period (year)	Maximum Water Level (m)	Minimum Water Level (m)
2	146.69	144.61
5	147.29	143.97
10	147.62	143.67
25	148.00	143.36
50	148.25	143.18
100	148.49	143.02
200	148.71	142.88

The water levels for the 50 year return period were used to develop conceptual options for the intake improvements presented in **Section 5** below.

4 PREVIOUS STUDIES

4.1 Historic Studies and Investigations

A planning study was performed in 2000 to examine historic issues with the intake at the Fort Providence WTP (Reid Crowther, 2000). More specifically, the study aimed to review two problem areas with the existing intake system:

- Raw water supply interruptions from low water levels in the Mackenzie River; and
- River ice deposits on top of the existing wet well for long periods of time each year.

Reid Crowther (2000), reviewed alternatives for supplying raw water to the WTP. During that review it was concluded that any options involving modifications or tie-ins to the existing wet well, or deepening the existing wet well, were unworkable due to constructability issues. Reid Crowther (2000) also concluded that new designs would not require, and should not include, wet well concepts. As opined in that report, wet wells were considered to be large, expensive structures that provide only limited functionality at significant capital cost.

As such, Reid Crowther (2000) recommended a conventional inclined shaft intake system, which was stated to be similar in composition and construction to those currently in use in various communities throughout the Northwest Territories. The recommended inclined shaft intake concept (Option 3B) was described to be familiar to owners and contractors, it can be efficiently executed onsite, may require a minimum of materials and expertise and can be designed to minimize potential for ice related damage. However it was noted that there will always be risk of damage for any structure constructed in ice forming rivers like the Mackenzie River. As a result it was recommended that by burying the intake into the river bottom and having only the screen exposed, the risk of ice damage should be minimized. Further, it was recommended that the Owner consider the twin intake concept with a dedicated fire line.

The recommended inclined shaft concept had two distinct advantages over the existing, per Reid Crowther (2000):

- Year round access to the raw water pump for maintenance purposes; and
- Improved year round raw water supply capability.

Further, Reid Crowther (2000) examined several end treatments (screen details). The treatments were considered to be a compromise between efficiency and durability with respect to ice damage, since it was deemed impractical to design an intake end to withstand a major ice strike. The intent was to have the screen assembly separate from the main intake, in the event of a major ice strike, rather than risking the entire intake. Secondary screens recessed into the main intake were designed to remain in place below the breakaway section, and protect the system components until such time as the intake end was repaired.

EBA (2000) reviewed available geotechnical data (report included as an appendix within Reid Crowther, 2000) to be able to make recommendations for construction of the proposed inclined shaft and intake within the Mackenzie River. EBA (2000) stated that directional drilling is often used to install pipelines into riverbeds, but stated that no information was available at that time to make design recommendations as such. However, they noted that due to available information on the riverbank, directional drilling may not be possible due to the cobbles in the till stratigraphy and near the water level, since directional drilling has only been shown to be effective in clay, silt, and some sands. EBA recommended geotechnical exploration to support the design.

Dillon Consulting Ltd. was commissioned by the GNWT to complete the preliminary design, detailed design and construction services to implement the recommended inclined shaft intake system that was recommended by Reid Crowther (2000). Dillon (2002) outlined studies of river bathymetry, river currents, and water levels within the Mackenzie River to make design recommendations on the location of the intakes within the river. Two options were presented to the GNWT, who then chose the final design option in order to minimize risks associated with damage of the intakes from ice, the capital cost of installation, and operational costs for the intended design life.

Additionally Dillon (2002) examined construction of the proposed intakes and inclined shaft via horizontal directional drilling (HDD) and via open cut. AMEC Earth and Environmental was retained to complete a geotechnical review of construction including shafts with those two construction methods (included as an appendix to Dillon, 2002). AMEC inferred subsurface conditions from a review of published geotechnical information, which was stated to consist of a thin organic mat overlying low plastic clay till to at least a depth of 9 m. Layers of fine sand were said to be interbedded with the clay till. Further, cobbles and boulders were said to vary in quantity throughout the clay till, and that stratigraphy below 9 m was unknown. It was concluded by Dillon (2002) that based on relatively larger unknown geotechnical conditions within the river that HDD be taken off the table for consideration. Slope stability of the riverbank was noted as a potential issue, as was discussed in ARKTIS (2014).

Additional drilling along the design intake alignment was conducted and presented in a subsequent memorandum (AMEC, 2002). It is assumed that this information was further used to understand design considerations made by Dillon (2002).

4.2 2013 Investigation

As reported in ARKTIS (2014), ARKTIS completed a condition assessment in fall of 2013 of the Fort Providence water supply wet well and associated components which include the wet well concrete enclosure, the corrugated steel pipe culvert and the three intake pipes; as well, draw down testing was undertaken to evaluate the existing performance of the wet well to supply water for delivery to the water treatment plant via two submersible pumps. The assessment and performance test was in partial fulfilment of the project scope of work, preceding this options analysis report.

The results detailed in ARKTIS (2014) found an aging system with some components in satisfactory condition and others nearing or considered to be at the end of their intended service life. As such, ARKTIS (2014) agreed with the current course of the project, recommending that replacement and/or upgrade to the facility continue to be pursued. The current system can remain in use during the typical implementation time for a new project which is assumed to include study of options; community review, consideration and input; design (schematic, design development and construction documentation); tender; and, construction/commissioning. However, the current system components should be monitored and maintained regularly over this time, which should include visual review of the CSP culvert; seasonal cleaning of the intake lines; and, perhaps even seasonal video inspection of the intakes.

Regardless of the existing conditions of the wet well components, inherent risk with the current system still remains; issues include water supply deficiencies if the Mackenzie River water level becomes lower than the existing intake openings (i.e. November 1999 reported starvation of wet well) and potential for ice cover to restrict access to the wet well, which can also disrupt water supply to the community. As such, it is recommended that during the study, design and construction of a new system, that a risk management/emergency plan be put in place for alternate supply of water should the current system fail to provide this need. Restoration only, cannot remedy these issues and this provides stronger evidence to support a new system.

Furthermore, performance testing confirmed that the existing wet well and gravity feed intake system cannot meet the fire protection requirement of 1000 L/min. water supply in the case of a fire emergency. As such, the existing water treatment plant storage cells (located within a concrete storage reservoir below the exiting water treatment plant) need to be relied upon for not only continuous drinking water supply for the community but also a minimum of 60,000 L water supply for fire protection.; it should be noted that the October 2000 study completed by Reid Crowther identified a water supply (potable water and fire storage) issues over the years leading up and including the 2020 design year based on population projections. As such, to meet both fire protection requirements and community water demand, a new water supply system is justifiably required.

Finally, it should be noted that slope stability of the riverbank is a major geotechnical concern for the existing WTP facility, as erosion of the riverbank, both natural and manmade, may work to destabilize the riverbank. This may have severe and direct serviceability implications to the WTP facility should a slope failure occur. While the actual risk of slope instability cannot be determined at this phase of the project, several measures can be implemented in the short term to help minimize the risk of instability. These measures may include, but are not limited to:

- Repairing and limiting further erosion from manmade processes (i.e. due to discharge of the backwash/overflow pipe) by modifying the discharge point of the pipe to a location further down the riverbank at the low water level in the Mackenzie River.
- Vegetating the slope to help minimize further erosion with possible additional erosion protection measures including, but not limited to, placement of riprap along the bank.

A geotechnical investigation (including drilling boreholes combined with test pits and geotechnical laboratory testing) should be performed prior to a final design phase to confirm geotechnical conditions. The results of this investigation should be combined with slope stability modelling to determine the actual risk of slope instability and further make recommendations to reduce that risk to acceptable levels.

5 CONCEPTUAL DESIGN OPTIONS

As mentioned in Reid Crowther (2000), there are several options that can be implemented for the design of a new intake system, but the success of each cannot be fully guaranteed. ARKTIS shares this opinion based on a review of information to date regarding currents, water levels, and issues with ice damage. A new intake system cannot be guaranteed to solve all of these issues, rather the conceptual design options serve to mitigate and minimize the risks associated with these issues. ARKTIS has explored several potential design concepts based on available information to date.

