Associated Engineering

75 YEARS

REPORT

Town of Outlook

Raw Water Supply Study

DECEMBER 2021

A Carbon Neutral Company

BEST MANAGED COMPANIES

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1 BACKGROUND

Associated Engineering (Sask.) Ltd. (AE) has been engaged by the Town of Outlook (Town) to complete a Raw Water Supply Study (RWSS) to assess several options for supplying raw water to the Town.

The Town's Raw Water Intake (RWI) is located on the east bank of a small subchannel in the South Saskatchewan River (SSR) as shown in Figure 1-1 below. In recent years, and due to active sand beds in this stretch of river, the subchannel has experienced periods where it becomes partially cut-off from the river, thus limiting the flow towards the existing StaticOrb fish screen mounted at the mouth of the RWI. In 2019, the subchannel was completely isolated from the river, and the Town was forced to rely on a temporary pumping mechanism that supplied water from the main river body into the subchannel where the RWI and fish screen are located.

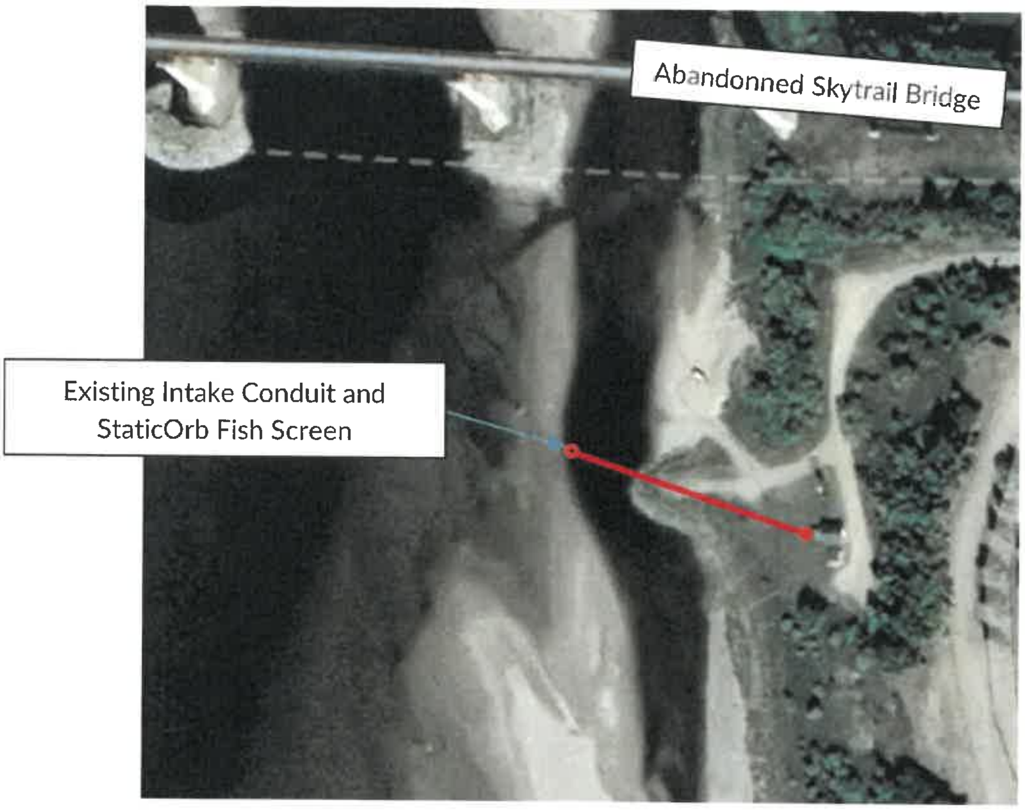

Figure 1-1
Existing Intake Conduit and Pump Station

Northwest Hydraulic Consultants (NHC) was previously retained to complete a preliminary Water Intake Assessment (Assessment) in July 2019. To provide improved and consistent river flow conditions the Assessment recommended a new intake be relocated near the east edge of Pier 3 of the abandoned train bridge, downstream of the existing intake location.

AE assisted the Town with the Investing in Canada Infrastructure Program (ICIP) grant fund application in March 2020 by preparing a Class 'D' cost estimate for the new intake option recommended by the NHC Water Intake Assessment report.

The Town secured a maximum of $5,395,225 in ICIP grant funding for the raw water intake and water treatment plant projects.

Although approved funding is based on the option of a new direct river intake, this conceptual RWSS will assess several different options for servicing raw water to the Town to confirm a preferable option at a desktop level. This study will compare the risks/reliability, costs, and ability to meet projected demands for the following options:

- Sourcing water from Broderick Reservoir or the B2 Canal;
- Groundwater Wells;
- Induced Surface Water Infiltration Wells; and
- A New Direct River Intake (as described in the NHC Water Intake Assessment from July 2019).

2 EXISTING RAW WATER INFRASTRUCTURE

2.1 Fish Screen and Intake Conduit

The Town's existing raw water supply infrastructure is shown in Figure 1-1 and consists of an end-of-pipe StaticOrb fish screen mounted at the mouth of a 600 mm diameter corrugated steel pipe (CSP) that transitions to a 900 mm diameter pipe before tying into the 1800 mm diameter manhole in front of the intake pump station. A 900 mm diameter pipe then conveys raw water from the manhole to the intake pump station wet well. The available record drawings for the intake pump station are appended in Appendix A, Record Drawings for reference.

2.2 River Intake Pump Station and Raw Water Mains

The Town has a River Intake Pump Station located on the east bank of the SSR. The building and wet well have an approximate area of 570 ft^2 and contain two (2) existing 60 HP Western Land Roller turbine pumps that pump into a 200 mm diameter common header. The 200 mm header has a tee connection to a 200 mm diameter raw water main (RWM) that leaves the pump station at the northeast corner of the building and transitions to the 400 mm diameter RWM that conveys raw water approximately 700 m to the Town's WTP on Railway Avenue.

On the other end, the 200 mm diameter common header reduces to a 150 mm diameter pipe, exits the west face of the building, and heads south. The Town Operator indicated that they are unaware of what these lines were previously used for and to their knowledge these lines are currently not in use.

Since the initial construction of the raw water pump house and wet well, a second, shallower, wet well and building expansion has been constructed. The second wet well services the Outlook West Regional Pipeline Association (OWRPA) and Riverview Golf Club. The above-ground building is divided by an intermediate wall into two separate rooms to separate the piping/pumps for OWRPA and Riverview. See Appendix B, Pump Station Photos provided by the Town.

The Riverside pump room is accessed by a door on the east face of the building and is situated further north, near the original wet well. The Riverview pump feeds a 150 mm diameter forcemain that exits the west face of the building and heads south to the golf course. There are no instrumentation and controls to allow for flow or pressure monitoring of the Riverview pump and forcemain.

The OWRPA pump room is accessed by a door on the south face of the building and is situated further south of the original wet well. The submersible pump feeds a pipeline of unknown diameter. The RWM exits the west face of the building and crosses under the SSR as it heads west to convey raw water to the WTP that services the Village of Milden, Milden Hutterite Colony, Dinsmore Hutterite Colony, and approximately 30 farms in the area.

The sketch below depicts the approximate general arrangement of the raw water intake pump station.

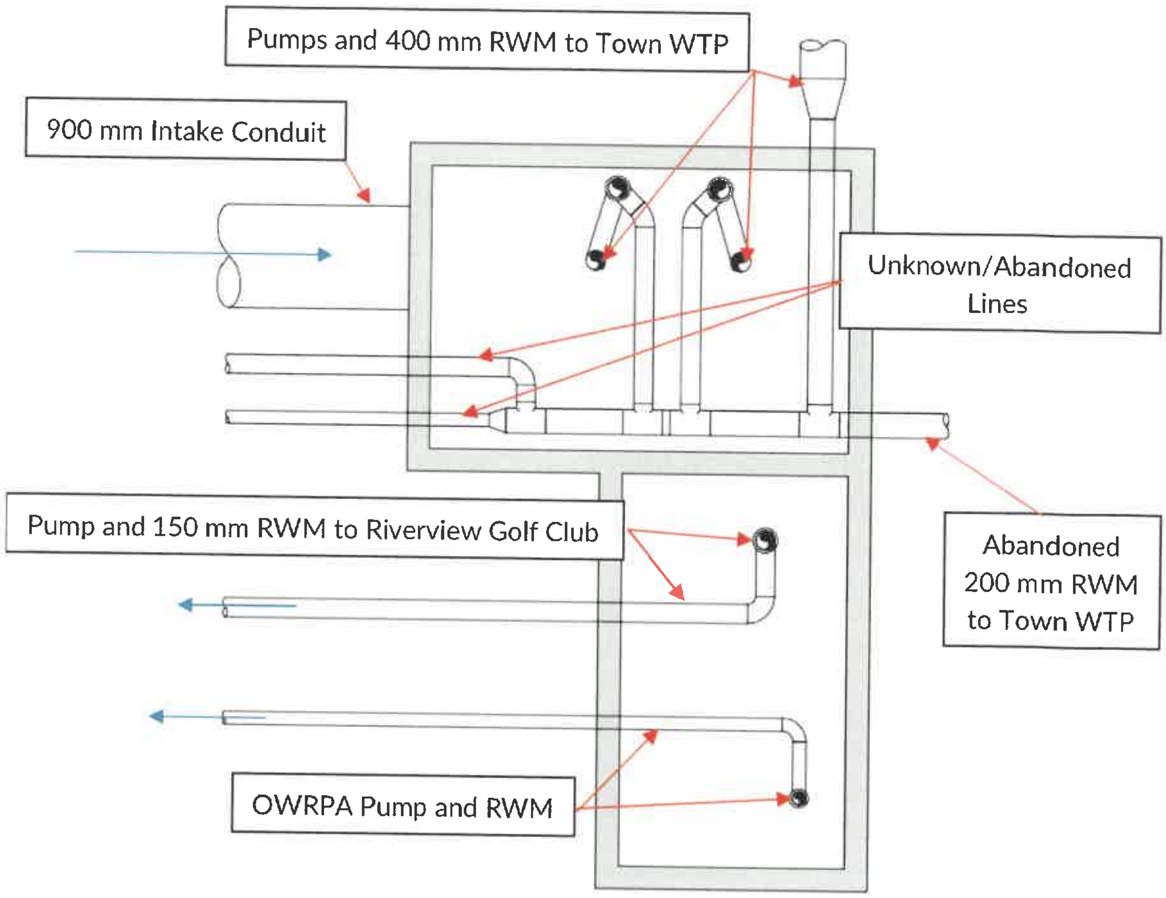

Figure 2-1
General Arrangement
Existing Raw Water Intake Pump Station (Not To Scale)

3 POPULATION AND WATER DEMAND ANALYSIS

The raw water supply for the Town must be able to provide a sufficient supply of water to all residents and businesses in the Town, the Riverview Golf Course at the south end of Town, and users of the Outlook West Regional Pipeline Association (OWRPA) including the Village of Milden, Milden Hutterite Colony, Dinsmore Hutterite Colony, and approximately 30 farms that are also serviced by the OWRPA. The Town also provides treated water to a regional pipeline to the east of Town, these demands are captured within the Town of Outlook demands identified below. Since the population of any community changes with time, it is important to ensure that a water system is capable of providing enough capacity to support a growing population. To predict the community's need for the future, historical population and water consumption data were collected and used to forecast future water usage. More information can be found in the sections below.

3.1 Historical Population and Water Consumption

3.1.1 Town of Outlook

To present an analysis of the historical population for the Town of Outlook, data was collected from the Water Security Agency's (WSA) *Community Water Use Records, Report 33*, and the *2016 Census* from Statistics Canada. WSA Community Water Use Records Reports (Appendix C) utilize a population-based on SaskHealth medical records for individuals receiving treatment in Outlook. Typically, this inflates the population as a portion of individuals utilizing health services in the Town reside in the surrounding agricultural communities and rural municipality and are not serviced with water from the Town's WTP. As such, the Statistics Canada Census data is considered to be more reliable and will be used to project future populations and demands. Table 3-1 shows the historical population with the annual growth rates.

Table 3-1
Historical Population (Town of Outlook)

Year	Census Canada Reported Population[1]	Overall Average Annual Growth Rate (%)
2016	2442[(2)(3)]	0.7
2011	2204	2.7
2006	1938	-
Average	-	1.7

Notes:

1) Data source: *Census from Statistics Canada* 2016
2) The 2016 Census Canada Population has been increased by 163 persons to account for 23 households at Rudy Landing and 42 households on the Eastside Utility that are serviced with treated water from the Town.
3) 2016 Census Canada Average Household size in SK was reported to be 2.5 persons/household. This was used to estimate the population of the 65 households in Rudy Landing and Eastside utility.

The available data from Census Canada indicates that the Town has experienced a moderate population increase since 2006 with an average annual growth rate of 1.7%. Therefore, 2% will be used to forecast growth for this report. This is fairly consistant with other recent studies completed for the Town where an assessment of the population was required, such as the Sewage Lagoon Expansion report completed by AE in 2020 (2.2% growth rate) and the 2018 Infrastructure Capital Plan (1.0% medium growth rate and 3.0% high growth rate).

While not preferred for population statistics, the historical raw water usage from the *Community Water Use Records* is very useful. Historical raw water usage was collected from the *Community Water Use Records* and compared to the WTP records, supplied by the community. Virtually all raw water entering the plant ends up being treated. Raw water demands include backwash water, plant service water, truck fill demands, and distribution system demands.

Average daily consumption is the average volume of water that is consumed by the community, including commercial and industrial use, in a single day for a specific year. This consumption can vary significantly from one year to another based on wet and dry years. Per capita consumption refers to the amount of water that one person typically consumes daily. It is calculated by dividing the average daily consumption by the population using the facility for that year. A review of the water use records between 2006-2019 showed that the average per capita consumption during this period was 503 L/cap-day (LCD) with the 90th percentile value being 618 LCD.

These values are quite high compared to typical consumption rates of 450 LCD for communities similar in size. Since the Town records raw water consumption, an allowance of 10% for plant losses would typically be required which would reduce the 90th Percentile consumption of treated water leaving the plant from 618 to 556 LCD. A value of 620 LCD will be used to project future raw water treatment demands.

Maximum day consumption is the maximum volume of water that is consumed by the community, including commercial and industrial use, in a single day for a specific year. This consumption can vary significantly from one year to another based on wet and dry years. The maximum day factor relates maximum day consumption to average daily consumption. The average maximum day factor over the period 2006-2019 was calculated to be 2.5.

Historical water demands for the Town of Outlook are presented in Table 3-2.

Table 3-2
Historical Raw Water Demands (Town of Outlook)(1)(2)

Year	Population[1]	Raw Water Usage (m^3/year)[2]	Average Day (m^3/day)	Max Day (m^3/day)	Per Capita (L/cap-day)	Max Day Factor
2019	2592	270,953	742	2970	286	4.0
2018	2541	309,593	848	1957	334	2.3
2017	2491	507,377	1,390	4143	558	3.0
2016	2442	455,842	1,249	2901	511	2.3
2015	2435	409,139	1,121	2848	460	2.5
2014	2418	398,865	1,093	2513	452	2.3
2013	2401	407,925	1,118	2417	465	2.2
2012	2384	393,177	1,077	2418	452	2.2
2011	2367	407,925	1,231	2900	520	2.4
2010	2344	436,887	1,197	2427	511	2.0
2009	2281	508,225	1,392	2904	610	2.1
2008	2219	503,447	1,379	3129	622	2.3
2007	2159	478,670	1,311	3234	607	2.5
2006	2101	502,884	1,378	3611	656	2.6
			Average[3]		503	2.5
			90th Percentile		618	2.9

Notes:

1) Data source: *Census from Statistics Canada* 2016. Dates between Census years interpolated based on the average growth rate based on
2) Data Source: WSA Community Water Use Records, Report # 33 (values were adjusted to account for 10% plant losses for those years where treated water usage was reported).

3.1.2 Outlook West Regional Pipeline Association (OWRPA)

OWRPA services the Village of Milden, Milden Hutterite Colony, Dinsmore Hutterite Colony, and approximately 30 farms with potable water from the water treatment plant located on NW 17 29 08 W3M. Raw water is provided to the OWRPA by the Town of Outlook's intake. OWRPA owns and operates a pump within the Town's intake pump station, raw water is pumped via pipeline across the river to the OWRPA plant.

Due to the limited statistical population information available for the Milden and Dinsmore Hutterite Colonies, as well as the roughly 30 farms serviced by OWRPA, it is difficult to determine an appropriate population growth rate for the serviced area. Therefore, for the OWRPA, water usage in the WSA Community Water Use Records Report #33, were analyzed to determine growth experienced in the reported water consumption.

Based on available information, the average daily consumption for OWRPA between 2006-2020 was calculated to be 317 m^3/day (3.7 L/s)with a 90th Percentile average daily consumption of 358 m^3/day (4.1 L/s).

Based on available information, the average maximum day factor over the period 2006-2020 was calculated to be 2.2.

Raw Water usage like average and maximum day consumption will vary from year to year based on wet and dry years. Between 2006 and 2020, the average consumption growth rate for OWRPA was calculated to be 3.2%.

Table 3-3 below summarizes the historical water usage for OWRPA.