As discussed in Reid Crowther (2000), an inclined shaft system is typical of many communities throughout the Northwest Territories; however, due to the issues with the 2004 intakes, it is unclear what caused the issues that were experienced (described in Section 1.1 above). As outlined above, it has been reported that the 2004 intake system has not been capable of maintaining consistent flow and pressure required for the treatment process and fire process. As a result, WTP operators began switching back and forth between the 2004 wet well system and the 1976 wet well system since 2004, and eventually abandoned using the 2004 intakes and exclusively have been using the 1976 system. As outlined in the project RFP, in October of 2004, divers tried to perform a visual inspection of the new intakes, however the divers were unable to locate the intakes due to extreme river conditions and safety concerns. Instead, one of the two new intakes (250 mm) was located using a submersible video camera from ice on the river, and the intake was found lying on its

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side. The other new intake (300 mm) was not able to be located with the submersible camera. Additionally, the Hamlet had further hired contractors to perform intake investigations in 2008 and 2009 in an attempt to identify the issues and to locate both intakes by placing a camera inside the intakes. However, both investigations were unsuccessful.

It is unclear whether the 2004 intakes were damaged by ice (or other means), or were subject to inadequate or substandard construction practices that rendered them as having insufficient flow. Based on discussions with the Hamlet, a considerable amount of time and money performing investigations to understand the problem with these intakes has been spent, with very little result. Again, it is unclear whether the success of the investigations were a function of design issues, inadequate construction techniques, or substandard equipment (or some combination of any of all those possibilities), however due to the relatively high capital cost of constructing a new intake, ARKTIS would like to further discuss options to improve the existing inclined shaft intakes that were constructed in 2004. The additional capital spent on a proper investigation may be small in comparison to an entirely new intake system, even if it runs the risk of showing that no improvements can be made to the 2004 system. It is ARKTIS' opinion that it is better to be certain of all issues associated with the 2004 intakes before improvements to that system are taken off the table for consideration in improving the overall function of the existing WTP. As such, ARKTIS has included options for rehabilitation to the existing 2004 intake system for completeness of this options analysis.

Five main conceptual design options are presented for the Hamlet's consideration in order to meet the required objectives for the Fort Providence WTP as outlined in Section 2.0 above. These five design options are discussed in detail within this Section and include:

- Option 1a - Repair/Modify the existing 2004 inclined shaft with intake in deeper water.
- Option 1b – Repair/modify the existing 2004 inclined shaft with dredging near the shoreline so that the intake is located in deeper water, yet closer to shore for maintenance.
- Option 2 – Relocation of the entire WTP to an entirely new location adjacent to the community.
- Option 3 – Adding auxiliary water storage capacity to the WTP.
- Option 4a – New deeper wet well with new intake in deeper water further out into the Mackenzie River.
- Option 4b – New deeper wet well with dredging so intake is in deeper water yet closer to shore for maintenance.

The advantages and disadvantages for each conceptual option are discussed within each section below to assess each option, consistent with the original criteria for the 2004 intake upgrades and the RFP criteria set forth by the Hamlet including:

- 25 year life for the intake;
- Year round access to the pump;
- Year round raw water supply capability;
- Provide appropriate fire flow requirements; and
- Meet applicable regulations.

It should be noted that ARKTIS has consulted a third party mechanical engineer and described each intake conceptual option provided herein. For each option, the required fire flow of 1,000 L/min. can be achieved. Additionally, all intake screens can be designed to meet applicable DFO requirements.

5.1 Option 1a – Repair of existing 2004 inclined shaft

As mentioned above in **Section 5**, an inclined shaft system is typical of many communities throughout the Northwest Territories. However, it is unclear whether the 2004 intakes were damaged by ice (or other means), or were subject to inadequate or substandard construction practices that rendered them as having insufficient flow. Based on discussions with the Hamlet, a considerable amount of time and money performing investigations to understand the problem with these intakes has been spent, with very little result. The additional capital spent on a proper investigation may be small in comparison to an entirely new intake system, even if it runs the risk of showing that no improvements can be made to the 2004 system. As such, it is ARKTIS' opinion that it is better to be certain of all issues associated with the 2004 intakes before improvements to that system are taken off the table..

Assuming that a proper investigation is performed and the issues with the 2004 inclined shaft intake system are identified, Option 1a consists of a new inclined shaft similar to the 2004 intake system (**Figures 1** and **2**). This will include a twin intake system (one for process water and one for fire flow) consisting of 250 mm diameter inclined shaft intake with an inline submersible pump that extends approximately 100 m out into the river which was set at a lower elevation than the previous wet well (from 1976) gravity fed intake pipes. The intake pipe will be set deeply into the river bottom to reduce the amount of pipe being expose to ice damage by providing a vertical riser out of the river bottom, similar to the 2004 intake system. The elevation of the intake point in the river will be set below the 50 year low water level elevation noted in **Section 3.3** above. The twin intake system would also be configured using isolation valves to serve as backup to each other if needed. The pipe could consist of a hybrid of 250 mm diameter insulated high density polyethylene (HDPE) and epoxy coated steel in order to maximize ice protection and construction loads. Since Option 1a consists of twin intake pipes, there will be a pump in each intake, with two additional backup pumps for a total of four pumps required for this option.

Advantages include:

- Year round accessibility to the pumps for servicing, and it should be noted that the pump does not need to be placed at the end of the intake, rather it can be set at an elevation below the low water level to prevent starvation during low water events.
- The intake pipes will be set into the river bed to prevent damage to the pipe from ice. The riser helps minimize the target for ice impact, however routine inspections should be conducted during low water events to the intake and screens.

Disadvantages include:

- As with all intakes in rivers like the Mackenzie River, isolated damage to intake screens is expected, and routine inspections should be performed, with damaged screens replaced as needed.
- Similarly siltation and/or scour is also expected in the Mackenzie River for all intake options. Routine inspections should look for these issues as well, and corrective measures performed as required.
- Silt loads in the Mackenzie River are expected to be high, which reduces the expected life of pumps. Pumps are expected to required replacement every 5 to 8 years based on discussions with a third party mechanical engineer.

Option 1a <u>is recommended</u> for further consideration within this study.

5.2 Option 1b - Repair of existing 2004 inclined shaft with dredging

Option 1b is a modified version of Option 1a, with the only modification being dredging of the river bottom near the shoreline such that the intake pipe can be set at a lower elevation closer to the shoreline, reducing the length of pipe extending into the river (approximate length of pipe is 30 m from the shore). All other details are identical to Option 1a, as shown in **Figures 3** and **4**. Since Option 1b consists of twin intake pipes, there will be a pump in each intake, with two additional backup pumps for a total of four pumps required for this option.

Advantages include:

- Year round accessibility to the pumps for servicing, and it should be noted that the pump does not need to be placed at the end of the intake, rather it can be set at an elevation below the low water level to prevent starvation during low water events.
- The intake pipes will be set into the river bed to prevent damage to the pipe from ice. The riser helps minimize the target for ice impact, however routine inspections should be conducted during low water events to the intake and screens.

Disadvantages include:

- As with all intakes in rivers like the Mackenzie River, isolated damage to intake screens is expected, and routine inspections should be performed, with damaged screens replaced as needed.
- Similarly siltation and/or scour is also expected in the Mackenzie River for all intake options. Routine inspections should look for these issues as well, and corrective measures performed as required.
- Silt loads in the Mackenzie River are expected to be high, which reduces the expected life of pumps. Pumps are expected to require replacement every 5 to 8 years based on discussions with a third party mechanical engineer.

Option 1b is recommended for further consideration within this study.