Table 3-3
Historical Raw Water Demands (OWRPA)(1)

Year	Raw Water Usage[1] (m^3)	Average Day (m^3/day)	Max Day (m^3/day)	Reported Usage Growth Rate (%)	Max Day Factor
2020	124981	342	543	7.4	1.6
2019	116354	319	634	-6.6	2.0
2018	124620	341	742	-5.9	2.2
2017	132418	363	-	16.6	-
2016	113595	311	751	3.1	2.4
2015	110127	302	789	-14.3	2.6
2014	128533	352	804	7.1	2.3
2013	119973	329	-	-9.1	-
2012	131968	362	-	13.9	-
2011	115857	317	-	1.1	-
2010	114628	314	-	0.1	-
2009	114516	314	-	19.8	-
2008	95621	262	-	-8.5	-
2007	104545	286	-	19.7	-
2006	87355	239	-	-	-
90th Percentile Average		358	-	3.2	2.2

Notes:
1) Data Source: WSA Community Water Use Records, Report # 33

3.1.3 Riverview Golf Club

The Riverview Golf Club (Riverview) is the third entity that relies on the Town's intake and pump station for raw water for irrigation purposes. Riverview Golf Club owns and operates a dedicated pump located in the Town's intake pump station which pumps into a 150 mm diameter forcemain that provides water connected directly to the golf course sprinkler system.

There was little data available for the pump or pipeline that feeds Riverview. The Town's Operator's confirmed that there are no flow metres or pressure gauges on the pipeline to Riverview. Riverview's Operator noted that the golf course uses FLEX800™ (FLX34) sprinklers to irrigate the golf course and that approximately 12 sprinklers run at a time. The Operator also indicated that sprinklers run in the evenings and sometimes in the day, however for the July/August months, Riverview averages 18-hrs/day. For the Riverview raw water demand projection, we have assumed Riverview irrigates the golf course for a maximum of 18-hrs/day.

According to available product data online, each FLX34 sprinkler has a flow output ranging from 71-337 m^3/day (0.8-3.9 L/s). Based on the manufacturer's specified flow output range and the golf course Operator's comments on irrigation run times and quantities, the daily raw water demand for the golf course can range from 635-3,033 m^3/day (7.3-35.1 L/s).

For the demand projections detailed in this report, an average of the minimum and maximum sprinkler flow rates was utilized. The average flow rate per sprinkler, 204 m^3/day (2.4 L/s) was multiplied by 12 sprinklers running simultaneously for 18-hours a day. Based on these assumptions, we estimate an average daily raw water demand of 1,840 m^3/day (21 L/s) for Riverview.

Through discussions with the Water Security Agency (WSA) it was determined that the original water licensing agreement for Riverview allocated 29 irrigable acres for the golf course. The WSA further noted that they utilize an average irrigation volume of 310 – 460 mm per acre when estimating useage for golf courses in SK. Based on the numbers provided by WSA, the the golf club may utilize significantly less raw water than what is estimated based on the Operator's comments. However, infrastructure for raw water supply should still be sized based on maximum scenario demands.

The Town may want to consider adding a flow metre to this forcemain in advance of the 2022 golf season to better assess the actual raw water demands of the golf course.

3.2 Projected Raw Water Demand

The projected water demands for each entity are summarized in the following sections.

3.2.1 Town of Outlook

For the Town of Outlook, the following parameters were used to predict the growth of the community and the related water demands for the population:

- A baseline 2021 population of 2,696 persons (projected from 2,279 persons from Stats Canada 2016 Census and added 163 persons for 65 households across Rudy Landing and Eastside Utility);
- An annual growth rate of 2%;
- A 90th Percentile per capita demand of 618 L/cap-day (based on raw water demand); and
- A maximum day factor of 2.5.

The Town's population and water projections are summarized in Table 3-4.

Table 3-4
Projected Water Demand (Town of Outlook)

Year	2021	2041 (20-Year Design Horizon)
Population[1]	2,696	4,006
Average Day Water Demand (m^3/day)	1,667 (20 L/s)	2,477 (29 L/s)
Annual Water Demand (ML)	609	904
Maximum Day Factor	2.5	2.5
Maximum Day Raw Water (m^3/day)	4,130 (48 L/s)	6,136 (71 L/s)

Notes:
1) Population adjusted by 163 additional residents to account for 65 households at Rudy Landing and Eastside Utility, based on 2016 Census Canada average of 2.5 persons per household in Saskatchewan.

The Town also noted an approximate backlog of 20 services that may be added to the Eastside Utility. A growth rate of 2% should adequately capture the additional 50 persons (2.5 persons per household/service) estimated and is therefore accounted for in the demand projections summarized above.

3.2.2 Outlook West Regional Pipeline Association (OWRPA)

For the OWRPA, the following parameters were used to predict the growth of the water demands for the association.

- A baseline 2021 average day demand of 358 m^3/day (4.1 L/s) (projected from 308 m^3/day (3.6 L/s) reported by the Operator for 2020).
- An annual consumption growth rate of 3.2%.
- A maximum day factor of 2.2.

Table 3-5
Projected Water Demand (OWRPA)

Year	2021	2041 (20-Year Design Horizon)
Average Day Water Demand (m^3/day)	358 (4.1 L/s)	659 (7.6 L/s)
Annual Water Demand (ML)	129	241
Maximum Day Factor	2.2	2.2
Maximum Day Raw Water (m^3/day)	776 (9 L/s)	1,450 (17 L/s)

3.2.3 Riverview Golf Club

Demands for Riverview Golf Club are associated with irrigation of the golf course and are not anticipated to increase. However, it should be noted that these will vary dependent on dry and wet years.

As noted above, the average day water demand for Riverview Golf Club is estimated to be 1,840 m^3/day (21 L/s).

3.2.4 Total Raw Water Supply

The total raw water supply is estimated to be the sum of the three entities that rely on the Town's intake and intake pump station for raw water supply. The various projected demands and total are summarized in Table 3-6.

Table 3-6
Projected Water Demand (Combined)

	Year		
Entity/Demand	**2021**	**2041 (20-Year Design Horizon)**	**Approximate % of Total**
Town of Outlook Maximum Day Raw Water (m^3/day)	4,130 (48 L/s)	6,136 (71 L/s)	65%
Outlook West Regional Pipeline Association Maximum Day Raw Water (m^3/day)	776 (10 L/s)	1,450 (17 L/s)	15%
Riverview Golf Club Average Day Water Demand (m^3/day)	1,840 (21 L/s)	1,840 (21 L/s)	20%
Total Maximum Day Raw Water Demand	6,746 (78 L/s)	9,426 (109 L/s)	-

The historical Mean, Maximum and Minimum surface water flow rates within the SSR during the period between 1911 and 2019 was 247, 885 and 37.5 m^3/s (247,000, 885,000 and 37,500 L/s) respectively, as measured by Environment Canada at their Saskatoon gauging station (#05HG001). The Water Security Agency (WSA) of Saskatchewan has advised that the corresponding mean surface water flow volume within the SSR during this period was 7,699,318 cubic decametres per year, which is equal to an average daily volume of 21,094,023 m^3/day (244,144 L/s). The Town's projected maximum day raw water requirement for all users reliant on the raw water supply system of 9,500 m^3/day (110 L/s) represents only 0.045% of the mean historical surface water flow volume within the SSR.

While the alternatives discussed below assume the continued servicing of raw water to OWRPA and Riverview Golf Club by the Town from one central source, the Town may want to consider options for separating the Town's raw water supply from these customers, particularly if the induced surface water wells option is preferred by the Town. Some of these options are discussed in Section 6, Opinion of Probable Cost.

4 RAW WATER SUPPLY ALTERNATIVES

Although approved funding is based on the construction of a new direct river intake, this conceptual RWSS explores several different options for servicing raw water to the Town to confirm a preferable option at a desktop level. The following sections describe potential concepts for each option, discusses the findings of the various desktop studies, identify potential risks and unknowns, and likely regulatory/approval requirements for each of the following options:

- Sourcing water from Broderick Reservoir or the B2 Canal;
- Groundwater Wells;
- Induced Surface Water Infiltration Wells; and
- A New Direct River Intake (as described in the NHC Water Intake Assessment from July 2019).

4.1 Broderick Reservoir Intake or B2 Canal Intake Turnout

Broderick Reservoir is located approximately ten (10) kilometres east of the Town and about two (2) kilometres south of Broderick. The B2 canal runs north along Ordway Rd, about two (2) kilometres east of the WTP. Broderick Reservoir feeds the B2 canal, Saskatoon Southeast Water Supply System (SSEWSS) and the M2 canal and is owned by the Water Security Agency (WSA) and is operated by the South Saskatchewan Irrigation District (SSRID) which is the largest irrigation district in the province, providing water for pivot irrigation in the farmland surrounding Outlook. The B2 canal is owned and operated by SSRID.

Preliminary discussions with WSA yielded the following information:

- For the Broderick Reservoir Intake option, the Town would need a Water Supply Agreement between the Town and WSA.
- For the B2 canal option, the Water Supply Agreement would involve the Town, WSA, and SSRID.
- WSA is currently completing hydrological modelling of Broderick Reservoir to assess the supply capacity.
  - WSA may consider the Town's projected 20-year design horizon raw water demands in their modelling.

As part of this study, AE contacted the Board of Directors at the SSRID (Board) to request available record drawings and to discuss the feasibility of servicing the Town. It is our understanding that the board briefly discussed the potential of supplying raw water to the Town. The Board noted that to appropriately consider the request and decide, the SSRID would require a formal proposal and a more defined design indicating the various infrastructure components, raw water demands, and any other pertinent information. The Board noted that following receipt of a formal proposal, they still reserve the right to reject the servicing request.

Sourcing from Broderick Reservoir or the B2 Canal has differing considerations and infrastructure requirements. The two options are conceptually described in the following sections.

4.1.1 Broderick Reservoir Intake Option

Drawing raw water directly from Broderick Reservoir would involve an intake conduit with an end-of-pipe fish screen within Broderick Reservoir that would convey water from the reservoir to a raw water pump station near Broderick Reservoir. The raw water pump station would pump raw water through a roughly 12-kilometre RWM to the Town's WTP on Railway Ave. This option would also require an intermediate raw water storage reservoir to provide back-up raw water during Magnicide H dosing events.

The main advantage of drawing water directly from Broderick Reservoir compared to drawing from the B2 canal is that it already has the storage capacity to provide the winter demands when the canals are not in service, and therefore eliminates the requirement for a large intermediate raw water storage reservoir that would be required if drawing directly from the seasonal use B2 canal. The Broderick Reservoir Intake option should still have an intermediate reservoir to store raw water during Magnicide H dosing periods which is anticipated to be significantly smaller than the raw water storage reservoir required for the B2 canal option.

A detailed bathymetric survey of Broderick Reservoir and a hydrotechnical review from NHC is required to confirm whether Broderick Reservoir has adequate depth to accommodate an intake conduit and end of pipefish screens. It should also be noted that dry/drought years like 2021 significantly lower the water level in Broderick Reservoir due to the higher irrigation usage. As such, an intake in Broderick Reservoir may be subject to the same temporary pumping scenarios experienced by the Town's existing intake when river levels in the SSR are low (like 2019). WSA would also need to ensure adequate volume is left in Broderick Reservoir at the end of each irrigation season to maintain supply to the Town and other users like the Village of Broderick and the pig farm that rely on Broderick Reservoir for raw water.

Key infrastructure components for the Broderick Reservoir Intake alternative include:

- Fish screens and intake conduit in Broderick Reservoir.
- A new raw water pump station at Broderick Reservoir.
- A raw water storage reservoir for raw water supply during Magnicide H dosing events.
- 400 mm diameter RWM from the new raw water pump station to the Town's WTP.
- Tee connection to the RWM outside of the WTP to accommodate bypass of raw water down to the Town's existing raw water pump station.
- Upgrades to Electrical, Instrumentation, and Controls (EIC) and flow control valve to control water levels in the wet well of the existing raw water pump station.
- New power and gas servicing to the new pump station; and
- Site access, roadway, and turnaround areas.

A conceptual plan for the Broderick Reservoir Intake Supply option is depicted in Figure C-701 in Appendix D, Figures.

4.1.2 B2 Canal Option

Drawing raw water directly from the B2 canal would involve a screened turnout/control structure between the B2 canal and a new raw water storage reservoir, a pump station at the raw water storage reservoir, and a roughly 3.5-kilometre-long RWM to convey raw water from the storage reservoir to the Town's WTP. RWM piping and valving outside of the WTP would need to be reconfigured to utilize the Town's existing RWM to convey raw water down to the existing raw water pump station where it could then be pumped to the Riverview Golf Club and OWRPA. This would require flow control upgrades to the existing raw water pump station in the SSR valley and the existing RWM to control the water elevations within the existing wet well where the OWRPA and Riverview Golf Club pumps are located.

The B2 canal is drained for the winter on October 1st and is not typically filled until the middle of May the following spring. A raw water storage pond should be sized to store the cumulative average day raw water demand for the Town and OWRPA, approximately 3,170 m^3/day (37 L/s) from October 1st to May 30th (242 days). The total projected storage requirement for these winter months is estimated to be around 770,000 m^3 of raw water. It is assumed that Riverview would require minimal water during October and May of each year and should not be included in the raw water storage requirements. At a depth of 3.5 m, an earthen raw water pond is estimated to need a minimum area of 220,000 m^2 (not accounting for freeboard). A raw water storage reservoir would need to be designed to be impervious to mitigate the exfiltration of raw water from the storage reservoir.

Key infrastructure components for the Broderick Reservoir Intake alternative include:

- Screened intake turnout from B2 Canal/control structure.
- Conduit from B2 Canal to a raw water storage reservoir.
- Lined raw water storage reservoir.
- New raw water pump station to pump from the raw water storage reservoir to the Town's WTP.
- Tee connection to the RWM outside of the WTP to accommodate bypass of raw water down to the Town's existing raw water pump station.
- Upgrades to EIC and flow control to control water levels in the wet well of the existing raw water pump station.
- New power and gas servicing to the new pump station; and
- Site access.

A conceptual sketch for the B2 Canal Intake and Reservoir option is depicted in Figure C-702 in Appendix D, Figures.

4.1.3 Risks and Unknowns

The following summarizes the risks and unknowns associated with both the Broderick Reservoir Intake and the B2 canal options.

- Magnicide H is algae control chemical that is introduced into the M1 canal system upstream of Broderick Reservoir periodically to manage algae. When algae treatment occurs, the water is held within the M1 Canal until the treatment is completed over a seven (7) day period. When dosing occurs, farmers are notified of the outage and that the SSRID will not be supplying water to them at that time. Broderick Reservoir does not receive any water from the M1 Canal during the algae treatment period. The Town would need to rely on the proposed raw water storage reservoir during these periods.
- While historically uncommon, Broderick Reservoir was nearly depleted in 2021 due to the drought conditions faced by the province. This poses a reliability risk and adding the Town's raw water demand to Broderick Reservoir could further increase the risk of such occurrences in future years.
- At the time of writing this report, limited record drawings for Broderick Reservoir and B2 canal were made available to confirm the design and actual conveyance capacities of the B2 canal to confirm if the canal or Broderick Reservoir have adequate capacity to service the Town.
- Sourcing raw water from the B2 canal requires a significantly large raw water storage reservoir. As the water will be sitting in this pond, algae control methods may be required. As well, this creates a significant amount of land that would need to be purchased.

- The Town would need to pursue a more detailed analysis of the B2 Canal option to present a proposal to the SSRID who may still decide to reject the service request.
- Future upgrades to the raw water supply system would need to be planned in conjunction and with the acceptance of the key stakeholders including the SSRID, SaskWater, and the WSA.
- The Town would need to negotiate a reasonable rate for raw water supply with the WSA.
- Broderick Reservoir is classified as a fish-bearing water body and as a result would need DFO review, authorization, and permitting to assess whether a new intake can be installed in the reservoir.

4.2 Groundwater Wells Option

For this study, it was assumed that any groundwater source that was located more than 10.0 kilometres from Outlook would not be an economical alternative when compared to the construction of a replacement surface water intake structure within the South Saskatchewan River (SSR). It is estimated that a 10.0-kilometre 400 mm diameter pipeline may cost between $8M to $13M. In addition, a pump station to pump the raw water from the groundwater wells back to the Town could cost more than $4M. These combined high-level costs are far greater than those of the Induced Surface Water Infiltration Wells and new Direct River Intake options.

Beckie Hydrogeologists Ltd. (BECKIE) completed a literature search and desktop review of the available hydrogeologic information and based on the results of this review, concluded that a groundwater source capable of supplying the Town's projected maximum day raw water requirement of 9,500 m^3 per day (110 L/s) is not available within a 10.0 km radius of Outlook.

The Tyner Valley Aquifer tributary discussed in the following sections would not have adequate capacity to service the projected 20-year design horizon demands as a groundwater-only source and is therefore not considered a viable water supply option for the Town.

Additionally, the Town's existing water treatment plant is designed to treat surface water, not groundwater.

Therefore, the development of a groundwater source is not a preferred option for the Town of Outlook and was not assessed further.

4.3 Induced Surface Water Infiltration Wells Option

In areas where the local geologic and hydrogeologic conditions are suitable, the construction of properly designed and located induced surface water infiltration wells will facilitate the extraction of naturally filtered and sediment-free surface water from an adjacent river or other surface water body. This negates the requirement for a direct surface water intake structure.

Some of the advantages induced surface water infiltration wells constructed adjacent to the SSR near Outlook are listed below:

- The construction and future operation and maintenance of induced surface water infiltration wells and the associated ancillary works and appurtenances would not require any activity within the SSR itself, which would result in significant regulatory, construction, and operational advantages.
- Induced surface water infiltration wells can be constructed, operated, and maintained year-round.
- Induced surface water infiltration wells can be operated during low water levels and flow conditions within the SSR and during freezing weather conditions, although the individual pumping rates may be reduced during these periods. Redundant/supplementary wells could be installed to compensate for reduced pumping ratesduring these periods. Redundant/supplementary wells have not been considered in the opinion or probable cost presented later in this report.
- Induced surface water infiltration wells would be developed within granular sediments that have a direct hydrogeologic connection with the SSR. The suspended sediments and organic material generally entraned in the surface water in the SSR would be removed through natural (in-situ) filtration within the granular aquifer sediments before the induced surface water reaches the producing wells.
- The production of clean sediment-free surface water from induced surface water infiltration wells should reduce the operation and maintenance costs of the raw water supply system and increase the overall service life of the pumping and water treatment equipment.
    - When AE completed the Infrastructure Capital Plan in 2018, the Town noted that the existing intake in the SSR results in sedimentation issues at the WTP which impact water quality. This also leads to dredging work at the existing intake every ten (10) years. Induced surface water wells would mitigate sedimentation and water quality issues at the WTP.
- The combined maximum day production capacity of an induced surface water infiltration wellfield is generally determined by the number of wells that are constructed within the wellfield. Depending on the characteristics and areal extent of the developed aquifer (to be determined by test drilling), wellfield expansion should be possible as the Town's raw water requirements increase over time.

4.3.1 Potential Locations of Induced Surface Water Infiltration Wells near Outlook

Potential locations for induced surface water infiltration well fields are described in the following sections.

4.3.1.1 Alluvial Sand Deposits (Aquifer)

Installing the induced surface water wells in any of the three areas identified in Figure C-703 in Appendix D, Figures would include the following components:

- Water wells, pitless adapters, removable ice guards, pumps, variable frequency drives (VFDs), EIC cables/controls.
- Submersible flow metres and flow control/isolation valaves in manholes adjacent to each well.
- 400 mm diameter RWM to the existing intake pump station and RWM tie-ins from each individual well to the 400 mm RWM header.
- Site access to the well field and to the individual wells.
- Slope instability monitoring equipment.

It appears that alluvial sands, potentially suitable for the development of induced surface water infiltration wells, are present on both sides of the SSR near the Town's existing surface water intake structure, at locations directly west of the Town's existing intake, along the golf course, and south of HWY 15 along the east bank. In particular, P. Machibroda Engineering Ltd. (PMEL) notes in their desktop geotechnical review that the surface water infiltration wells should be located within the flood plain of the river to avoid the river valley slopes which consist of predominantly, low permeability glacial till. The area along the golf course, the sand bar directly west of the Town's existing intake, and the sand bars south of HWY 15 along the east bank of the SSR are expected to be stable and not to be impacted by the landslide experienced by the eastern river slope near the existing raw water intake infrastructure. Refer to Figure C-703 in Appendix D, Figures.

Should the preliminary hydrotechnical test drilling and observation well(s) installation programs prove successful, wells installed in the areas noted would not be at risk of damage or failure due to sheer or tilting/uplift caused by the landslide. Therefore, slope instability monitoring would be limited to monitoring slope movement for the RWM conveying raw water from the well field to the existing intake pump station. There are several options for the required RWM which should be assessed in the next stages of design. Consideration should be given to routing the RWM along HWY 15 and north along McKenzie St to the WTP on Railway Ave. This route would further limit slope instability to a small stretch of the RWM.

PMEL's detailed desktop/visual assessment titled Geotechnical Review New Raw Water Supply Study East Bank of South Saskatchewan River Town of Outlook, Saskatchewan (PMEL Project No. 18533) is attached in Appendix E, PMEL Report.

With reference to the September 6, 1994 report that was prepared by E.A. Christiansen Consulting Ltd. (#0150-002) prior to the construction of the existing Highway 15 bridge, the alluvial sediments that underlie the SSR appear to extend to a depth of 25 metres. The reported sand depth of 25 metres indicates good potential for the successful construction of induced surface water infiltration wells adjacent to the SSR near Outlook. However, the depth and characteristics of the alluvial sands and the suitability of these sands for the development of induced surface water infiltration wells must be evaluated through a preliminary test drilling and observation well(s) installation program and subsequent hydrogeologic assessment. The three (3) areas proposed for the preliminary test drilling and observation well(s) installation work (referenced as sites A, B and C)are illustrated in Figure No. C-703 in Appendix D, Figures.

The pumping capacity of induced surface water infiltration wells previously constructed under the direction of BECKIE for other similar projects has ranged from <500 to 2,000 m^3 per day. For this study and subject to the results of the preliminary test drilling and observation well(s) installation work, BECKIE has estimated that an induced surface water infiltration well installed within the alluvial aquifer adjacent to the SSR near Outlook may have an individual maximum day pumping capacity of approximately 750 m^3/day (8.7 L/s).

Based on an assumed maximum day pumping capacity of 750 m^3/day (8.7 L/s) and the 20-year design horizon raw water demand projection of 9,500 m^3/day (110 L/s), the Town could need 13 induced surface water infiltration wells if installed in alluvial sands. As stated earlier, the final number of wells would need to be confirmed by the preliminary drilling and pump testing investigations.

At the assumed well(s) capacity and current raw water demands, the Town would initially need to construct 9 wells to service themselves, OWRPA, and Riverview. Additional wells could be added as demand increases in the coming years and/or to provide redundancy to the raw water supply system..

4.3.1.2 Tyner Valley Aquifer North of Outlook

It appears that an east-west trending tributary of the Tyner Valley Aquifer (TVA) intersects (underlies) the SSR approximately 1.5 kilometres north of Outlook; the inferred north and south boundaries of this tributary are illustrated in the figure below. The TVA tributary is not a main channel of the aquifer and would not be able to service the projected 20-year design horizon demands as a groundwater only source.

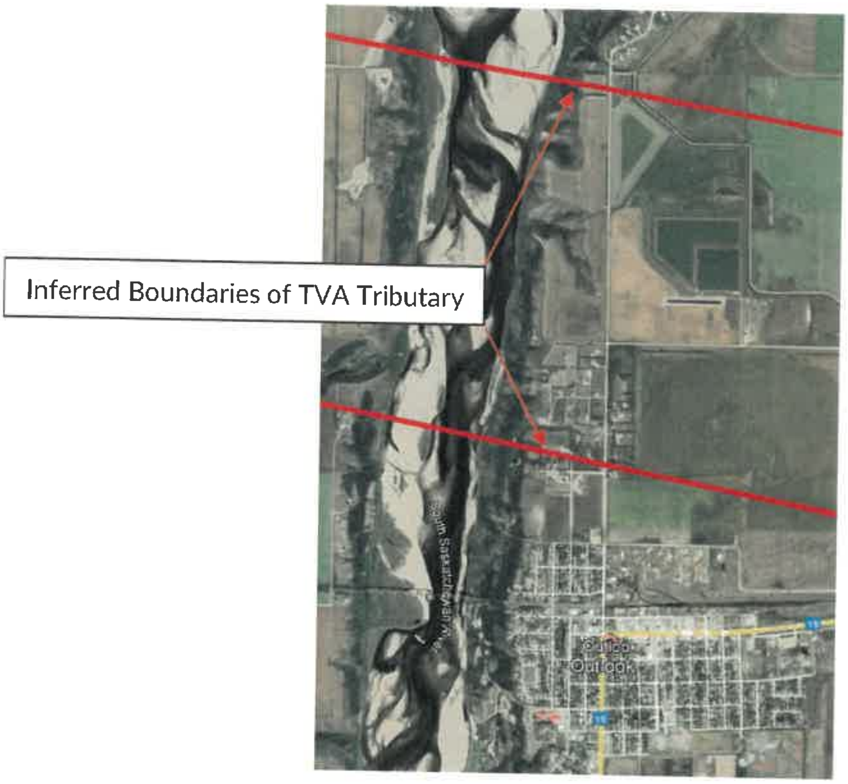

Figure 4-1
Inferred Boundaries of Tyner Valley Aquifer Tributary

Installing the induced surface water wells on the eastern bank of the SSR within the inferred TVA zone depicted above would require a new pump station that would pump raw water from the wells to the water treatment plant and include the following components:

- Water wells, pumps, pitless adapters, removable ice guards, EIC cables/controls.
- Discharge structure to outlet groundwater captured by the interceptor wells into the SSR.
- A conveyance pipeline from the interceptor wells to the groundwater disposal point in the SSR.
- An all-weather access road into the river valley.
- RWM from each well to a common header pipe.
- A new raw water pump station/wet-well to pump water from the wells to the WTP.
- Raw-water pipeline to convey water from the new pump station to the WTP.
- Piping adjustments outside the WTP to utilize the existing RWM for conveying raw water down to the Town's existing raw water pump station to feed the OWRPA and golf course pumped systems.

- Electrical, Instrumentation, and Controls upgrades at the existing raw water pump station to control raw water flow coming down the river valley into the existing raw water pump station wet well.
- A separate well(s) control building may also be required.

As with the alluvial sand aquifer, the depth and characteristics of the TVA and the suitability of this aquifer for the development of induced surface water infiltration and interceptor wells must be evaluated through preliminary test drilling and an observation well(s) installation program, and subsequent hydrogeologic assessment.

For this study and subject to the results of the preliminary test drilling and observation well(s) installation work, BECKIE has estimated that an induced surface water infiltration well installed within the TVA may have an individual maximum day pumping capacity of approximately 1,250 m^3/day (14.5 L/s).

In terms of the total dissolved solids (TDS) concentration, which is a measure of the overall water quality, the water quality in the TVA is expected to be much poorer than in the SSR. Therefore, induced surface water infiltration wells within the TVA may have to be constructed in pairs, on a line perpendicular to the SSR and spaced approximately 10 metres apart. The wells closest to the SSR would be the induced surface water infiltration or supply wells, and the wells furthest from the SSR would intercept the poorer quality groundwater; the poorer quality groundwater would be pumped back into the SSR.

A supply/interceptor well(s) system was successfully developed within the TVA at a location south of the Town of Eston, Saskatchewan. This well(s) system replaced a direct surface water intake in the SSR and has now been successfully operated by the Eston-Kindersley Water Administration Board for the past 40 years. The groundwater from the interceptor wells is pumped back into the SSR through the old direct intake structure.

A schematic cross-section previously (2013) prepared by Beckie to illustrate the supply well and interceptor well(s) process within the Eston-Kindersley well field is included as Figure No. 1 (Schematic Cross Section Showing Induced Infiltration and Shield Well Interception Processes) in Appendix D. Figures.

Based on the estimated maximum day pumping capacity of 1,250 m^3/day (14.5 L/s), and the 20-year design horizon raw water demand projection of 9,500 m^3/day (110 L/s), the Town could need 16 wells (8 productions and 8 shields) if induced surface water infiltration wells were to be drilled into the TVA. As stated earlier, the final number of wells would need to be confirmed by the preliminary drilling and pump testing investigations.

4.3.2 Risks and Unknowns

Hydrotechnical: Hydrotechnical risk and disadvantages associated with the development of all induced surface water infiltration well fields include:

- The characteristics of the target aquifer(s) and the suitability of the aquifer(s) for the development of induced surface water infiltration wells must be evaluated through a preliminary test drilling and observation well(s) installation program and subsequent hydrogeologic assessment.
- The maximum day pumping capacity of the well(s) cannot be determined until at least one test/supply well has been installed and the pump tested.
- Multiple wells operating simultaneously would be required to supply the Town's projected maximum day raw water requirement of 9,500 m^3 per day (110 L/s).

- The wells are subject to ice damage during the spring thaw. This risk can be mitigated with removable ice guards placed around each well; a photograph of a typical ice breaker is included in Appendix D, Figures.
- The well sites may be subject to local bank erosion over time. Design considerations to mitigate this risk should be assessed in the preliminary and detailed design stages.
- The efficiency and the maximum day pumping capacity of all water wells will decrease with normal operation over time as the openings in the intake screen and in the surrounding sediments become partially plugged due to normal chemical, biological and mechanical processes within the developed aquifer; regular and effective rehabilitation work is required to maintain well efficiency and pumping capacity.Mechanical rehabilitation work is required on induced infiltration wells every ± 3 years, which is more frequent than traditional groundwater wells. The estimated average cost of well rehabilitation work is $7,000 per well per year.
  - If the rehabilitation work is completed concurrently on multiple water wells, the fixed costs for items such as personnel/equipment mobilization and hydrogeologic analyses and reporting would be reduced on a cost per well basis. The estimated average cost of the rehabilitation work could then be reduced to $18,000 per well, or $6,000 per well per year.
- Land access and control are required for each well site.
- Water produced from induced surface water infiltration wells might be GUDI positive (groundwater under the direct influence of surface water). This should not be a significant risk for the Town since they already operate a surface water supply and treatment system. Once the water quality within the developed aquifer has stabilized, the water produced from the wells should be chemically similar to the surface water within the SSR.
- The potential risk of premature and irreversible plugging of the well intake screens and/or of the surrounding aquifer sediments due to over-pumping. This risk is mitigated by adherence to the recommended (post-construction) operational and maintenance procedures to ensure wells aren't being over-pumped.
- During a significant flood event, the wells may be temporarily inaccessible until floodwaters subside. However, the wells could continue to operate and there would be no risk to the mechanical/electrical equipment from flooding as this would be designed to be submersible.
- A new lift station and RWM from the well field directly to the WTP would be preferable to a new RWM from the well field to the existing intake pump station. This cost/benefit analysis of a new lift station from the well field directly to the WTP should be considered during preliminary design. While this would increase project capital costs, a new lift station and RWM would allow the Town to further limit slope instability risks for this option to a smaller segment of the RWM.

Specific hydro-technical risks associated with installing a well field within TVA include:

- Separate regulatory approval will be required for pumping groundwater back to the SSR from the interceptor (shield) wells.
- If this option is pursued, the preliminary test drilling and observation well installation work would be conducted in an inferred section of the TVA that is upstream of the outfall outfall from the Town's existing sewage treatment lagoons. However, depending on the aquifer contidions encountered, some of the water wells may have to also be located downstream of the outfall. . It should be noted that a 2020 Preliminary Sewage Treatment Lagoon Expansion study completed by AE discussed the likelihood of leakage occurring out of the northern cell of the existing lagoon. This may impact the groundwater quality in the shield wells.
- The wells would be located downstream of the assumed discharge point from the lagoon operated by LCBI (private college). It is unknown if this discharge reaches the SSR. The Town indicated that it may be possible to re-direct the LCBI effluent into the Town's sanitary system to mitigate this risk.

- PMEL noted that the eastern bank of the SSR within the TVA appears to have a more active slope compared to the location of the existing intake and further upstream. This introduces stability concerns and potential risks to the wells and other infrastructure.
- While the volume of effluent disposal or leakage into the TVA and/or into the SSR would be minimal relative to the average historical surface water flows within the SSR, the public perception of this could present a risk.

Geotechnical: PMEL completed a desktop/visual slope assessment for the induced surface water infiltration wells alternative. The assessment notes the following:

- Geotechnical issues would be similar to that of a new river intake concerning impacts from slope movement if induced surface water infiltration wells are installed near the site of the existing intake.
- If the well field is installed at the site of the existing intake pump station, the wells may experience distress. The distress could involve the shearing of the wells if they intersect the slip surface as well as heaving/tilting from upward movement at the toe of the landslide.
- The potential well field locations identified above may limit the slope instability risk to the RWM only. Since the induced surface water infiltration wells would be installed in the flood plain of the river (stable ground), the wells would not be subject to tilting or uplift from the landslide.
- Due to the ongoing slope movement of the SSR's east bank, any infrastructure on the slope is at risk of experiencing distress and damage.
- Buried RWM from the wells in any of the three locations presented above to the pump station may experience breakages, particularly if the buried pipes cross between different landslide blocks (locations with tension cracking). Accordingly, the new RWM from the well field to the existing intake pump station should be monitored closely and designed to accommodate slope movements to the extent possible. For the piping, monitoring could include instrumentation to monitor the slope movement and stresses on the pipe. Ongoing monitoring of the infrastructure may enable the Town to repair the piping before breaks occur.
- Because the wells would be installed within the lower river terrace (flood plain), the ground surface at the well sites may be subject to slumping and/or bank erosion over time. Erosion control measures around the well field would be considered the final design to mitigate potential degradation.

If this alternative is preferred, in the next stages of design, PMEL recommends test drilling and a detailed geotechnical investigation to confirm the anticipated stability of the three areas identified, soil conditions, and to better establish elevations for the slip plane near the existing intake pump station where the RWM from the well field would tie-in.

4.4 New Direct River Intake Option

A new direct river intake would replace the existing intake conduit and end-of-pipe fish screen with a new pipe and fish screen at a more suitable location within the SSR. Northwest Hydraulic Consultants (NHC) was retained to complete a Water Intake Assessment in July 2019. The assessment recommended the intake be relocated to a point near the east edge of Pier 3 of the abandoned train bridge downstream of the existing intake location.

Key infrastructure components for this option include:

- New end-of-pipe fish screen and airburst system.
- New intake conduit from fish screen structure in SSR to existing manhole outside of the existing raw water pump house.