5.3 Option 2 - Relocation of WTP

Option 2 includes the construction of an entirely new WTP in a new area away from the existing WTP. The purpose of relocating the WTP would be to place the WTP intake in a portion of the Mackenzie River, or other water body, to avoid issues with river siltation clogging the intake, as well as issues with ice damaging WTP infrastructure and low water levels (placed in deeper waters). Based on examination of aerial photographs, outside of the Mackenzie River, there are no possible water bodies within reasonable distance to the Hamlet as suitable potable water sources. As well, please note that other areas along the Mackenzie River will likely be subject to the same issues as the existing WTP site. Additionally, the Hamlet has already spent a considerable amount of money on repairing and/or upgrading the existing WTP and from the 2013 investigation, it appeared many components of the existing WTP were in proper serviceable condition. However, please note that the existing WTP is now 22 years old which, according to the GNWT (1993), is passed its “design economic life” of 20 years, but not its “design expected life” of 40 years.

Following presentation of all options to the Hamlet Council, at the Council’s request, Option 2 was considered further with respect to potential cost related to the other options herein. Please note that this option does not include a site selection study which is considered outside the scope of this report. **It is critical that a site selection study be performed before considering this option since the option is highly dependent on the potential for deeper water/slower current/less siltation along the Mackenzie River near Fort Providence for its successful implementation. Site selection would include not only an analysis of the Mackenzie River depths and river flow (Note: analysis may require bathymetric survey, localized river flow measurements, etc. for which sufficient time to complete the analysis may be required, such as a one year cycle to obtain localized river flow rates), but also the assessment of available land, analysis of geotechnical conditions for siting of a new building, and topographical survey. It is possible that an appropriate site providing conditions favourable to installation of a new intake meeting the requirements of Section 2.0 above will not be found.**

For the purposes of cost comparisons with other options, Option 5 consists of a new water treatment plant and inclined intake shaft similar to the intake shaft in Option 1a. The new WTP is assumed to be of similar construction to the existing (i.e. concrete foundation with reservoir cells below grade, wood frame construction, wood siding, and asphalt shingle roof, back-up generator, fuel oil furnace). To account for the design horizon of 2040, ARKTIS determined from population projections and water consumption data (using Table 1 above), that the required water storage in 2040 would need to increase by 40% from the existing WTP water storage. Calculations are summarized in **Appendix B**. Using the original WTP architectural, structural and mechanical/electrical drawings, ARKTIS found the approximate cost of constructing the existing WTP in 2015 dollars, then multiplied that cost by 1.4 (increase of 40%) to estimate the increased building size required to facilitate water storage for the 2040 design horizon.

Advantages include:

- New construction can address known issues with current facilitate without modification to existing systems.

- Selection of a new site to better address the issues of siltation, ice damage, low water levels (if possible along the Mackenzie River near Fort Providence).
- New facility with new technologies and new life cycle.

Disadvantages include:

- Highest overall capital cost and capital plus O&M and repair cost.
- Ideal site is unknown at this time and possibly does not exist.
- Additional cost and time to perform site selection study.
- Potentially longer duration to implement investigation, design, and construction for a fully functional facility.
- Ability to meet each of the Section 2.0 criteria is unknown at this time.

At the request of the Hamlet Council, Option 2 <u>is recommended</u> for further consideration within this study.

5.4 Option 3 - Addition of auxiliary storage capacity to existing wet well

Option 3 includes the addition of auxiliary storage capacity to the existing WTP to aid with times of low flow or low water levels in the Mackenzie River. This option includes an external prefabricated insulated water reservoir located adjacent to the WTP building. This option is viewed as a short term measure to meet some of the objectives set forth above, and is not expected to solve all of the existing operations and maintenance issues currently experienced with the 1976 wet well system. The storage reservoir would need to be sized to provide contingency storage during winter months when ice would block access to the wet well (and any mobile pumps), in the event of a pump malfunction and the pump could not be accessed. However, based on the water demand calculations presented above, the size of the storage tank would exceed any available

Advantages include:

- Contingency storage for low water level events below the existing 1976 intake level.
- Readily available supply for fire flow requirements at any time of the year.

Disadvantages include:

- Based on the water generation estimates provided in **Table 1** above the size of the required storage reservoir would be so large that it exceeds any available space at the existing WTP site.
- Does not meet the requirement for year round access to the pumps.
- Siltation and/or scour is also expected in the Mackenzie River for all intake options. Routine inspections should look for these issues as well, and corrective measures performed as required.
- Silt loads in the Mackenzie River are expected to be high, which reduces the expected life of pumps. Pumps are expected to required replacement every 5 to 8 years based on discussions with a third party mechanical engineer.

Option 3 <u>is not recommended</u> for further consideration within this study.

5.5 Option 4a - Installation of new wet well within a jetty

Option 4a includes the construction of a new wet well system. As shown in **Figures 5** and **6**, the wet well will consist of a 2 m diameter CSP vertical culvert installed within a jetty into the river from the shoreline. The top elevation of the jetty will be set above the 50 year high water level of the Mackenzie River (from **Table 3** above). The jetty will extend approximately 40 m into the river, such that the bottom of the vertical CSP intake pipe is below the 50 year low water level (**Table 3** above). The vertical CSP pipe will be slotted between the low and high water levels, and submersible pumps will be placed within the vertical CSP pipe. The jetty will be constructed of both run of quarry (600 mm minus material) and 150 mm minus granular material. A non-woven geotextile will wrap the entire jetty to act as a filter to prevent clogging of the granular material that makes up the jetty. The geotextile will be covered with rip-rap to protect against wave, flow, and ice forces. Since Option 4a consists of a single CSP wet well, a single pump is required with a backup pump, for a total of two pumps for this option. A pump house can be built on the jetty to allow for enclosed maintenance of the pumps and piping; the pump house avoids confined space issues currently experienced for maintenance of the 1976 wet well.

Advantages include:

- Year round accessibility to the pumps for servicing.
- The CSP well pipe will provide year round raw water supply.
- Readily available supply for fire flow requirements at any time of the year.
- While silt loads in the Mackenzie River are expected to be high, which can reduce the expected life of pumps if they're subject to high siltation, the geotextile filter layer on the jetty will reduce the amount of silt getting into the CSP well. As such, the expected service life of the pumps is 8 to 10 years.
- Jetty is a robust structure that is expected to provide superior protection and resistance to ice impact forces for WTP infrastructure.

Disadvantages include:

- Siltation and/or scour is also expected in the Mackenzie River for all intake options. Routine inspections should look for these issues as well, and corrective measures performed as required. The geotextile layer is considered sacrificial, and will need to be replaced as it becomes clogged with silt from the River. The frequency of removal is difficult to predict at the conceptual level, as such it was assumed that it will require replacement every 8 years.

Option 4a is recommended for further consideration within this study.

5.6 Option 4b - Installation of new wet well within a jetty with dredging

Option 4b is a modified version of Option 4a, and includes the construction of a new wet well system. As shown in **Figures 7** and **8**, the wet well will consist of a 2 m diameter CSP vertical culvert installed within a jetty into the river from the shoreline, however the modification from Option 4a is dredging an area in the river near the shore to allow a short jetty extending into the Mackenzie River, while still maintaining the bottom elevation below the 50 year low water level. The top elevation of the jetty will be set above the 50 year high water level of the Mackenzie River (from **Table 3** above). The jetty will extend approximately 25 m into the river, such that the bottom of the vertical CSP intake pipe is below the 50 year low water level (**Table 3** above). The vertical CSP pipe will be slotted between the low and high water levels, and submersible pumps will be placed within the vertical CSP pipe. The jetty will be constructed of both run of quarry (600 mm minus material) and 150 mm minus granular material. A non-woven geotextile will wrap the entire jetty to act as a filter to prevent clogging of the granular material that makes up the jetty. The geotextile will be covered with rip-rap to protect against wave, flow, and ice forces. Since Option 4b consists of a single CSP wet well, a single pump is required with a backup pump, for a total of two pumps for this option. A pumphouse can be built on the jetty to allow for enclosed maintenance of the pumps and piping; the pumphouse avoids confined space issues currently experienced for maintenance of the 1976 wet well.

Advantages include:

- Year round accessibility to the pumps for servicing.
- The CSP well pipe will provide year round raw water supply.
- Readily available supply for fire flow requirements at any time of the year.
- While silt loads in the Mackenzie River are expected to be high, which can reduce the expected life of pumps if they're subject to high siltation, the geotextile filter layer on the jetty will reduce the amount of silt getting into the CSP well. As such, the expected service life of the pumps is 8 to 10 years.

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- Jetty is a robust structure that is expected to provide superior protection and resistance to ice impact forces for WTP infrastructure.

Disadvantages include:

- Siltation and/or scour is also expected in the Mackenzie River for all intake options. Routine inspections should look for these issues as well, and corrective measures performed as required. The geotextile layer is considered sacrificial, and will need to be replaced as it becomes clogged with silt from the River. The frequency of removal is difficult to predict at the conceptual level, as such it was assumed that it will require replacement every 8 years.

Option 4b is recommended for further consideration within this study.

6 OPTION SUMMARY

Five main conceptual design options were presented above and include:

- Option 1a- Repair/Modify the existing 2004 inclined shaft with intake in deeper water.
- Option 1b – Repair/modify the existing 2004 inclined shaft with dredging near the shoreline so that the intake is located in deeper water, yet closer to shore for maintenance.
- Option 2 – Relocation of the entire WTP to an entirely new location adjacent to the community.
- Option 3 – Adding auxiliary water storage capacity to WTP.
- Option 4a – New deeper wet well with new intake in deeper water further out into the Mackenzie River.
- Option 4b – New deeper wet well with dredging so intake is in deeper water yet closer to shore for maintenance.

The advantages and disadvantages for each conceptual option were discussed in detail above, and a summary is provided in **Table 4** below.

Table 4: Summary of Conceptual Options

Criteria	Option 1a	Option 1b	Option 2	Option 3	Option 4a	Option 4b
Year round access to pump	Pass	Pass	NA	Fail	Pass	Pass
Year round raw water supply	Pass	Pass	NA	Fail	Pass	Pass
Risk to ice damage	Moderate to High	Moderate to High	NA	Low to Moderate	Low	Low
Risk of plugging from silt	Moderate to High	Moderate to High	NA	Moderate to High	Moderate	Moderate
Ease of Construction	Moderate	Moderate to Difficult	High	Low	Low	Low to Moderate
Ease of Maintenance	Moderate	Moderate to Difficult	NA	Low to Moderate	Low	Low to Moderate
Capital Cost (see Table 5 below)	Low	Low	High	Low to Moderate	Moderate	Moderate
Ranking	4	3	NA	-	2	1

Notes:

- Green – Low risk, or low effort
- Yellow – Moderate risk, or moderate effort
- Red – High risk, or high effort
- NA = Unknown at this time without further site selection study and analysis.

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6.1 Costing

Table 5 below includes a summary of a Class D, order of magnitude cost estimate, for each conceptual option. Actual cost break downs for each option are included in **Appendix C.** The Class D cost estimates do not include costs for GST nor any other applicable taxes. It also does not account for any construction that takes longer than one construction season. All quantities were roughly estimated and are expected to change as the design progresses. Unit rates were based on reputable industry modelling databases, contractor quotes, and experience from previous similar projects in the Northwest Territories and Nunavut. It should be noted that the Class D estimate is "*based upon a statement of requirements, and an outline of potential solutions, this estimate is strictly an indication (rough order of magnitude) of the final project cost, and should be sufficient to provide an indication of cost and allow for ranking all the options being considered*" (Construction Economist, 2002). As such, the estimate should be best viewed as a tool, highlighting the cost relativity of the options presented, and should solely be used to aid in the selection of a conceptual design option to move forward in design, as per the Hamlet's direction. Note that by typical industry definition, a Class D estimate has an accuracy of +/- 50% of the final project cost, and should not be used to set construction budgets, until a preliminary or schematic design can be completed for one of the options. The construction budget should then be updated throughout the design process including schematic, design development and construction document phases of design.

Table 5: Summary of Capital and O&M Class D Costs for Conceptual Options

Conceptual Option	Capital Cost	Capital + O&M and Repair Cost Over Design Life (NPV[1])
Option 1a	$1,329,800	$1,766,907
Option 1b	$1,586,800	$2,124,650
Option 2	$3,766,000	$4,203,107
Option 4a	$2,717,300	$3,070,932
Option 4b	$2,952,000	$3,318,735

[1] NPV = Net Present Value

As presented in **Appendix C**, annual operations and maintenance (O&M) costs were calculated two ways. The more conservative analysis assumed annual O&M as 5% of the direct capital cost. For each option, repairs include at least one round of pump replacements for all options, maintenance dredging performed twice for the options that include dredging, and replacement of the geotextile twice for Options 4a and 4b.

7 CONCLUSIONS AND RECOMMENDATIONS

The six conceptual options described in detail in **Section 5** above have been summarized in **Table 4** above for the criteria outlined in **Sections 2** and **5** above. For each option, a colour code has been applied for each consideration of the conceptual options as follows:

- Green – Low risk, or low effort
- Yellow – Moderate risk, or moderate effort
- Red – High risk, or high effort

The colour code has been applied based on ARKTIS' assessment of the advantages and disadvantages of each conceptual option for the specific considerations detailed in **Section 5** above. **Table 4** is intended to be a visual aid that ranks conceptual design options relative to each other in order to make an informed decision for moving into subsequent phases of the project. Options 1a, 1b, 4a, and 4b meet all requirements of the project. However, ARKTIS recommends Options 4a and 4b over 1a and 1b since Options 1a and 1b are still dependent upon a proper investigation to determine the issues with the 2004 intakes (and associated costs not considered herein), the relative ease of construction and maintenance, and the lower risk of damage to ice impact. As well, the ability of Option 2 to meet the project objectives is unknown at this time; though, through proper site selection, the goal would be to meet all of the criteria listed in Section 2.0. **Before proceeding with Option 2, a detailed site selection study is required.** While Option 4a and 4b have higher capital construction and O&M costs than Options 1a and 1b, ARKTIS feels that the higher cost is worth the minimized risk of damage to the intake from both ice damage and siltation. Additionally, while the capital construction and O&M costs of Option 4b are slightly higher than those for Option 4a, ARKTIS recommends that the Hamlet of Fort Providence consider Option 4b over Option 4a since the jetty does not extend into the river as far, thus minimizing effects of river flow and ice impact to infrastructure. Of particular note, detailed engineering and modelling that is outside of the scope of this report is required to fully understand the jetty's effect on river flow, silt deposition, scour, and ice flow.

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9 LIMITATIONS

This report has been prepared exclusively for the use of the Incorporated Hamlet of Fort Providence. The information, opinions and recommendations contained in this report should not be used for any other purpose, at another location, or by any other party. The options and recommendations made within this report were based on available data and observations presented within this report any may change in light of additional new data, observations, and input. Any use of, or reliance on this report by any third party is at that party's sole risk. The contents of this report were prepared in accordance with generally accepted principles and practices. No other warranty, expressed or implied, is given.

10 CLOSURE

ARKTIS would like to thank the Hamlet for retaining its services and welcomes the opportunity to complete additional services in the future. If you have any questions whatsoever please feel free to contact Greg Fairthorne at 867.446.4129 or fairthorne@arktissolutions.com.

ARKTIS SOLUTIONS INC.