4.4.1 Fish Screening

Records indicate that the existing fish screen opening sizes are not compliant with the current DFO *Freshwater Intake End-of-Pipe Fish Screen Guideline* (Guideline)[1] of 2.5 mm (0.1 in.), applicable to intakes with capacities of 12,960 m^3/day (150 L/s) or less. The new fish screen would need to meet the DFO Guideline of 2.5 mm slot openings which indicates a larger fish screen is required to provide adequate surface area to maintain the projected raw water demands for the Town. The DFO Guideline suggests that fish screens should be sized to satisfy approach velocities of approximately 0.11 m/s and 0.038 m/s for the subcarangiform and anguilliform groups of fish respectively. At this time, a Fish and Fish habitat Study has not been completed and therefore the conceptual fish screen sizing is based on the more conservative anguilliform fish group (0.038 m/s approach velocity).

It is anticipated that three (3) in-river 600 mm diameter static fish T- screens would be able to meet the Town's projected water demands highlighted in earlier sections.
To prevent blockage of the slot openings by various debris in the river, an air backwash system can be provided with the T screens. The automated airburst system consists of an air compressor tank (typically located at the intake pump station), piping from the compressor to the T screen, and the air-burst manifold piping contained within the T screen. The air-burst system is programmed to periodically send a high-pressure burst of air outwards from inside the fish screen to clear any debris that may have been trapped onto the surface of the screen causing partial blockage.

The fish screens and support structure would be supported on a concrete pad, installed in the preferred location within the river. Consideration during detailed design should be given to providing some form of protection for the in-river static fish screens to prevent damage from larger debris such as logs and ice that may be floating down the SSR.

4.4.2 Intake Conduit

The existing records indicate that the current intake conduit is a 600 mm diameter corrugated steel pipe (CSP) that transitions to a 900 mm diameter pipe before tying into the 1800 mm diameter manhole in front of the intake pump station.

When designing an intake, the conduit should be sized to maintain a minimum velocity of 0.5 m/s and a maximum velocity of 1.1 m/s to prevent sediment deposition and scouring respectively. Based on a 0.8 m/s velocity and a 20-year design horizon raw water demand of 9,500 m^3/day, the existing CSP conduit should be replaced with a new 450 mm diameter High-Density Polyethylene (HDPE) DR 11 pipe from the existing manhole structure on the east bank of the SSR to the proposed location of the new in-river static fish screens.

4.4.3 Risks and Unknowns

At the time of this report, the following list of unknowns presents risks to the preliminary option presented in the NHC Water Intake Assessment (July 2019).

[1] Freshwater Intake End-of-Pipe Screen Guideline

Hydrotechnical:

- The hydro-technical report generated by NHC as well as the intake concept presented therein is to be considered preliminary.
- In preparation for this study, AE discussed with NHC the opportunity of locating the intake fish screens at a different scour hole present directly west of the existing intake pump station, adjacent to the edge of the manmade spur into the river from the western bank of the SSR.
- Additional design work including a staged discharge curve for the river levels is needed to finalize this proposed intake concept and may result in variations from what has currently been presented.
- A detailed river hydrology report with a definite design concept in line with a detailed hydrological study should be completed at the earliest opportunity.

Geotechnical: PMEL completed a desktop/visual slope assessment for the new river intake alternative. The assessment notes that the geotechnical issues with the new direct river intake would be related to construction in the river and the long-term impacts of infrastructure on the slope.

PMEL indicates that the long-term issues with the new direct river intake would relate to slope movement impacting existing and new infrastructure on the slope.

- Since the SSRs east bank is experiencing ongoing slope movement, any infrastructure on the slope is at a high risk of experiencing distress and damage.
- At-grade structures could experience differential movement/damage (tilting), similar to the abandoned rail bridge.
- The raw water intake conduit from the river to the intake pump station may experience breakages, particularly if the buried conduit crosses between different landslide blocks (locations with tension cracking).
    - This could lead to costly in-river repair work if a breakage were to occur.
    - Accordingly, any new infrastructure associated with the new direct river intake including the intake conduit, screen, and base structure should be monitored closely and designed to accommodate slope movements to the extent possible.
- For the intake conduit, monitoring could include instrumentation to monitor the slope movement and stresses on the pipe. Ongoing monitoring of the infrastructure may enable the Town to repair the intake conduit before breaks occur.
- The Town may also consider installing slope monitoring instrumentation on existing raw water supply infrastructure like the existing pump house/wet-wells and approach manhole on the raw water conduit.
- PMEL notes that the soil conditions within the river channel are anticipated to consist of a thick layer of sand and silt based on borehole logs from previous projects in the river valley near the HWY 15 bridge. The construction of a cofferdam in sandy soils will need to consider seepage measures to prevent seepage flow around/under the cofferdam.
    - A geotechnical investigation and design should be undertaken by a qualified geotechnical engineer with experience in cofferdam designs.
    - To account for this unknown, we have assumed sheet piling around the extents of the cofferdam would be required.

PMEL's detailed desktop/visual assessment titled Geotechnical Review New Raw Water Supply Study East Bank of South Saskatchewan River Town of Outlook, Saskatchewan (PMEL Project No. 18533) is attached in Appendix E, PMEL Report.

Abandoned Rail Bridge: The rail bridge north of the Town's existing intake was abandoned several years ago and has been closed to public access due to the risk of the bridge collapsing. AE completed the bridge assessment and determined that the slope instability of the eastern river valley has resulted in Pier 1, near the eastern shoreline tilting. The tilt has caused the buckling of bridge structural steel members that should be in tension. It is anticipated that when it fails, the bridge failure will be catastrophic, in that entire sections of the bridge will collapse. Placing a new intake/fish screen structure within the vicinity of the bridge creates the risk of damage occurring due to falling debris, as well as risking being cut-off from water if a collapse occurs.

It should also be noted that if bridge decommissioning is planned at a future date, the cofferdam required to decommission the bridge would likely cut off the new intake from the river, resulting in the need for temporary pumping for raw water supply during the decommissioning stage.

As stated earlier, AE discussed with NHC, alternative locations within the SSR that may be suitable for a direct river intake. NHC noted that there appears to be a scour hole on the west side of the SSR, adjacent to and downstream of the spur that extends into the river from the western shoreline. NHC noted that this location would need to be explored in greater detail during future stages of design. Placing the intake fish screens near the existing spur would provide easier access for potential future maintenance as compared to if the screens are situated in the location identified in the preliminary NHC report. Placing the intake end-of-pipe fish screens at this location would require a longer river crossing RWM conduit that may be installed by open-cut/cofferdam or trenchless options dependent on the geotechnical investigation.

In addition to the above-noted uncertainties, the impact of potential upstream facilities/discharges should be considered. A few of these are noted below.

- Riverview Golf Club is located a short distance upstream from the proposed intake.
- Progress Ave stormwater outfall (main outfall from the Town) is located roughly 600 m upstream of the proposed intake.
- Tufts Crescent stormwater outfall is located roughly 550 m upstream of the proposed intake.
- Skytrail stormwater outfall located adjacent to the proposed intake.

As noted in the 2018 Infrastructure Capital Plan completed by AE for the Town, a formal assessment of the raw water pump station should be completed to assess the condition of the existing facility. Should the existing pump well and pump station be required for the preferred alternative, additional assessment should be completed to review the condition and remaining service life of the existing facility as well as the risk associated with bank instability. Should the existing facility require replacement, a new intake pump station could be installed at the end of the spur on the west side of the river with consideration for a new RWM installed by Horizontal Directional Drilling (HDD) under the landslide slip plane on the east bank of the SSR to the top of the bank, and away from the top of the slip plane.

5 REGULATORY AND PERMITTING CONSIDERATIONS

The following studies, timing windows, offsetting costs, permitting and approval timings would apply to the various alternatives:

Table 5-1
Regulatory/Permitting Considerations for Broderick Reservoir Intake and B2 Canal

Regulator	Study And/Or Application	Timing	Description
Water Security Agency (WSA)	Aquatic Habitat Protection Permit (AHPP)	Preliminary Design	WSA will provide a permit to allow work in the wetted area of waterbody and to provide provisions to minimize the impact on water quality.
WSA	Water Rights License (WRL)	Detailed Design	WSA will provide a license to allow the constructed system to be commissioned for regular service.
WSA	Approval to Construct and Operate Works (ATC/ATO)	Detailed Design	WSA will provide a permit to construct the raw water supply works and to begin operation upon completion of construction.
WSA	Permit to Conduct Ground Water Investigation[1]	Preliminary Design	WSA will provide a permit to allow the preliminary test drilling and observation well(s) installation work and if this work is successful, the subsequent construction and pump testing of the water supply wells..
Ministry of Environment (MOE)/WSA	Approval to Discharge Groundwater[2]	Detailed Design	MOE/WSA may need to provide approval to discharge groundwater from the interceptor wells into the SSR.
Oceans and Fisheries Canada (DFO)	Fish/Fish Habitat Study[3]	Conceptual Design	Study of the fish and fish habitat that will govern the design of the fish screen and impacts/mitigations required to minimize the impact on fish and fish habitat. This study would be used in
DFO	DFO Review	Preliminary Design	DFO will review the Project activities to determine

Regulator	Study And/Or Application	Timing	Description
			compliance with the *Fisheries Act*
DFO	DFO Authorization	Detailed Design	If it is determined that harm to fish and fish habitat could occur from construction, DFO will issue an authorization detailing measures that must be undertaken to minimize the impact.
DFO	Offset Plan	Construction	Work on an offsetting plan with DFO to offset the loss of permanent fish habitat by permanent infrastructure left in the river.
Transport Canada	Navigable Waters Act Permit	End of Conceptual Design	The SSR is a "scheduled" waterway under the Navigable Waters Act. Construction of the new intake will require a review and potential design modifications to permanent in-river works before permit issuance from Transport Canada.

Notes:

1) If the preliminary test drilling and observation well(s) work is successful and the Town decides to proceed with the subsequent development of an induced surface water infiltration well field, the test/supply wells could be constructed, and pump tested under the Permit to Conduct Ground Water Investigation.
2) If water well(s) development within the TVA is pursued, a separate approval may be required from the MOE and/or from the WSA to allow groundwater from the TVA interceptor wells to be discharged into the SSR. Under natural (non-pumping conditions), likely, groundwater discharge from the TVA into the SSR is already occurring, so the net effect of groundwater discharge from the interceptor wells on the SSR should be negligible.
3) The Fish/Fish Habitat Study would be used in the request for authorization to DFO.

The following table indicates which permits/approvals apply to which alternative.

Table 5-2
Applicable Permits and Approvals

Approval/Permit	Broderick Reservoir Intake	B2 Canal	Induced Surface Water Infiltration Wells	DRI
Aquatic Habitat Protection Permit (AHPP)	X	X	X	X
Water Rights License (WRL)	X	X	X	X
Approval to Construct and Operate Works (ATC/ATO)	X	X	X	X
Permit to Conduct Ground Water Investigation			X	
Approval to Discharge Groundwater[1]			X	
Fish/Fish Habitat Study	X	X		X
DFO Review	X	X		X
DFO Authorization	X	X		X
Offset Plan	X	X		X
Navigable Waters Act Permit				X

Notes:
1) Applicable only to TVA induction wells option where shield wells would need ot discharge groundwater from the aquifer into the SSR.

6 OPINION OF PROBABLE COST

A Class 'D' Opinion of Probable Cost of the capital costs for the construction of each of the studied alternatives is summarized in the table below. These prices include a **30% Contingency** for unknowns and a **15% Engineering Contingency** to account for the next stages of engineering design, geotechnical investigations, and other approvals. A Class 'D' Opinion of Probable Cost should be considered **accurate to within a range of -20% to + 30%.** In addition to the Capital Cost estimates, the costs associated with each of the three entities relying on the raw water source (Town, OWRPA, Riverview) for each alternative are broken out in Table 6-1.

Table 6-1
Opinion of Probable Cost[1]

Alternative	Town of Outlook	OWRPA	Riverview Golf Club	Combined
Broderick Reservoir Intake	$21M – $34M	$2M – $3M	$3M – $4M	$26M – $42M
B2 Canal	$26M – $58M	$1M – $2M	$2M – $3M	$26M – $42M
Groundwater Wells (not viable)	$29M – $47M	$2M – $4M	$1M – $2M	$34M – $56M
Induced Surface Water Wells	$5M – $8M	$700k – $1M	$1.0M – $12M	$7M – $11M[2][3]
Direct River Intake	$8M – $13M	$100k – $200k	$100k – $200k	$8.0M – $13M

Note:

1. Associated Engineering has no control over the actual cost of labour and materials, the bidders method of determining prices, or market conditions. This opinion of probable cost of construction is made based on experience and best judgment based on the scope of work proposed in the design. Associated Engineering cannot and does not guarantee that proposals, bids, or actual construction costs will not vary from this or subsequent cost estimates. Associated Engineering has prepared this estimate in accordance with generally accepted principles and practices in the engineering profession. Associated Engineering staff are available to discuss its contents with any interested party.
2. The capital cost estimate for the Induced Surface Water Wells assumes that wells could successfully be installed along the golf course. This cannot be confirmed as the final location until the completion of preliminary drilling and pump testing programs
3. The capital cost estimate for the Induced Surface Water Wells is based on an assumed per well production flow rate of 750 m^3/day.

Detailed breakdowns of each estimate are attached in Appendix F, Opinion of Probable Cost.

6.1 OWRPA and Riverview Golf Club Considerations

Due to the significant raw water demands associated with OWRPA (15%) and Riverview Golf Club (19%), the Town should consider the significant capital costs added to the raw water supply upgrades to accommodate these users.

For all options, the opportunity may exist to continue to use the existing intake and pump station and/or to construct and operate separate induced surface water infiltration wells or well field(s) to service OWRPA and Riverview. The Town could consider reducing capital cost requirements for the Induced Surface Water Wells option by only switching the Town of Outlook's raw water supply to induced surface water infiltration wells with a new pump station and RWM directly to the Town's WTP.

The induced surface water wells option also provides the Town with the opportunity to phase in future wells to service these customers (OWRPA and Riverview).

The Town may want to consider adding a flow metre to this forcemain in advance of the 2022 golf season to better assess the actual raw water demands of the golf course.

7 ALTERNATIVE COMPARISON

Of the four alternatives, the Broderick Reservoir Intake, B2 Canal, and Groundwater options were not included in the comparisons below. These alternatives were discredited due to the substantially greater opinion of probable costs when compared to the Induced Surface Water Infiltration Wells or new Direct River Intake options.

The key advantages and disadvantages of the Induced Surface Water Infiltration Wells and new Direct River Intake options are summarized in the table below.

Table 7-1
Alternative Comparison

Alternative	Pros	Cons
Induced Surface Water Infiltration Wells	• Simple construction/install relative to the other options. • Saturated alluvial sands directly underlying the SSR will allow the ability to maintain raw water supply during dryer summers and freezing conditionswhen SSR levels may be lower. • The produced water will be free of entrained sediment and other suspended materials normally present in the SSR. • Since multiple water wells would be required, the raw water system would be intrinsically redundant. • Depending on the final location, the wells could be located outside of the landslide zone (three areas identified in section 4.3 of this report), thus decreasing the risk of damage to the wells and lengthy/expensive emergency raw water supply interruptions • Limiting slope instability risk to the RWM from the wells to the intake pump station coupled with potential slope/movement monitoring would allow the Town to proactively repair problem stretches of the pipe before a break occurs. • Surface water source so WTP treatment equipment should require minimal alterations if any. • Subject to the aquifer extent (to be confirmed by test drilling), additional	• The reported alluvial sand depth of 25 metres below the SSR indicates good potential for the successful construction of induced surface water infiltration wells. However, requires preliminary drilling and pump testing to determine the feasibility of the proposed locations. • While monitoring movement in the slope and RWM may identify problem areas in the pipe in advance of failure, the slope instability may still cause ongoing maintenance/repair issues on the raw watermain pipe from the wells to the raw water intake pump station. The frequency of potential repairs can not be adequately predicted. • Access to the wells for regular maintenance may be challenging for some of the locations and would need to be considered during detailed design. • Regular maintenance is required on the wells at an average cost of approximately $6,000 per well annually. The 20-year design horizon estimates 13 wells will be needed (750 m^3/day/well) meaning a potential for annual maintenance costs of around $78k. • Additional land purchase may be required for the well field, depending on the preferred location and results of the preliminary drilling/pump testing investigations.

Alternative	Pros	Cons
	wells could be added on over the next twenty years as the Town and OWRPA demands grow. • Regulatory/permitting requirements are simpler and more standard than some of the specialty permits/approvals required by some of the other alternatives.	• Requires electricals servicing to the well field.
Direct River Intake	• Similar O&M procedures and annual costs to the existing raw water supply system. • Maintains raw water supply infrastructure in a localized location with established site access. • Limits the need to purchase additional land. • Least impact to capital infrastructure cost to facilitate the large raw water demands from Riverview Golf Club. • Surface water source so WTP treatment equipment should require no alterations. • Eliminates sedimentation and turbidity issues experienced at the WTP due to the direct river intake.	• Significant impact to the environment due to cofferdam required for construction. • Significant regulatory/permitting requirements with longer processing times than some of the other options. • New infrastructure may be at risk of damage from the ongoing landslide. • The abandoned Skytrail (rail bridge) is at risk of failure due to slope movement of the eastern river valley slope. A failure is expected to be catastrophic and would likely damage the new intake fish screen structure. Repairs would result in the intake needing to be taken offline and potentially a cofferdam to complete repairs. This risk may be mitigated by relocating the intake to the location discussed in Section 4.4.3 and shown on Figure C-704 in Appendix D. • Should a project to decommission the abandoned Skytrail bridge come to light in the future, the resulting construction may require several cofferdams which may cut off the intake from water for the duration of construction. • Limited scour holes are present on this stretch of the SSR. There is potential for another scour hole along the east edge of the spur that runs into the SSR from the west bank, but this would need to be investigated in greater detail with a more detailed bathymetric survey. • The NHC study completed in 2019 is considered preliminary and the

Alternative	Pros	Cons
		report notes that a detailed hydrological assessment should be completed to confirm the feasibility of this alternative. • While the preliminary NHC assessment indicates sufficient depth near Pier 3 with a 1:100 year low-flow river level, future droughts may lead to lower river levels that may lead to similar issues that the Town faces at the existing river intake location. • Future expansion of the intake to meet demands beyond the 20-year design horizon would require subsequent cofferdams to be built in the river leading to large upgrade costs in the future.