[Original Hardcopy Signed]	[Original Hardcopy Signed]
Greg Fairthorne, P.Eng. VP, Infrastructure Engineering	Jason Thorpe, M.Sc., P.E., P.Eng. Geotechnical Engineer

APPENDIX A – FIGURES

1 OPTION 1A SITE PLAN
1:250

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DATE:	CHECKED BY:
SCALE:	PLOT DATE:

PROJECT:

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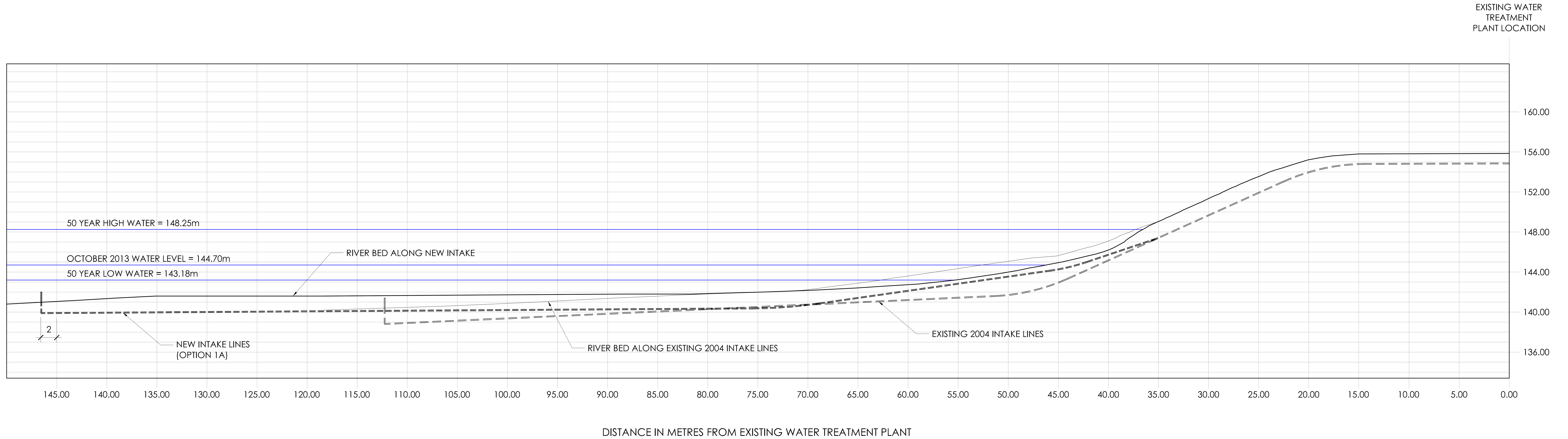
WTP OPTIONS ANALYSIS REPORT

FORT PROVIDENCE, NT

DRAWING NAME:
OPTION 1A

DRAWING NO.:
FIGURE 1



1 OPTION 1A PROFILE

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OPTION 1A

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FIGURE 2

1 OPTION 1B SITE PLAN
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OPTION 1B

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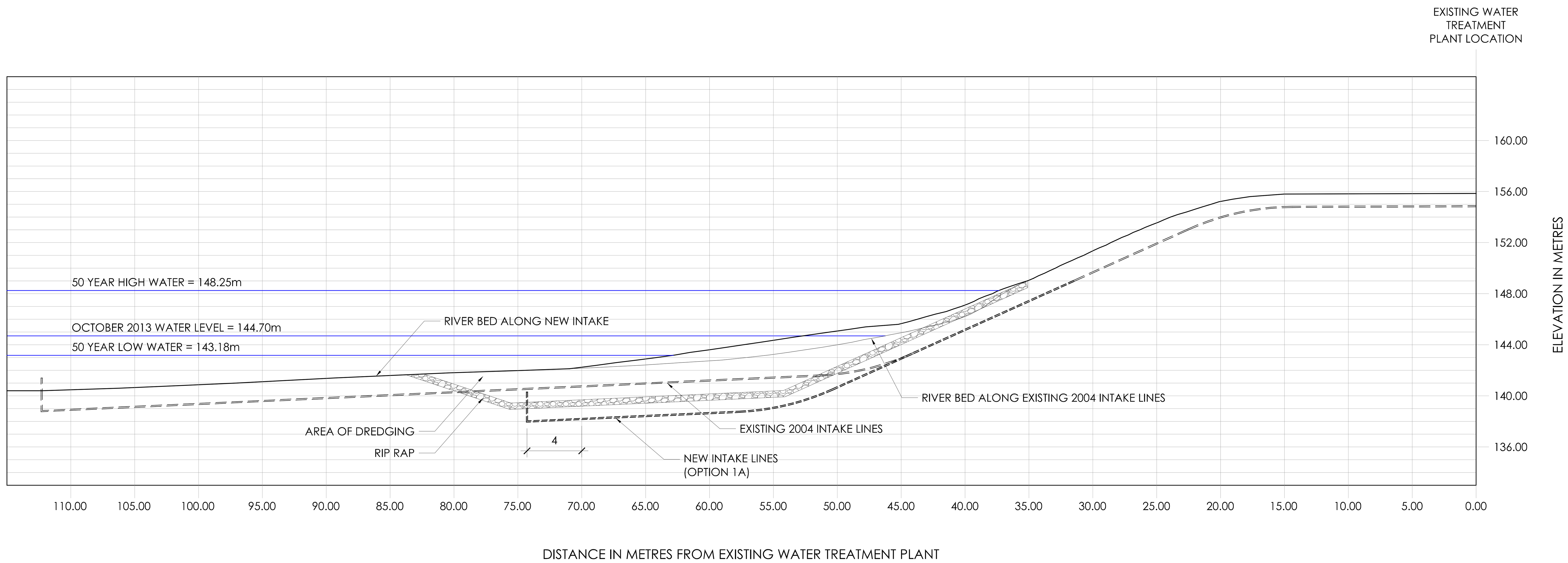
FIGURE 3



1 OPTION 1B SITE PLAN

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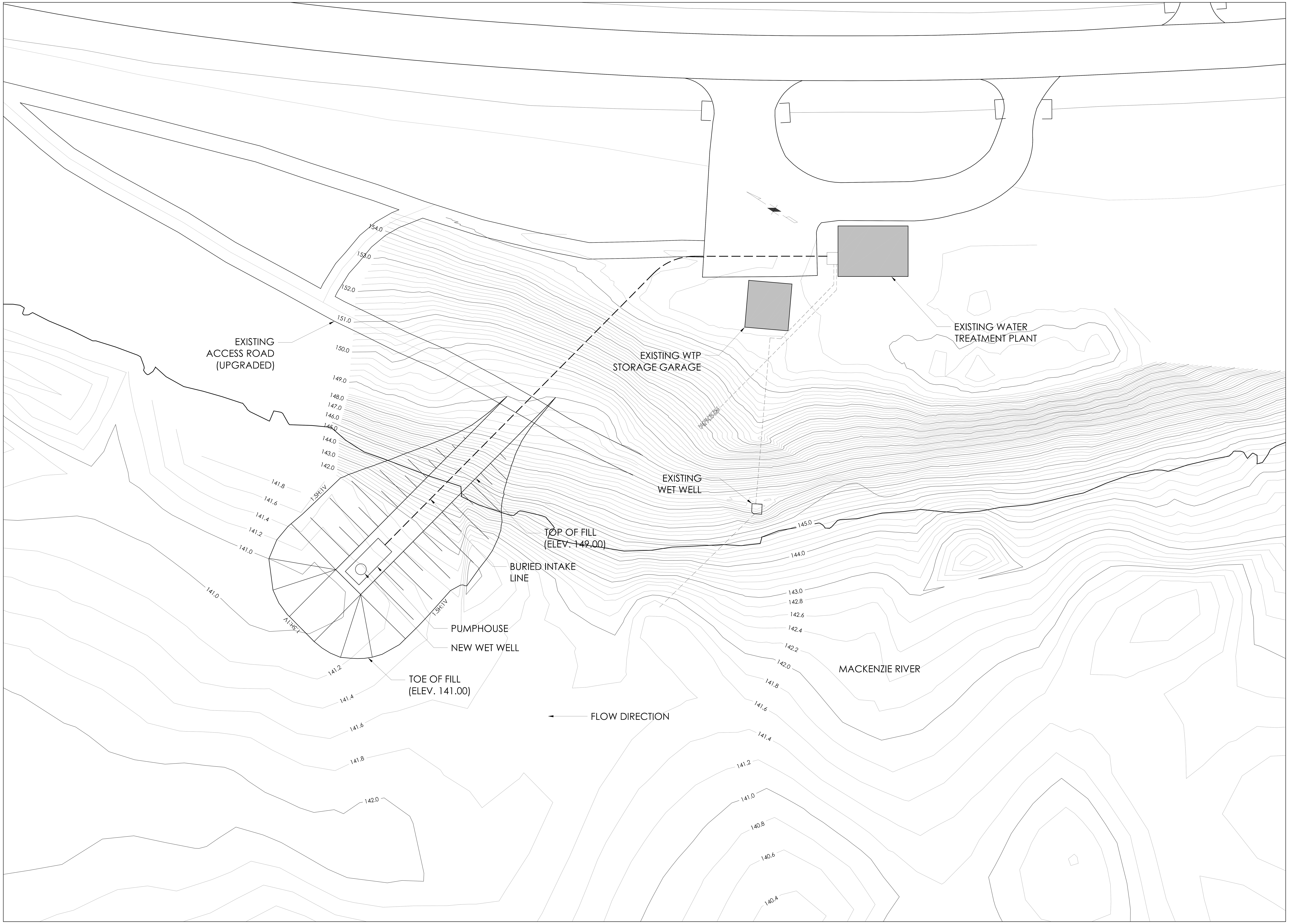
OPTION 1B