The comparison matrix depicted in the table below compares the Induced Surface Water Infiltration Wells and Drirect River Intake alternatives against one another by assigning weighted rating criteria for key considerations with a maximum Total Score of 30.0. The various considerations like capital cost have been assigned a maximum score based on their perceived impacts on the overall project. Each alternative is then scored on each rating criteria with a higher score indicating less concern and a worse score indicating greater concern. The Total Score for each alternative indicates the rank of each alternative with the lowest score indicating the least preferred alternative and the highest score indicating the most preferred alternative.

Class D capital cost estimates were prepared for all of the options to assist in the initial evaluation. These estimates are summarized in Section 6 above and detailed estimates are attached in Appendix F, Opinion of Probable Cost. Costs are compared, in a manner where the least cost option was assigned a full score value of five (5) and higher costs were assigned a portion of 5 points relative to the ratio between the higher cost alternative with the lowest cost alternative.

Table 7-2
Evaluation Matrix

Criteria	Rating Criteria	Induced Surface Water Infiltration Wells ($7M – $11M)	New Direct River Intake ($8M – $13M)
Capital Cost (Class 'D')	5.0	5.0	4.2
Annual O&M Cost (Class 'D')	5.0	1.0	5.0
Regulatory / Permitting / Approvals	5.0	5.0	3.0
Slope Instability Risk	5.0	4.0	3.0
Constructability Risk	4.0	4.0	2.0
Reliability Risk	3.0	3.0	2.0
Future Expandability	2.0	2.0	1.0
Impacts to Existing Treatment System	1.0	1.0	0.0
Total Score	30.0	25	20.2

8 RECOMMENDATIONS

Based on the findings and anlysis presented in this Raw Water Supply Study, the Induced Surface Water Infiltration Wells within alluvial sands along the SSR appears to be the most favourable options for raw water supply to the Town.

The following key considerations are summarized from the earlier text and make this the most favourable option assessed:

- Flexibility to phase in wells to reduce up-front capital costs.
- Lowest opinion of probable costs.
- Avoids in-river work and associated regulatory risks.
- Avoids constructability risks around the deep sand layers in the SSR valley and the anticipated water infiltration/seepage impacts on cofferdam construction.
- Wells are anticipated to be safe from slope instability in the flood plain, potentially limiting slope instability risks to the RWM only.
- There is a potential to pump raw water from the wellfield directly to the WTP which should be explored further in preliminary design.
    - The potential also exists to move the OWRPA wells to the west bank of the SSR (Site 'B' identified on Figure C-703 in Appendix D, Figures).
    - Refer to Ficure C-705 in Appendix D, Figures for conceptual sketches of potential RWM alignments to convey raw water from the potential well field(s) to the Town's WTP and OWRPA WTP.
- Induction wells will improve water quality at the WTP by eliminating the potential for sediments to migrate from the SSR to the WTP and may also reduce water quality risks associated with storm outfalls downstream of the existing direct river intake.

We recommend that the Town proceed with preliminary hydrogeological test drilling at both Site 'A' and 'B' identified on figure C-703 in Appendix D, Figures as the first step in Preliminary Design of this option.

We also recommend the Town install flow monitoring equipment on the Riverview RWM in advance of the 2022 golf season to better assess the golf club's raw water demands and to ensure adequate raw water supply infrastructure is incorporated into the final design.

9 NEXT STEPS

We recommend that the take the following 'Next Steps' to progress the raw water supply project.

- Intall a flowmeter on the Riverview Golf Club RWM in advance of the 2022 golf season to track and assess raw water demands for the golf course.
- Undertake the hydrogeological test drilling at Sites 'A' and 'B' identified on figure C-703 in Appendix D, Figures.
- Following successful completion of the hydrogeological test drilling and analysis, proceed with completion of the Preliminary Design of a new alluvial sands Induction Well field(s) including:
  - A geotechnical investigation of the preferred site(s);
  - Assessment of the existing intake pump station to determine remaining life span;
  - Determine the raw water conveyance strategy from the raw water well field(s) to the WTP and other raw-water users.

CLOSURE

This report was prepared for the Town of Outlook to assess several alternatives for the supply of raw water to the Town. Based on the desktop study and findings presented herein, we recommend the Town proceeds with the preliminary design of a new raw water supply to the Town's WTP and other raw water users by means of a new Induced Surface Water Infiltration Wellfield(s).

The services provided by Associated Engineering (Sask.) Ltd. in the preparation of this report was conducted in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practicing under similar conditions. No other warranty expressed or implied is made.

Respectfully submitted,
Associated Engineering (Sask.) Ltd.

Prepared by:

Rahim Ahmad, P.Eng.
Project Manager

PROFESSIONAL ENGINEER
A.R. AHMAD
MEMBER 29471
2021-12-08
YR. MN. DAY
SASKATCHEWAN

Reviewed by:

Scott Miller, P.Eng.
Division Manager - Water

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Discipline	Sask. Reg. No.	Signature
Civil	12123	

ASSOCIATED ENGINEERING
QUALITY MANAGEMENT SIGN-OFF

Signature:

Date: December 8, 2021

Respectfully submitted,

Prepared by:

Mike Famulak, P.Geo.
Project Hydrogeologist
Beckie Hydrogeologists (1990) Ltd.

PROFESSIONAL GEOSCIENTIST
M. S. FAMULAK
MEMBER 9716
2021 12 03
YR. MN. DAY
SASKATCHEWAN

Association of Professional Engineers and Geoscientists of Saskatchewan
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Discipline	Sask.Reg.No.	Signature
Hydrogeological	9716	M.S. Famulak

APPENDIX A – RECORD DRAWINGS

TOWN OF OUTLOOK

CONTRACT No. 1

RIVER INTAKE,

PUMPHOUSE MODIFICATIONS

&

RAW WATER SUPPLY MAIN

Agriculture Canada

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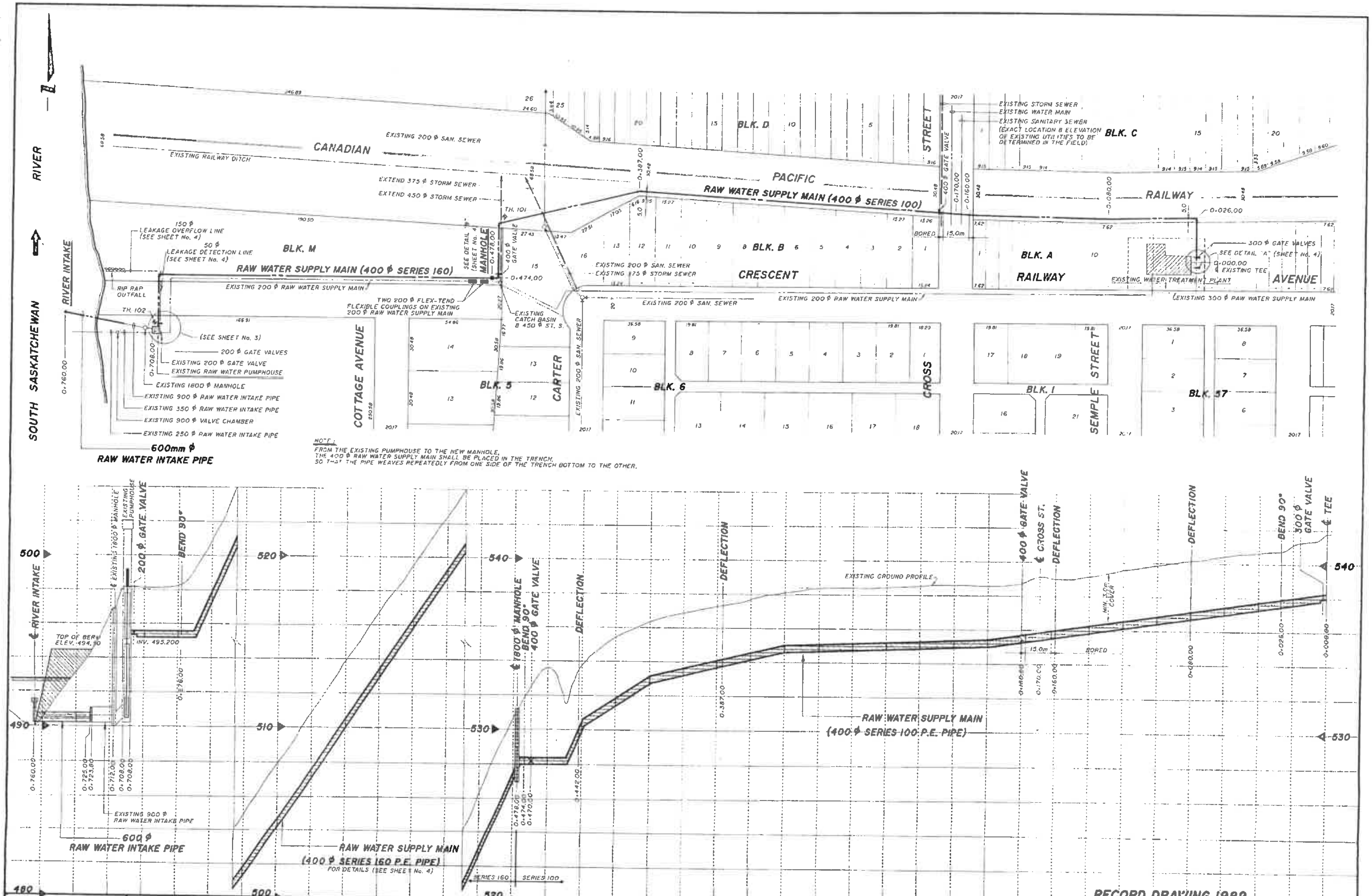
OUTLOOK

PFRA No. 114153



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TOWN OF OUTLOOK
CONTRACT No. 1
RIVER INTAKE, PUMPHOUSE MODIFICATIONS AND RAW WATER SUPPLY MAIN
RAW WATER SUPPLY MAIN

SCALE: 1:1000H 1:100V
DATE: OCT 1988
JOB: 4704
DRAWING NO.
PFRA No. 114154
SHEET 1 OF 6

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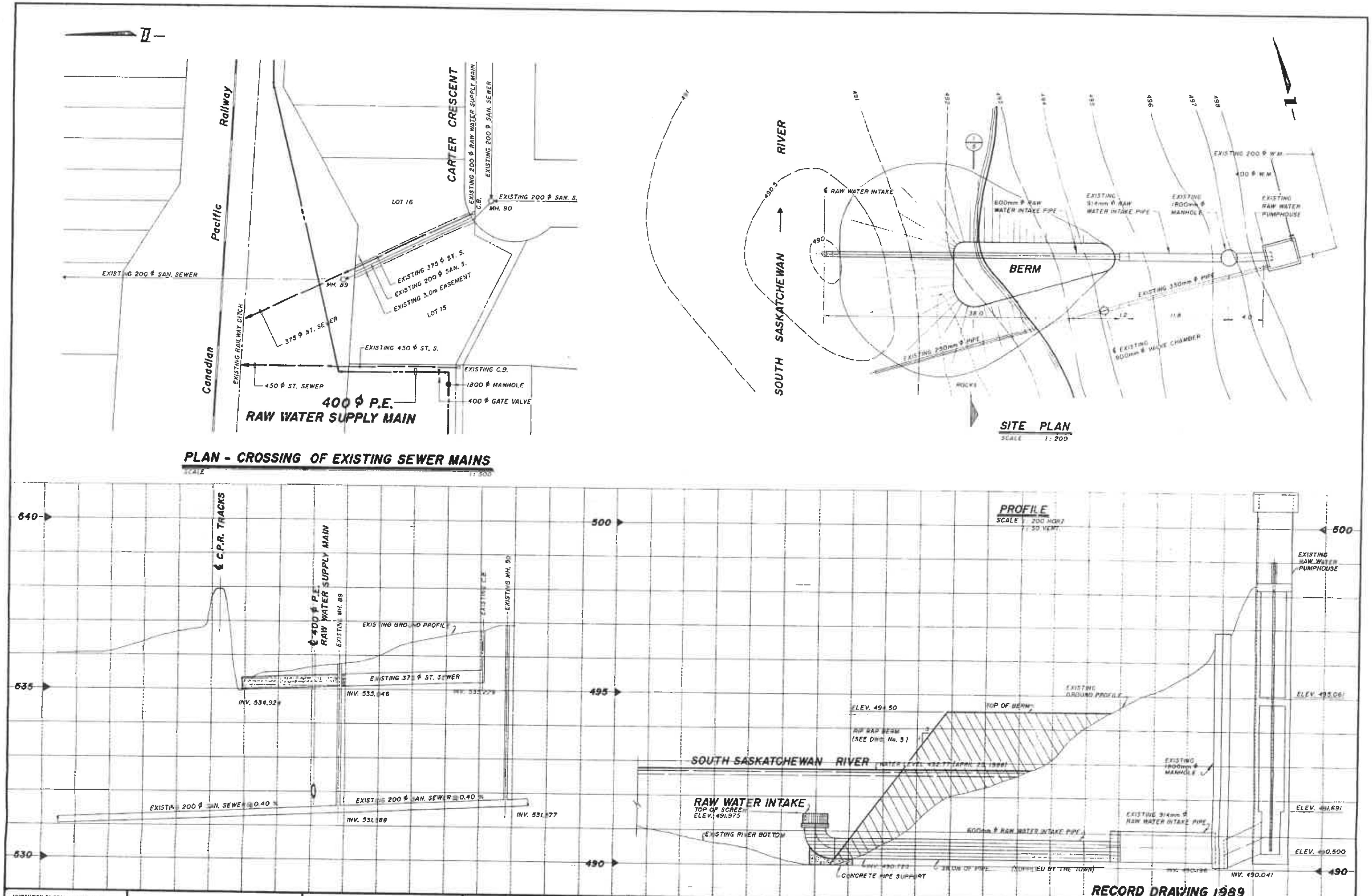

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RIVER INTAKE, PUMPHOUSE MODIFICATIONS AND RAW WATER SUPPLY MAIN
RAW WATER INTAKE