DRAWING NO.:
FIGURE 4



1 OPTION 4A SITE PLAN

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OPTION 4A

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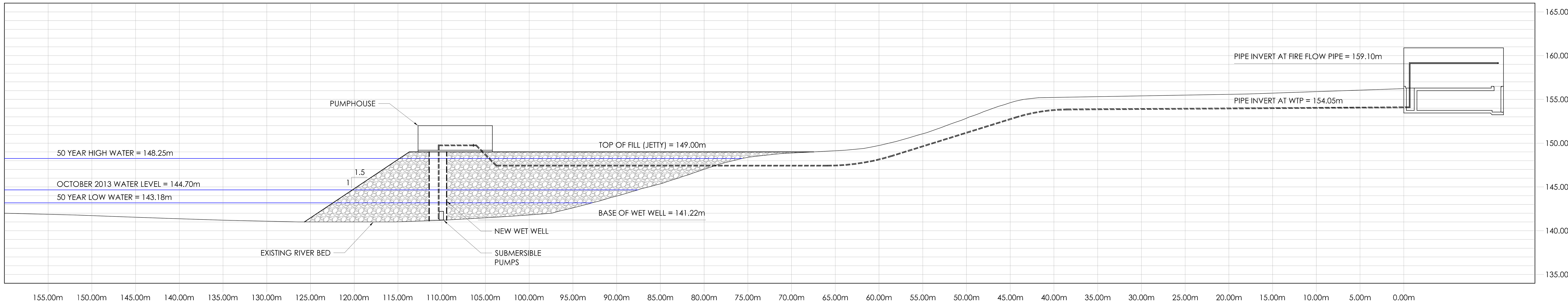
FIGURE 5



1 OPTION 4A PROFILE

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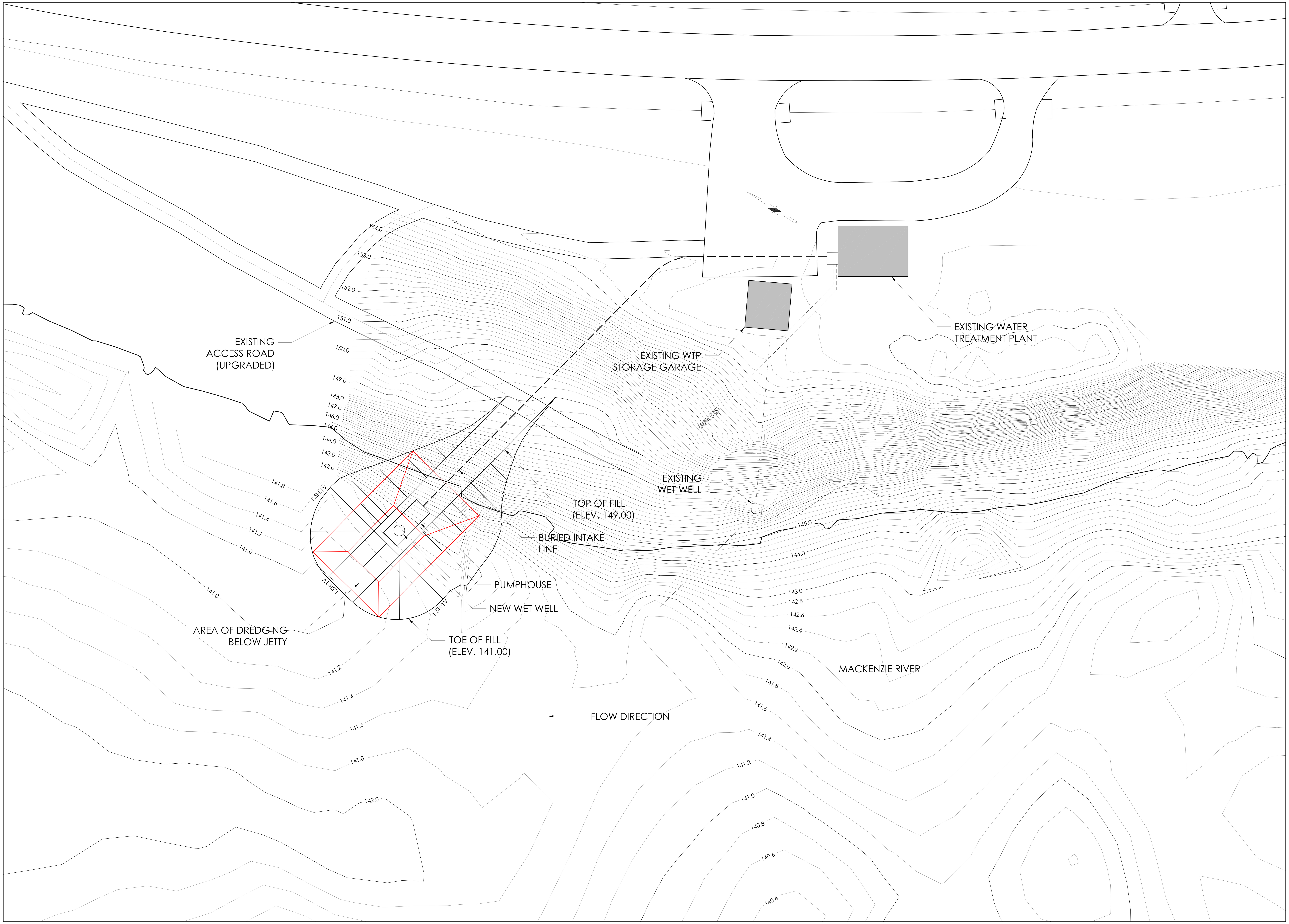
OPTION 4A

DRAWING NO.:

FIGURE 6



1 OPTION 4B SITE PLAN
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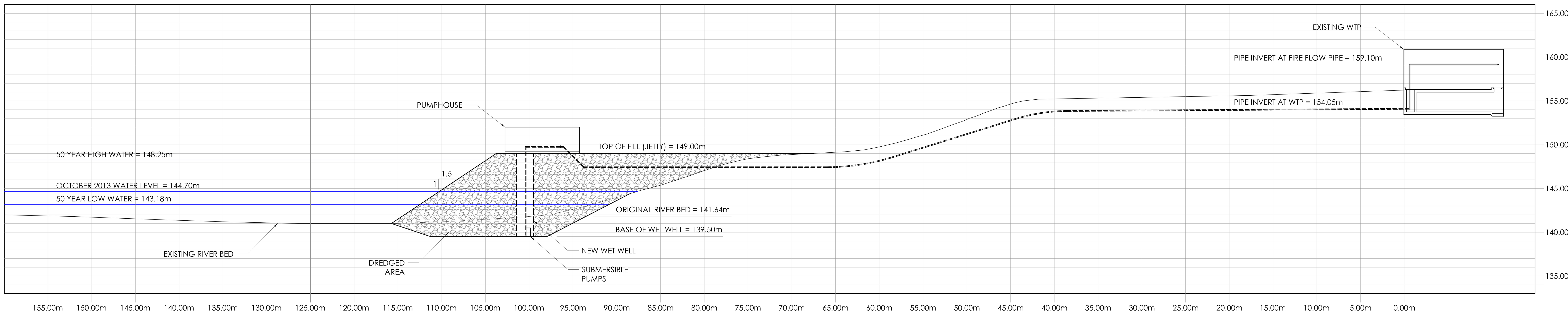
OPTION 4B

DRAWING NO.:
FIGURE 7



1 OPTION 4B PROFILE

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OPTION 4B

DRAWING NO.:

FIGURE 8

APPENDIX B – 2040 WATER STORAGE CALCULATIONS

Fort Providence Potable Water Storage Requirements for 2040 Population and Water Consumption

Current Storage = 172,000 L

Fire Storage

Required Fire Storage = 60,000 L

Emergency Storage

Emergency Storage = 0 L (water source within 3.2 km as per GNWT (2013))

Equalization Storage

2040 Average Day Water Use = 116.15 m^3/day (refer to Table 1 above)

2040 Maximum Day Factor = 2.1 (1.5 maximum day demand factor per GNWT (2013) x 7 days water consumption/5 days delivery)

Maximum Day = 116.15 m3/day x 2.1 = 243.92 m3/day

10% Plant Use = 24.4 m3/day

Assuming water plant treats water 23 hours in a day, the required plant production rate is:

268,320 L/(23 hours x 60 minutes) = 194.4 L/min. (Say 195 L/min.)

Equalization Storage = 268,320 L – [(8 hours x 60 min./hr.) x 195 L/min.] = 175,000 L

Total Recommended Storage

2040 Total Storage Requirement = 60,000 L + 0 L + 175,000 L = 235,000 L

Facility Increase Factor

2040 Cost Increase Factor = 235,000 L/172,000 L = approx. **1.4**

APPENDIX C – COST ESTIMATE

Fort Providence Water Treatment Plant - Intake Options Analysis
OPTION 1a

DIRECT COSTS					
Description	Quantity	Unit	Unit Cost	Unit	Cost
New Intake Lines	240	m	$ 1,000.00	/m	$ 240,000.00
Pump Offtake	120	m	$ 35.00	/m	$ 4,200.00
Intake Tie-in Allowance					$ 50,000.00
Pipe Trench (Excavation)	60	m^3	$ 100.00	/m^3	$ 10,000.00
Pipe Trench (Granular)	15	m^3	$ 90.00	/m^3	$ 5,000.00
Pipe Trench (Dredging)	360	m^3	$ 690.00	/m^3	$ 248,400.00
Pipe Trench (Rip Rap)	360	m^3	$ 225.00	/m^3	$ 81,000.00
Intake System (Screen and Flange)	2	ea.	$ 18,750.00	ea.	$ 37,500.00
Pumps	4	ea.	$ 32,000.00	ea.	$ 128,000.00
Intake System Anchoring					$ 10,000.00
				Sub-total =	*$ 814,100.00*
INDIRECT COSTS					
				Freight and Bridge Toll =	$ 61,057.50
				Lodgings =	$ 61,057.50
				Per Diem =	$ 42,740.25
				Mobilization/Demobilization =	$ 84,855.00
				TOTAL =	**$ 1,063,800.00**
				25%	**$ 1,329,800.00** ±50%

O&M = $ 37,505.00 5% of Direct Capital Cost

Fort Providence Water Treatment Plant - Intake Options Analysis
OPTION 1b

DIRECT COSTS					
Description	Quantity	Unit	Unit Cost	Unit	Cost
New Intake Lines	100	m	$ 1,000.00	/m	$ 100,000.00
Pump Offtake	50	m	$ 35.00	/m	$ 1,750.00
Intake Tie-in Allowance					$ 50,000.00
Pipe Trench (Excavation)	50	m^3	$ 100.00	$/m^3$	$ 10,000.00
Pipe Trench (Granular)	15	m^3	$ 90.00	$/m^3$	$ 5,000.00
Dredging	1260	m^3	$ 460.00	$/m^3$	$ 579,600.00
Rip Rap	360	m^3	$ 225.00	$/m^3$	$ 81,000.00
Intake System (Screen and Flange)	2	ea.	$ 18,750.00	ea.	$ 37,500.00
Pumps	4	ea.	$ 32,000.00	ea.	$ 128,000.00
Intake System Anchoring					$ 10,000.00
				Sub-total =	*$ 1,002,850.00*
INDIRECT COSTS					
				Freight and Bridge Toll =	$ 75,213.75
				Lodgings =	$ 75,213.75
				Per Diem =	$ 52,649.63
				Mobilization/Demobilization =	$ 63,487.50
				TOTAL =	**$ 1,269,400.00**
				25%	**$ 1,586,800.00** ±50%

O&M = $ 46,942.50 5% of Direct Capital Cost

Fort Providence Water Treatment Plant - Intake Options Analysis
OPTION 2

DIRECT COSTS						
Description	Quantity	Unit	Unit Cost	Unit		Cost
New WTP	1	ea.	$ 1,441,000.00	/ea.	$	1,441,000.00
New Intake Lines	240	m	$ 1,000.00	/m	$	240,000.00
Pump Offtake	120	m	$ 35.00	/m	$	4,200.00
Intake Tie-in Allowance					$	50,000.00
Pipe Trench (Excavation)	60	m^3	$ 100.00	$/m^3$	$	10,000.00
Pipe Trench (Granular)	15	m^3	$ 90.00	$/m^3$	$	5,000.00
Pipe Trench (Dredging)	360	m^3	$ 690.00	$/m^3$	$	248,400.00
Pipe Trench (Rip Rap)	360	m^3	$ 225.00	$/m^3$	$	81,000.00
Intake System (Screen and Flange)	2	ea.	$ 18,750.00	ea.	$	37,500.00
Pumps	4	ea.	$ 32,000.00	ea.	$	128,000.00
Intake System Anchoring					$	10,000.00
				Sub-total =	*$*	*2,255,100.00*

INDIRECT COSTS			
Freight and Bridge Toll =	$	169,132.50	
Lodgings =	$	169,132.50	
Per Diem =	$	118,392.75	
Mobilization/Demobilization =	$	301,005.00	
TOTAL =	**$**	**3,012,800.00**	
25%	**$**	**3,766,000.00**	±50%
O&M =	$	37,505.00	5% of Direct Capital Cost

Fort Providence Water Treatment Plant - Intake Options Analysis
OPTION 4a

DIRECT COSTS					
Description	Quantity	Unit	Unit Cost	Unit	Cost
Jetty Fill Materials (Run of Quarry)	2500	m^3	$ 280.00	$/m^3$	$ 700,000.00
Jetty Fill Materials (150mm minus)	2500	m^3	$ 165.00	$/m^3$	$ 412,500.00
Geotextile	950	m^2	$ 48.00	$/m^2$	$ 45,600.00
CSP Wet Well Shaft	8	m	$ 2,500.00	/m	$ 20,000.00
1/4" Plate Wet Well Shaft	8	m	$ 2,500.00	/m	$ 20,000.00
Wet Well Pumps	2	ea.	$ 35,000.00	ea.	$ 70,000.00
6" HDPE piping (insulated and heat traced)	110	m	$ 1,000.00	/m	$ 110,000.00
In Pumphouse Intake Pipes	25	m	$ 175.00	/m	$ 4,375.00
Pipe Trench (Excavation)	240	m^3	$ 100.00	$/m^3$	$ 24,000.00
Pipe Trench (Granular)	52	m^3	$ 87.96	$/m^3$	$ 4,573.92
Concrete Slab on Grade	30	m^2	$ 475.00	$/m^2$	$ 14,250.00
Exterior Walls	75	m^2	$ 530.00	$/m^2$	$ 39,750.00
Roof	30	m^2	$ 850.00	$/m^2$	$ 25,500.00
Mechanical/Electrical Allowance	30	m^2	$ 3,500.00	$/m^2$	$ 105,000.00
Access Road Improvement	180	m	$ 65.00	/m	$ 11,700.00
				Sub-total =	*$ 1,607,248.92*

INDIRECT COSTS		
Freight and Bridge Toll =	$ 120,543.67	
Lodgings =	$ 120,543.67	
Per Diem =	$ 84,380.57	
Mobilization/Demobilization =	$ 241,087.34	
TOTAL =	**$ 2,173,800.00**	
25%	**$ 2,717,300.00**	±50%
O&M =	$ 24,737.45	5% of Direct Capital Cost