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PFRA No. 114155
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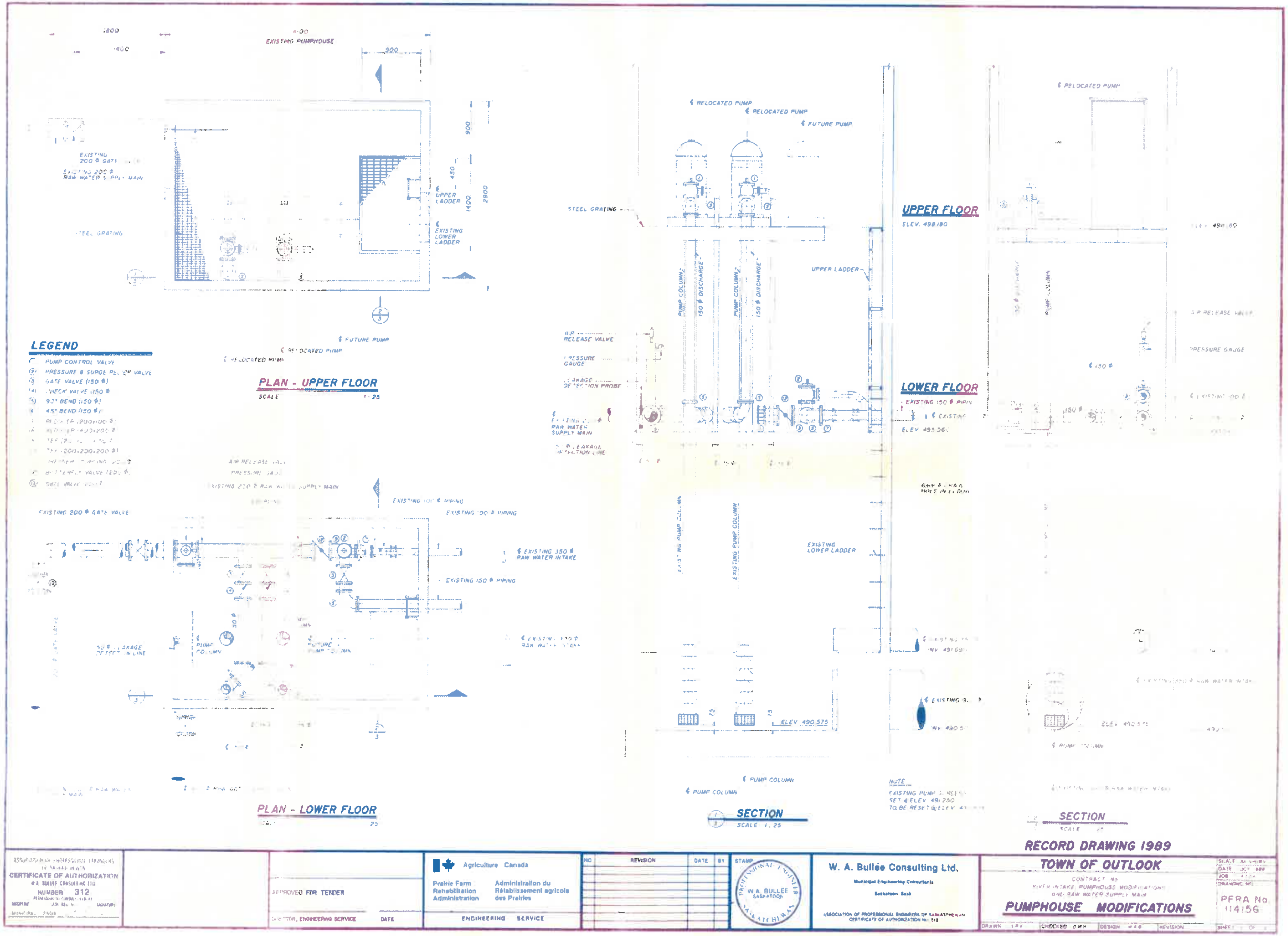
EXISTING PUMPHOUSE
LEGEND
PUMP CONTROL VALVE
PRESSURE & SURGE RELIEF VALVE
GATE VALVE (150 ϕ)
CHECK VALVE (150 ϕ
90° BEND (150 ϕ)
45° BEND (150 ϕ)
PLAN - UPPER FLOOR
PLAN - LOWER FLOOR
UPPER LADDER
EXISTING LOWER LADDER
STEEL GRATING
EXISTING 200 ϕ GATE VALVE
EXISTING 350 ϕ RAW WATER INTAKE
EXISTING 150 ϕ PIPING
℄ RELOCATED PUMP
℄ FUTURE PUMP
PUMP COLUMN
150 ϕ DISCHARGE
UPPER FLOOR
ELEV. 498.180
LOWER FLOOR
ELEV. 493.060
AIR RELEASE VALVE
PRESSURE GAUGE
EXISTING LOWER LADDER
EXISTING PUMP COLUMN
ELEV. 490.575
SECTION
SCALE 1:25
NOTE
RECORD DRAWING 1989
TOWN OF OUTLOOK
PUMPHOUSE MODIFICATIONS
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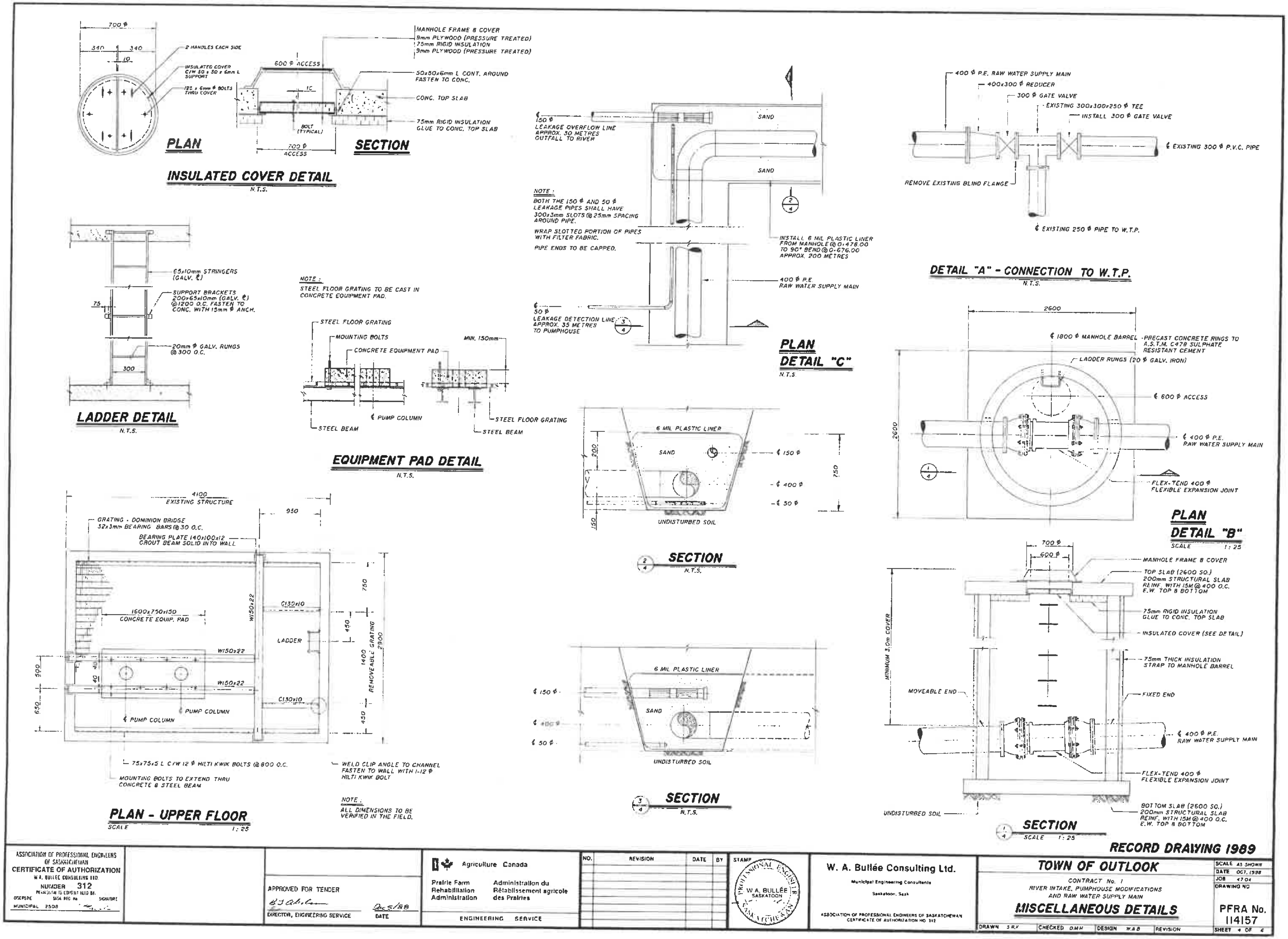
INSULATED COVER DETAIL
N.T.S.
PLAN
SECTION
700 ⌀
340
340
10
2 HANDLES EACH SIDE
INSULATED COVER C/W 50 x 50 x 6mm L SUPPORT
12⌀ x 6mm ⌀ BOLTS THRU COVER
600 ⌀ ACCESS
700 ⌀ ACCESS
BOLT (TYPICAL)
MANHOLE FRAME & COVER
9mm PLYWOOD (PRESSURE TREATED)
75mm RIGID INSULATION
9mm PLYWOOD (PRESSURE TREATED)
50x50x6mm L CONT. AROUND FASTEN TO CONC.
CONC. TOP SLAB
75mm RIGID INSULATION GLUE TO CONC. TOP SLAB
LADDER DETAIL
N.T.S.
65x10mm STRINGERS (GALV. ℄)
SUPPORT BRACKETS 200x65x10mm (GALV. ℄) @1200 O.C. FASTEN TO CONC. WITH 15mm ⌀ ANCH.
20mm ⌀ GALV. RUNGS @ 300 O.C.
75
300
EQUIPMENT PAD DETAIL
N.T.S.
NOTE:
STEEL FLOOR GRATING TO BE CAST IN CONCRETE EQUIPMENT PAD.
STEEL FLOOR GRATING
MOUNTING BOLTS
CONCRETE EQUIPMENT PAD
MIN. 150mm
℄ PUMP COLUMN
STEEL BEAM
STEEL FLOOR GRATING
STEEL BEAM
PLAN - UPPER FLOOR
SCALE 1:25
4100 EXISTING STRUCTURE
950
GRATING - DOMINION BRIDGE 32x3mm BEARING BARS @ 30 O.C.
BEARING PLATE 140x100x12 GROUT BEAM SOLID INTO WALL
1600x750x150 CONCRETE EQUIP. PAD
W150x22
W150x22
W150x22
C130x10
C130x10
LADDER
750
450
1400
450
2900 REMOVEABLE GRATING
900
650
40
40
℄ PUMP COLUMN
℄ PUMP COLUMN
75x75x5 L C/W 12 ⌀ HILTI KWIK BOLTS @ 800 O.C.
MOUNTING BOLTS TO EXTEND THRU CONCRETE & STEEL BEAM
WELD CLIP ANGLE TO CHANNEL FASTEN TO WALL WITH 1-12 ⌀ HILTI KWIK BOLT
NOTE:
ALL DIMENSIONS TO BE VERIFIED IN THE FIELD.
PLAN DETAIL "C"
N.T.S.
SAND
SAND
150 ⌀ LEAKAGE OVERFLOW LINE APPROX. 30 METRES OUTFALL TO RIVER
NOTE:
BOTH THE 150 ⌀ AND 50 ⌀ LEAKAGE PIPES SHALL HAVE 300x3mm SLOTS @ 25mm SPACING AROUND PIPE.
WRAP SLOTTED PORTION OF PIPES WITH FILTER FABRIC.
PIPE ENDS TO BE CAPPED.
INSTALL 6 MIL PLASTIC LINER FROM MANHOLE @ 0+478.00 TO 90° BEND @ 0+676.00 APPROX. 200 METRES
400 ⌀ P.E. RAW WATER SUPPLY MAIN
50 ⌀ LEAKAGE DETECTION LINE APPROX. 35 METRES TO PUMPHOUSE
SECTION
N.T.S.
6 MIL PLASTIC LINER
SAND
200
150
750
℄ 150 ⌀
℄ 400 ⌀
℄ 50 ⌀
UNDISTURBED SOIL
SECTION
N.T.S.
6 MIL PLASTIC LINER
SAND
℄ 150 ⌀
℄ 400 ⌀
℄ 50 ⌀
UNDISTURBED SOIL
DETAIL "A" - CONNECTION TO W.T.P.
N.T.S.
400 ⌀ P.E. RAW WATER SUPPLY MAIN
400x300 ⌀ REDUCER
300 ⌀ GATE VALVE
EXISTING 300x300x250 ⌀ TEE
INSTALL 300 ⌀ GATE VALVE
℄ EXISTING 300 ⌀ P.V.C. PIPE
REMOVE EXISTING BLIND FLANGE
℄ EXISTING 250 ⌀ PIPE TO W.T.P.
PLAN DETAIL "B"
SCALE 1:25
2600
2600
℄ 1800 ⌀ MANHOLE BARREL
PRECAST CONCRETE RINGS TO A.S.T.M. C478 SULPHATE RESISTANT CEMENT
LADDER RUNGS (20 ⌀ GALV. IRON)
℄ 600 ⌀ ACCESS
℄ 400 ⌀ P.E. RAW WATER SUPPLY MAIN
FLEX-TEND 400 ⌀ FLEXIBLE EXPANSION JOINT
SECTION
SCALE 1:25
700 ⌀
600 ⌀
MANHOLE FRAME & COVER
TOP SLAB (2600 SQ.) 200mm STRUCTURAL SLAB REINF. WITH 15M @ 400 O.C. E.W. TOP & BOTTOM
75mm RIGID INSULATION GLUE TO CONC. TOP SLAB
INSULATED COVER (SEE DETAIL)
75mm THICK INSULATION STRAP TO MANHOLE BARREL
FIXED END
MOVEABLE END
MINIMUM 3.0m COVER
℄ 400 ⌀ P.E. RAW WATER SUPPLY MAIN
FLEX-TEND 400 ⌀ FLEXIBLE EXPANSION JOINT
BOTTOM SLAB (2600 SQ.) 200mm STRUCTURAL SLAB REINF. WITH 15M @ 400 O.C. E.W. TOP & BOTTOM
UNDISTURBED SOIL
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RIVER INTAKE, PUMPHOUSE MODIFICATIONS AND RAW WATER SUPPLY MAIN
MISCELLANEOUS DETAILS
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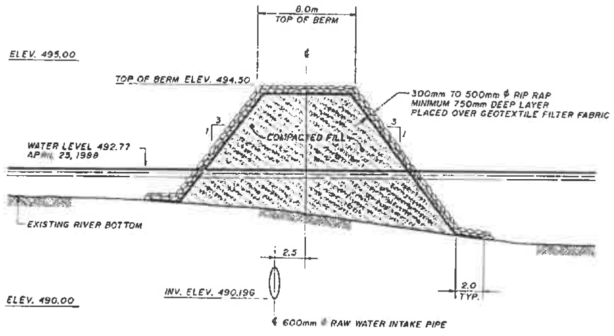

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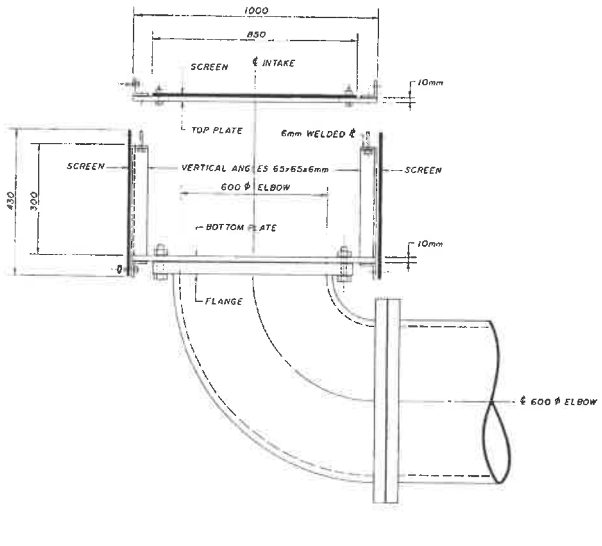

SECTION (1/5)
SCALE 1:10

TEST HOLE LOG

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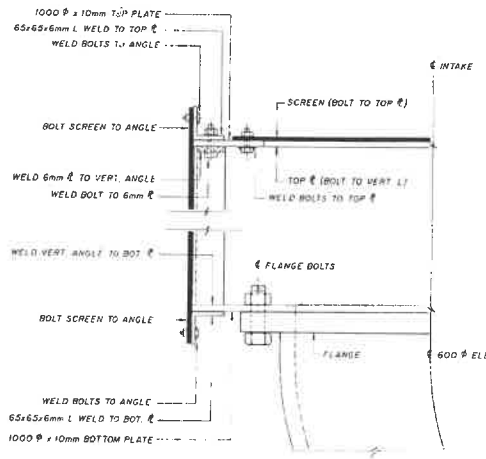
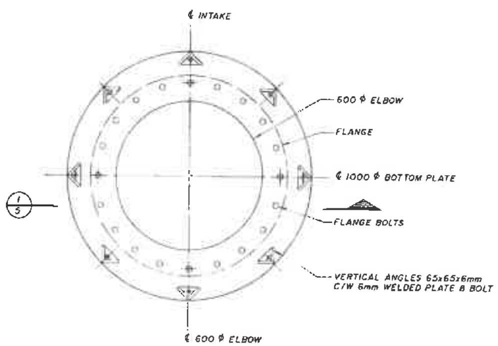
TEST HOLE LOG

Clifton Associates Ltd.





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DETAIL - SECTION (1/5)
N.T.S.