Fort Providence Water Treatment Plant - Intake Options Analysis
OPTION 4b

DIRECT COSTS						
Description	Quantity	Unit	Unit Cost	Unit		Cost
Jetty Fill Materials (Run of Quarry)	1840	m^3	$ 280.00	$/m^3$	$	515,200.00
Jetty Fill Materials (150mm Minus)	1840	m^3	$ 165.00	$/m^3$	$	303,600.00
Geotextile	715	m2	$ 48.00	/m2	$	45,600.00
CSP Wet Well Shaft	9.5	m	$ 2,500.00	/m	$	23,750.00
1/4" Plate Wet Well Shaft	9.5	m	$ 2,500.00	/m	$	23,750.00
Intake Pumps	2	ea.	$ 35,000.00	ea.	$	70,000.00
6" HDPE piping (insulated and heat traced)	100	m	$ 1,000.00	/m	$	100,000.00
In Pumphouse Intake Pipes	27	m	$ 175.00	/m	$	4,725.00
Pipe Trench (Excavation)	240	m^3	$ 30.00	$/m^3$	$	7,200.00
Pipe Trench (Granular)	52	m^3	$ 87.96	$/m^3$	$	4,573.92
Concrete Slab on Grade	7.5	m^3	$ 5,000.00	$/m^3$	$	37,500.00
Exterior Walls	75	m^2	$ 700.00	$/m^2$	$	52,500.00
Roof	30	m^2	$ 1,100.00	$/m^2$	$	33,000.00
Mechanical/Electrical Allowance	30	m^2	$ 3,500.00	$/m^2$	$	105,000.00
Access Road Improvement	180	m	$ 65.00	/m	$	11,700.00
Dredging	680	m^3	$ 600.00	$/m^3$	$	408,000.00
				Sub-total =	*$*	*1,746,098.92*

INDIRECT COSTS			
Freight and Bridge Toll =	$	130,957.42	
Lodgings =	$	130,957.42	
Per Diem =	$	91,670.19	
Mobilization/Demobilization =	$	261,914.84	
TOTAL =	**$**	**2,361,600.00**	
25%	**$**	**2,952,000.00**	+/- 50%
O&M =	$	25,964.95	5% of Direct Capital Cost

OPERATIONS AND MAINTENANCE ANALYSIS 1

Option 1a			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Item Description	Cost	Present Value	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039
New Intake Lines (Capital)	$1,329,800.00	1329800	1329800																								
- Flushing	$3,740.00	93500	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740
- Replace Intakes	$37,500.00	75000											37500										37500				
- Replace Two Pumps	$64,000.00	64000													64000												
	Total Present Value =	**$1,562,300.00**																									

Option 1b			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Item Description	Cost	Present Value	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039
New Intake Lines (Capital)	$1,586,800.00	1586800	1329800																								
- Flushing	$3,740.00	93500	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740
- Replace Intakes	$37,500.00	75000											37500										37500				
- Replace Two Pumps	$64,000.00	64000													64000												
- Dredging	$61,410.00	307050					61410					61410					61410					61410					61410
	Total Present Value =	**$2,126,350.00**																									

Option 2			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Item Description	Cost	Present Value	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039
New WTP Building	$2,329,600.00	2436200	2436200																								
New Intake Lines (Capital)	$1,245,300.00	1329800	1329800																								
- Flushing	$3,740.00	93500	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740
- Replace Intakes	$37,500.00	75000											37500										37500				
- Replace Two Pumps	$64,000.00	64000													64000												
	Total Present Value =	**$3,998,500.00**																									

Option 4a			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Item Description	Cost	Present Value	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039
Raw Water Jetty (Capital)	$2,717,300.00	$2,717,300.00	1329800																								
- Flushing	$3,740.00	$93,500.00	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740
- Replace Two Pumps	$64,000.00	$192,000.00								64000								64000								64000	
- Replace Geotextile	$50,000.00	$150,000.00								50000								50000								50000	
	Total Present Value =	**$3,152,800.00**																									

Option 4b			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Item Description	Cost	Present Value	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039
Raw Water Jetty (Capital)	$2,952,000.00	$2,952,000.00	1329800																								
- Flushing	$3,740.00	$93,500.00	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740	3740
- Replace Two Pumps	$64,000.00	$192,000.00								64000								64000								64000	
- Replace Geotextile	$50,000.00	$150,000.00								50000								50000								50000	
	Total Present Value =	**$3,387,500.00**																									

OPERATIONS AND MAINTENANCE ANALYSIS 2

Option	Cost	Annual O&M	Repairs	Total Present Value
1a	$1,329,800.00	$37,505.00	$139,000.00	$1,766,906.92
1b	$1,586,800.00	$46,942.50	$139,000.00	$2,124,650.12
5a	$2,717,300.00	$24,737.45	$342,000.00	$3,070,932.18
5b	$2,952,000.00	$25,964.95	$342,000.00	$3,318,735.47
6	$3,766,000.00	$37,505.00	$139,000.00	$4,203,106.92

	Option 1a			Option 1b			Option 2			Option 5a			Option 5b		
	Capital	O&M	Repair	Capital	O&M	Repair	Capital	O&M	Repair	Capital	O&M	Repair	Capital	O&M	Repair
Present Value	$1,329,800.00	$400,357.48	$36,749.44	$1,586,800.00	$501,100.68	$36,749.44	$3,766,000.00	$400,357.48	$36,749.44	$2,717,300.00	$264,066.70	$89,565.48	$2,952,000.00	$277,169.99	$89,565.48
Yr 0	$1,329,800.00			$1,586,800.00			$3,766,000.00			$2,717,300.00			$2,952,000.00		
1		$34,726.85			$43,465.28			$34,726.85			$22,905.04			$24,041.62	
2		$32,154.49			$40,245.63			$32,154.49			$21,208.37			$22,260.76	
3		$29,772.68			$37,264.47			$29,772.68			$19,637.38			$20,611.81	
4		$27,567.29			$34,504.14			$27,567.29			$18,182.76			$19,085.01	
5		$25,525.27			$31,948.28			$25,525.27			$16,835.89			$17,671.31	
6		$23,634.51			$29,581.74			$23,634.51			$15,588.79			$16,362.32	
7		$21,883.81			$27,390.50			$21,883.81			$14,434.06			$15,150.30	
8		$20,262.78			$25,361.57			$20,262.78			$13,364.87	$53,181.84		$14,028.05	$53,181.84
9		$18,761.84			$23,482.94			$18,761.84			$12,374.88			$12,988.94	
10		$17,372.07			$21,743.46			$17,372.07			$11,458.22			$12,026.79	
11		$16,085.25	$13,143.52		$20,132.83	$13,143.52		$16,085.25	$13,143.52		$10,609.47			$11,135.92	
12		$14,893.75			$18,641.51			$14,893.75			$9,823.58			$10,311.04	
13		$13,790.51	$18,538.52		$17,260.66	$18,538.52		$13,790.51	$18,538.52		$9,095.91			$9,547.26	
14		$12,768.99			$15,982.09			$12,768.99			$8,422.14			$8,840.05	
15		$11,823.14			$14,798.23			$11,823.14			$7,798.27			$8,185.23	
16		$10,947.35			$13,702.07			$10,947.35			$7,220.62	$24,809.72		$7,578.92	$24,809.72
17		$10,136.44			$12,687.10			$10,136.44			$6,685.76			$7,017.52	
18		$9,385.59			$11,747.32			$9,385.59			$6,190.52			$6,497.70	
19		$8,690.36			$10,877.14			$8,690.36			$5,731.96			$6,016.39	
20		$8,046.63			$10,071.43			$8,046.63			$5,307.37			$5,570.73	
21		$7,450.58	$5,067.40		$9,325.40	$5,067.40		$7,450.58	$5,067.40		$4,914.24			$5,158.09	
22		$6,898.69			$8,634.63			$6,898.69			$4,550.22			$4,776.01	
23		$6,387.67			$7,995.03			$6,387.67			$4,213.17			$4,422.23	
24		$5,914.51			$7,402.80			$5,914.51			$3,901.08	$11,573.92		$4,094.65	$11,573.92
25		$5,476.40			$6,854.45			$5,476.40			$3,612.11			$3,791.35	