PLAN - INTAKE
SCALE 1:10

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Saskatoon, Sask.
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TOWN OF OUTLOOK
CONTRACT No. 1
RIVER INTAKE, PUMPHOUSE MODIFICATIONS AND RAW WATER SUPPLY MAIN
INTAKE & BERM DETAILS

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PFRA No. 114158
SHEET 5 OF 6

LEGEND

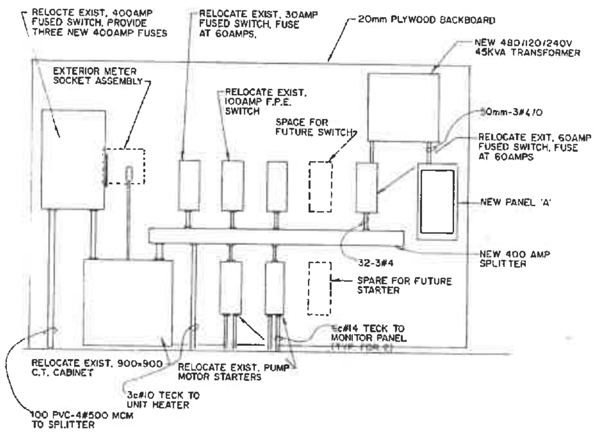
INCANDESCENT LIGHTING FIXTURE	
120V DUPLEX RECEPTACLE	
SPST SWITCH	$
MOTOR	
JUNCTION BOX	
REMAIN	○
REMOVE	●
RELOCATE	
TELEPHONE OUTLET	▲

PANEL 'A'

120/240V single phase three wire — 24 cct capacity

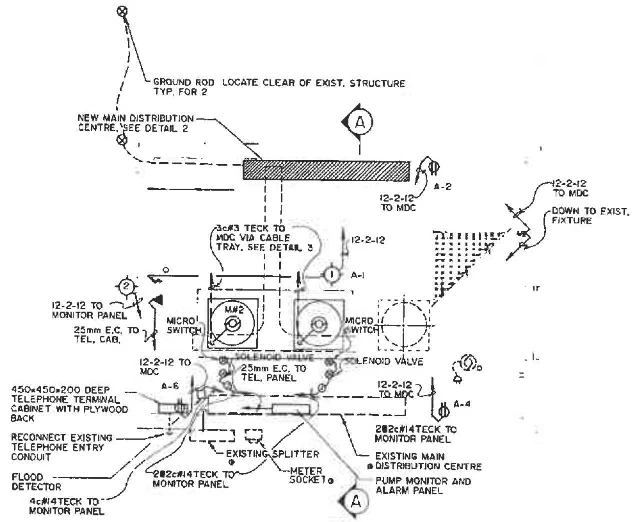
Surface mounting — 225A Mains

CCT	LOAD	BKR	CCT	LOAD	BKR
1	Lighting	1/15	2	Receptacle	1/15
3	Lighting	1/15	4	Receptacle	1/15
5	Spare	1/15	6	Telephone	1/15
7	CONTROL	1/15	8		
9			10		
11			12		
13			14		
15			16		
17			18		
19			20		
21			22		
23			24		



DETAIL 2 - MAIN DISTRIBUTION CENTRE

SCALE 1:25

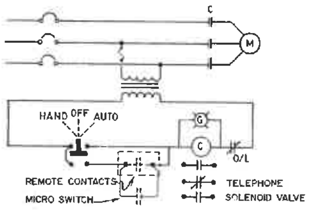


PLAN - UPPER FLOOR

SCALE 1:25

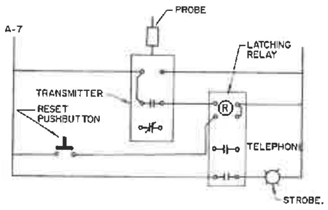
NO	DESCRIPTION	LOCATION	POWER HP-VOLT PH	CIRCUIT	MOTOR CONTROL STARTER	ACCESS	BREAKER	FEEDER	DISCONNECT	REMARKS
1	Pump #1	Pump house	60/480/3	M.D.C.	Sz 4 150MCP	a,b,b,c	3P-175A	3c#3Teck	-	Connect remote control interlock and valve
2	Pump #2	Pump house	60/480/3	M.D.C.	Sz 4 150MCP	a,b,b,c	3P-175A	3c#3Teck	-	Connect remote control interlock and valve

ACCESSORY LEGEND
a - HAND-OFF-AUTO SELECTOR
b - N.O. AUX. CONTACT
c - N.C. AUX. CONTACT



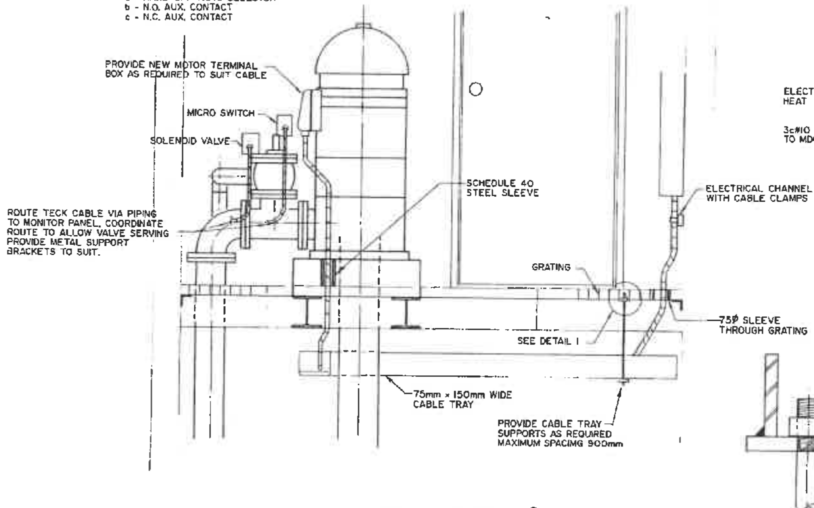
PUMP MOTOR CONTROL SCHEMATIC

N.T.S. (TYPICAL FOR 2)



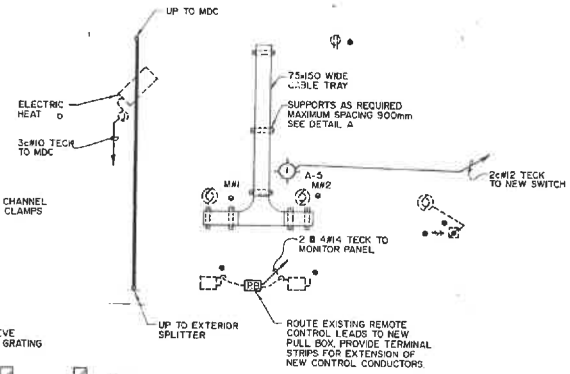
FLOOR ALARM CONTROL SCHEMATIC

N.T.S.



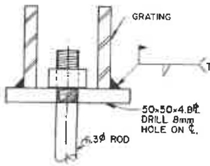
DETAIL 3 - SECTION (A)

N.T.S.



PLAN - LOWER FLOOR

SCALE 1:25



DETAIL 1

N.T.S.

RECORD DRAWING 1989

ABEL ANGUS, BUTLER ENGINEERING LTD. Mechanical & Electrical Engineers	ASSOCIATION OF PROFESSIONAL ENGINEERS OF SASKATCHEWAN CERTIFICATE OF AUTHORIZATION ANGUS, BUTLER ENGINEERING LTD. NUMBER 152 ELECTRICAL 1952	APPROVED FOR TENDER — DIRECTOR, ENGINEERING SERVICE — Dec 9/88 DATE	Agriculture Canada — Prairie Farm Rehabilitation Administration — Administration du Rétablissement agricole des Prairies — ENGINEERING SERVICE	NO. / REVISION / DATE / BY	STAMP: PROFESSIONAL ENGINEER SASKATCHEWAN
W. A. Bullée Consulting Ltd. Municipal Engineering Consultants Saskatoon, Sask. ASSOCIATION OF PROFESSIONAL ENGINEERS OF SASKATCHEWAN CERTIFICATE OF AUTHORIZATION NO. 311	TOWN OF OUTLOOK — CONTRACT No. 1 — RIVER INTAKE, PUMPHOUSE MODIFICATIONS AND RAW WATER SUPPLY MAIN — PUMPHOUSE - ELECTRICAL	DRAWN K.T. — CHECKED J.M.H. — DESIGN M.L.A. — REVISION	SCALE AS SHOWN — DATE OCT 1989 — JOB 47.04 — DRAWING NO. — PFRA No. 114159 — SHEET 6 OF 6		

APPENDIX B – PUMP STATION PHOTOS

Outlook West Regional Pipeline Association

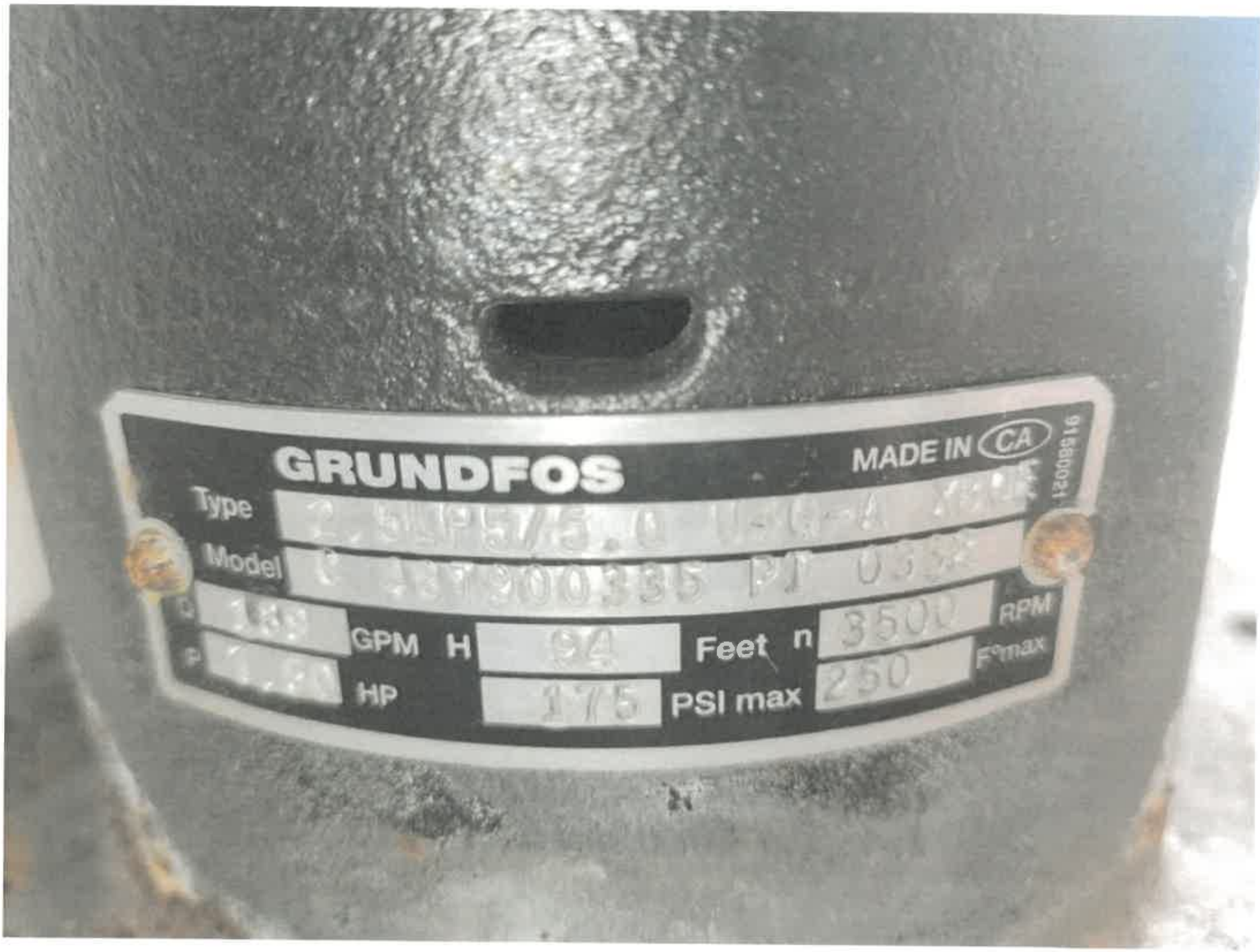


Photo 1

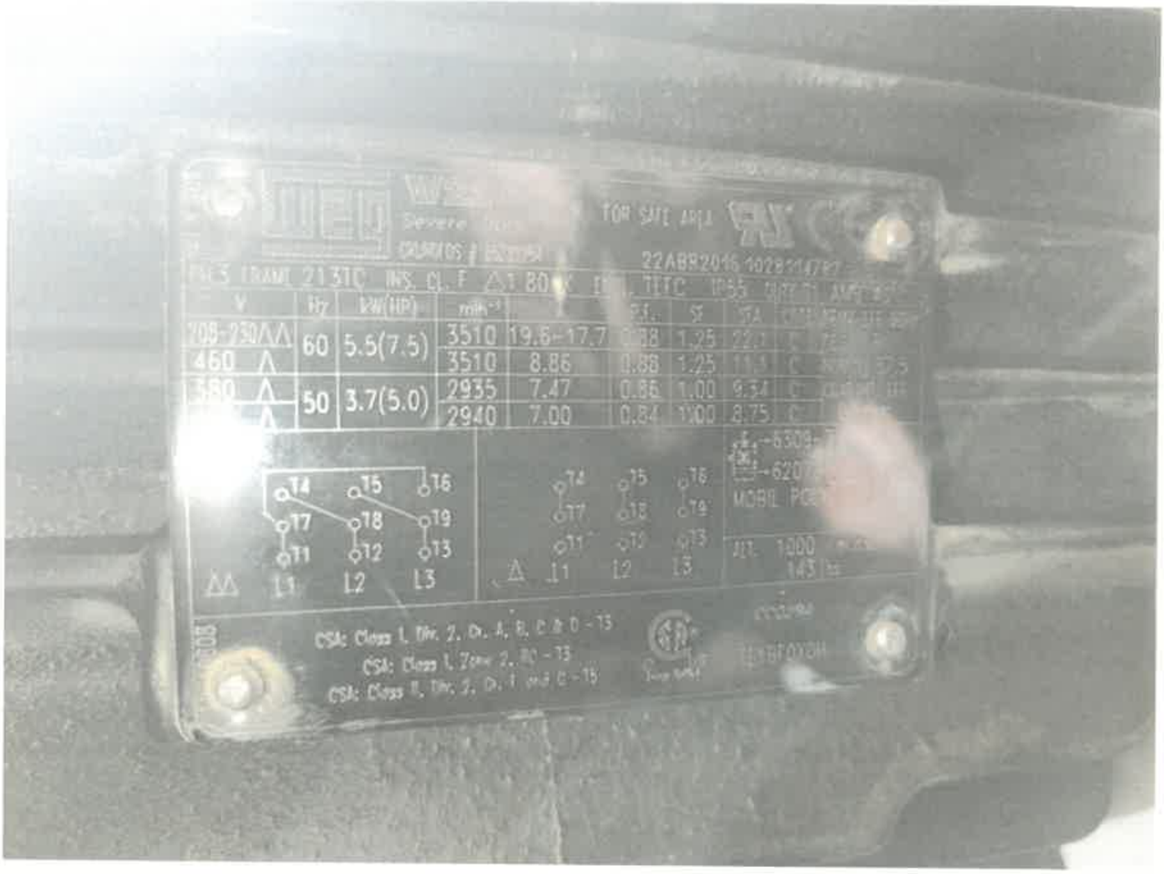

FOR SAFE AREA
22ABR2016
60
5.5(7.5)
3510
19.6-17.7
3510
8.86
50
3.7(5.0)
2935
7.47
2940
7.00
T4
T5
T6
T7
T8
T9
T1
T2
T3
L1
L2
L3
MOBIL
CSA: Class I, Div. 2, Gr. A, B, C & D - T3
CSA: Class I, Zone 2, IIC - T3

Photo 2

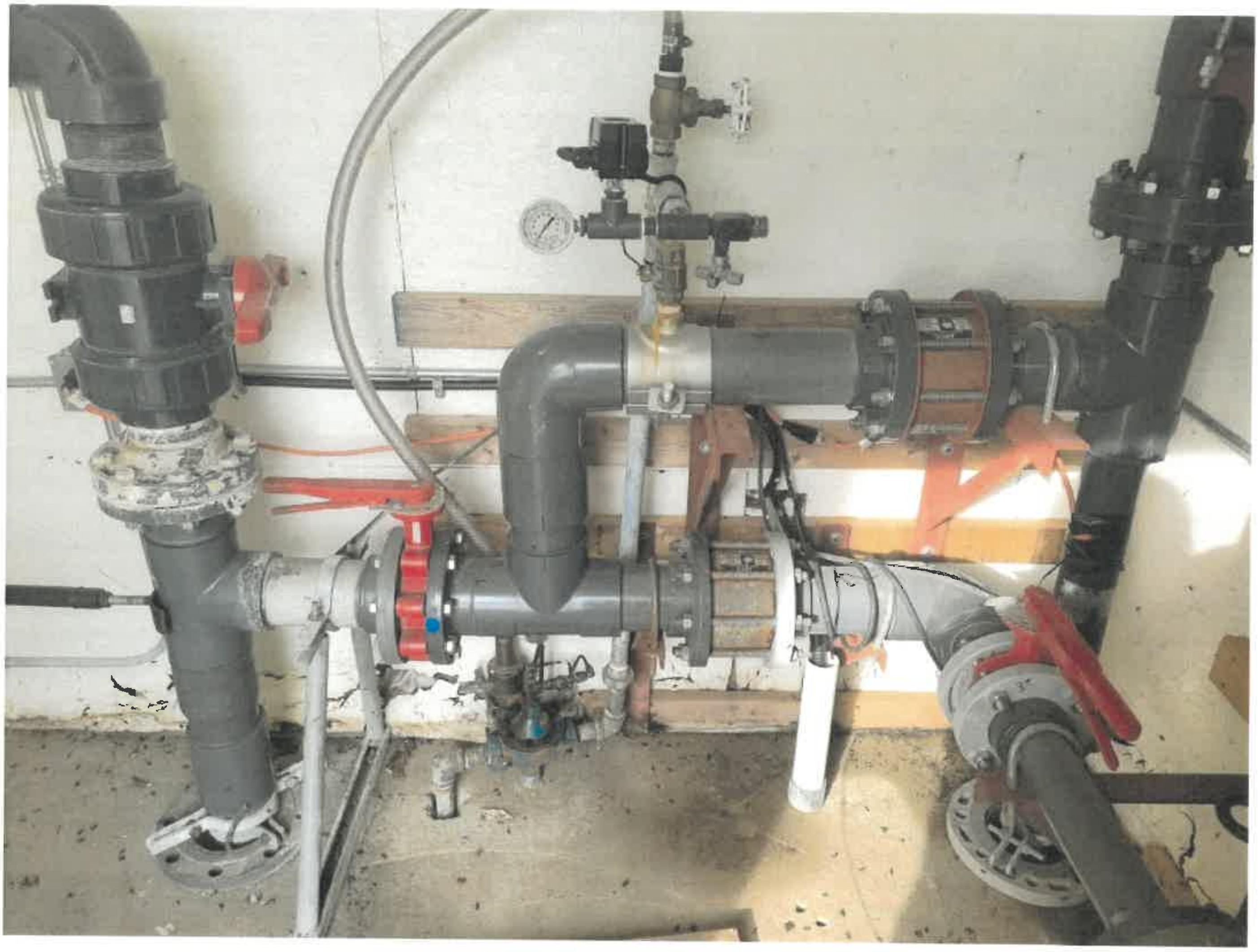


Photo 3

Riverview Golf Club

Photo 4

Photo 5

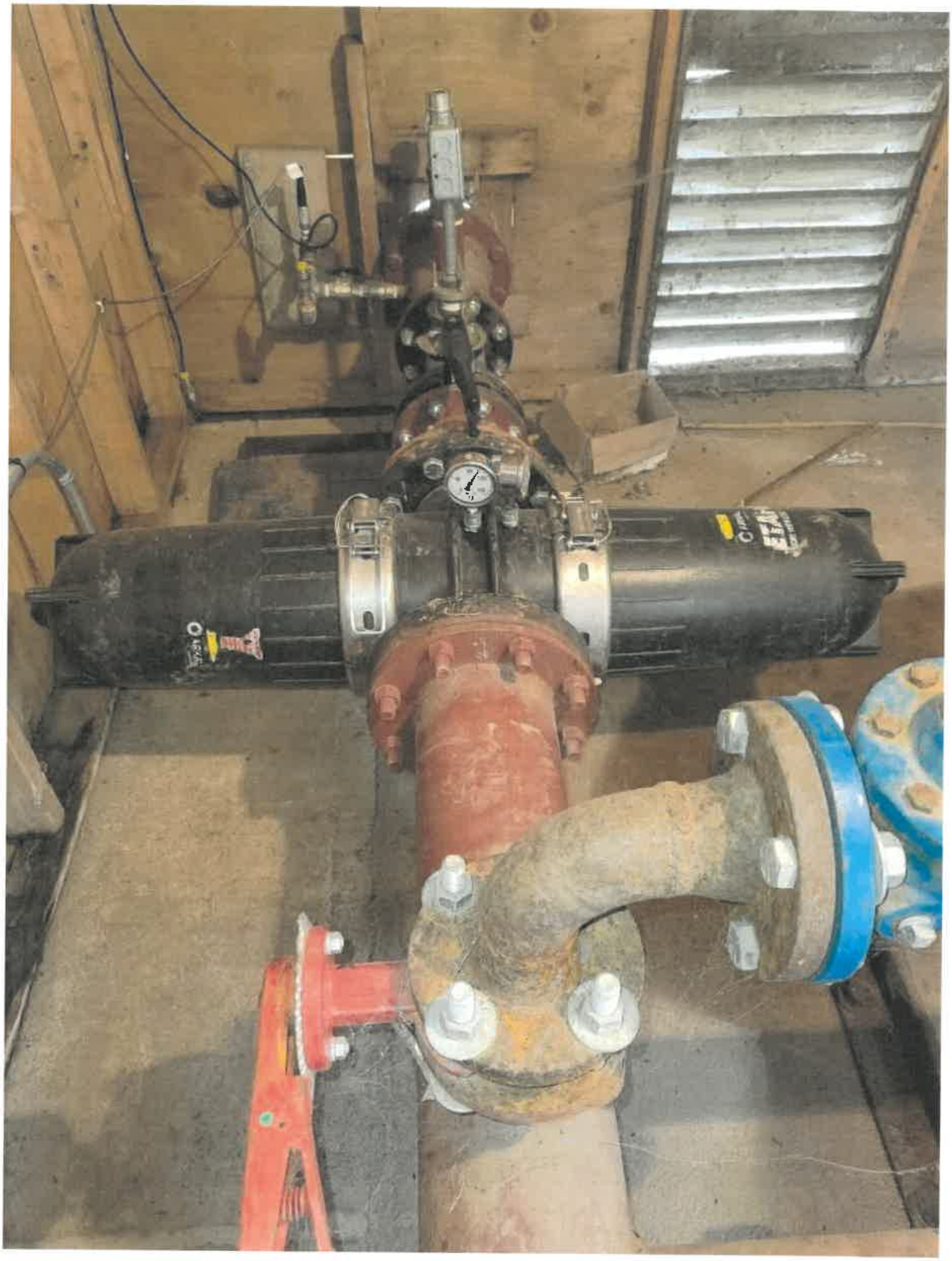


Photo 6

Photo 7

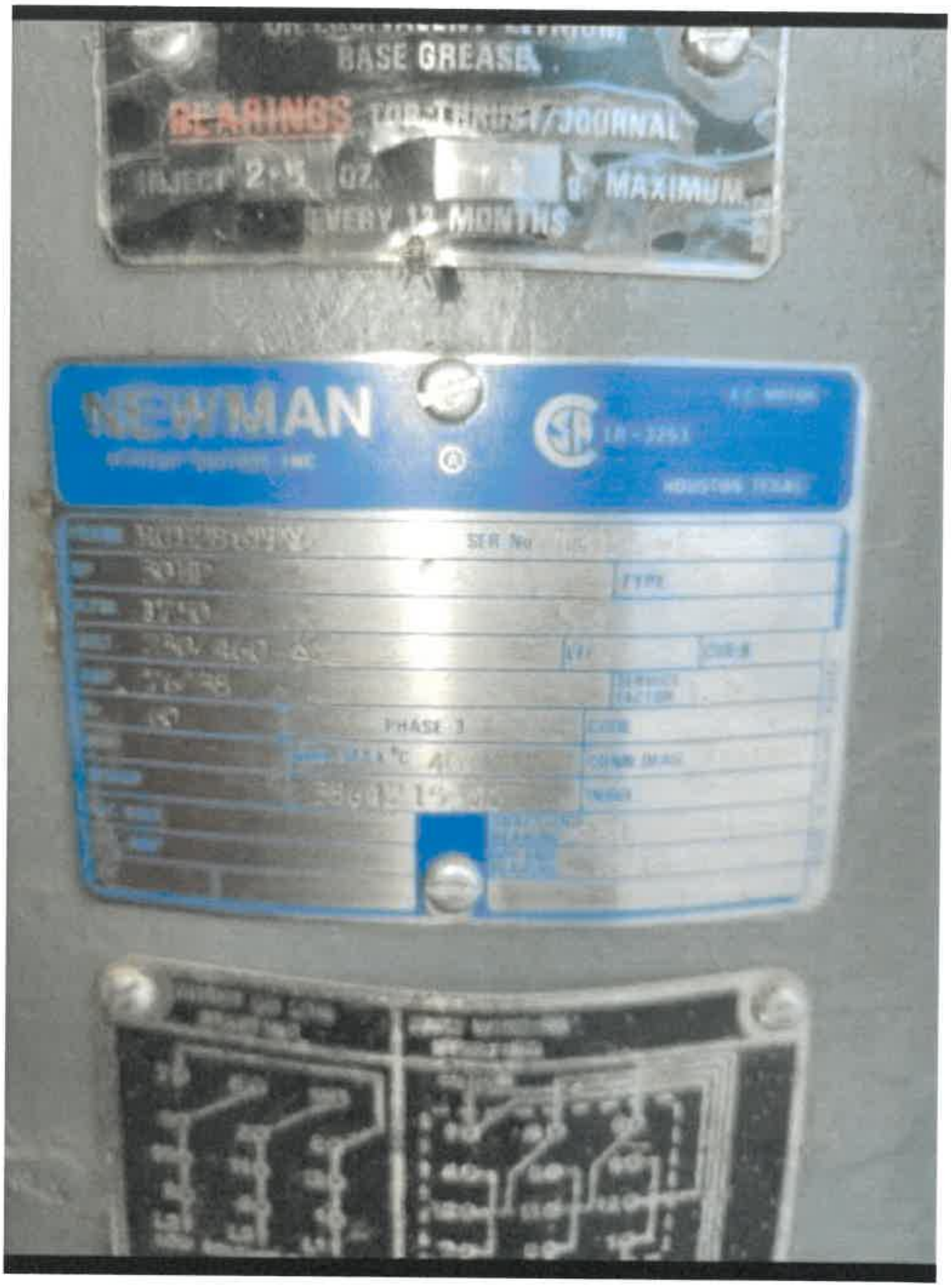
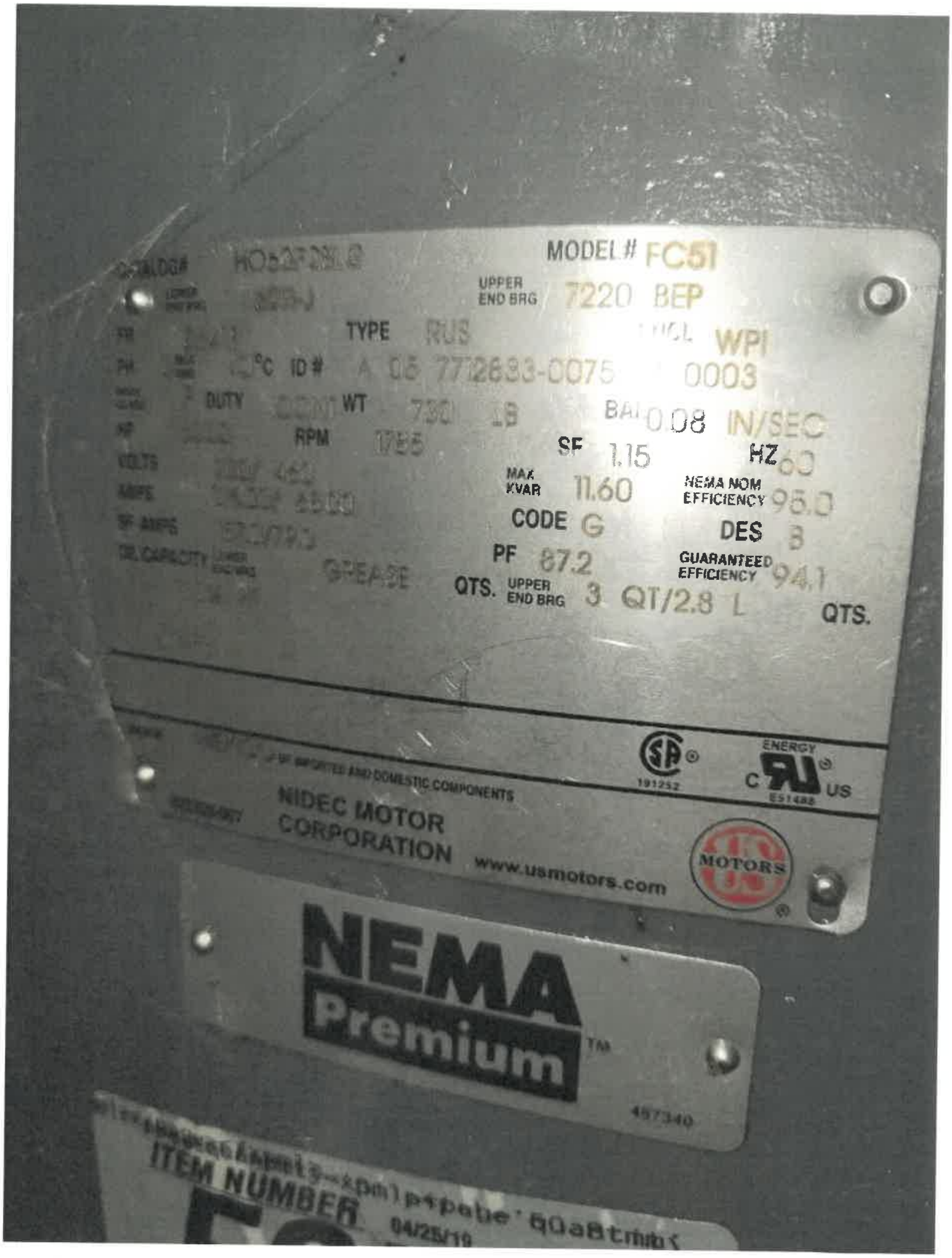

BASE GREASE
MAXIMUM
EVERY 12 MONTHS
NEWMAN
SER No
TYPE
PHASE 3

Photo 8

Town of Outlook

Photo 9



MODEL # FC51
UPPER END BRG 7220 BEP
TYPE RUS
WPI
ID # A 05 772633-0075 0003
DUTY
WT
0.08 IN/SEC
RPM
SF 1.15
HZ 60
VOLTS
AMPS
MAX KVAR 11.60
NEMA NOM EFFICIENCY 95.0
CODE G
DES B
PF 87.2
GUARANTEED EFFICIENCY 94.1
GREASE
QTS. UPPER END BRG 3 QT/2.8 L QTS.
OF IMPORTED AND DOMESTIC COMPONENTS
NIDEC MOTOR CORPORATION
www.usmotors.com
US MOTORS
NEMA Premium
ITEM NUMBER
04/25/19

Photo 10

Photo 11

Photo 12

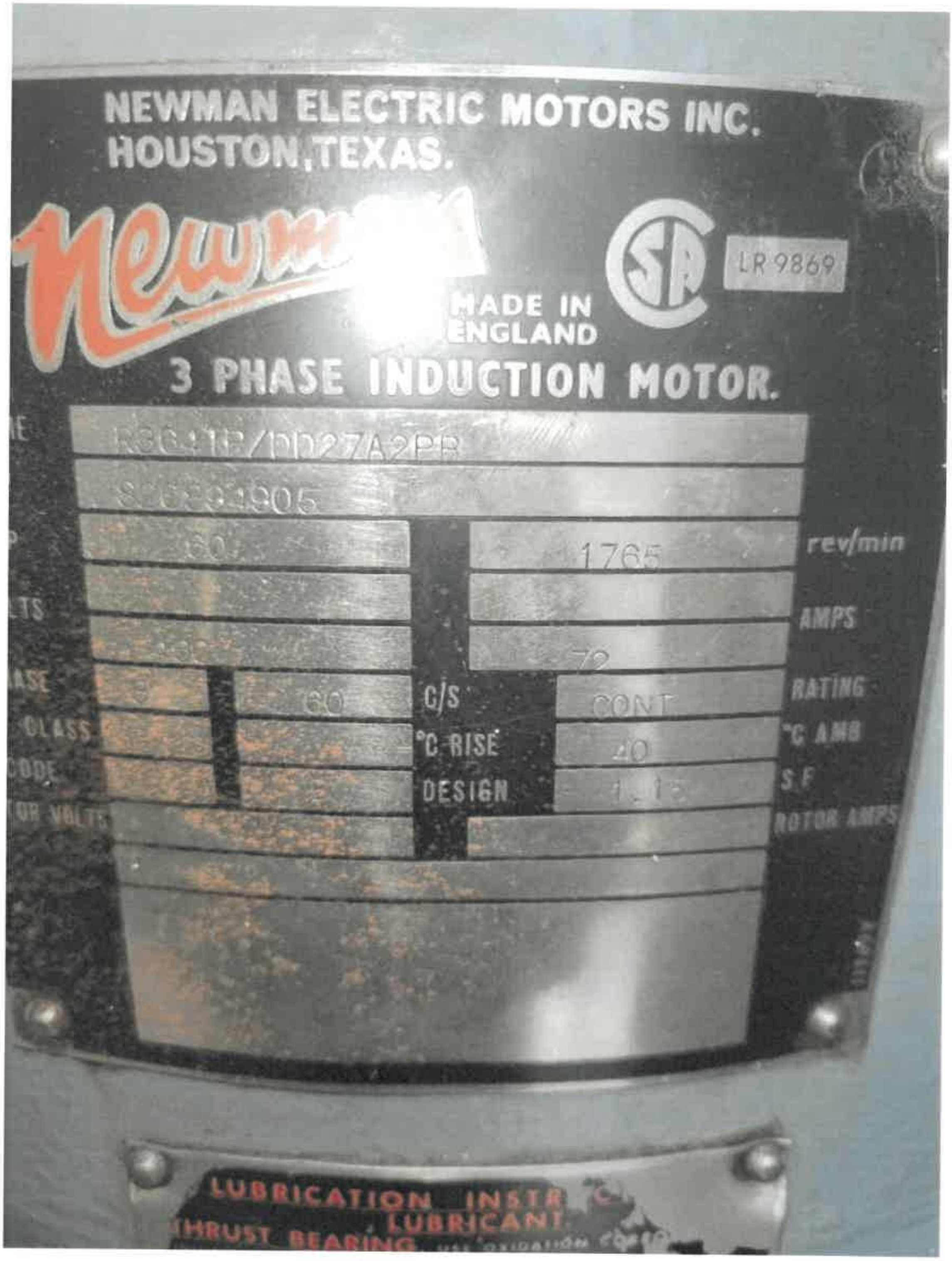

NEWMAN ELECTRIC MOTORS INC.
HOUSTON, TEXAS.
LR 9869
MADE IN ENGLAND
3 PHASE INDUCTION MOTOR.
1765
rev/min
AMPS
C/S
60
CONT
RATING
°C RISE
40
°C AMB
DESIGN
S.F
ROTOR AMPS
LUBRICATION INSTR
LUBRICANT
THRUST BEARING

Photo 13

Photo 14

APPENDIX C – WSA COMMUNITY WATER USE RECORDS REPORTS

SASKATCHEWAN

Water Security Agency

SASKATCHEWAN
COMMUNITY WATER USE RECORDS
2005 to 2019

REPORT NO. 33

Prepared and submitted by:
THE WATER SECURITY AGENCY
Regulatory Division
111 Fairford Street East
Moose Jaw, Saskatchewan S6H 7X9

September 2020

Disclaimer Notice:
The data presented is provided by the various communities listed in this report. While every effort is made to ensure that the content of this report is accurate, the report is provided "as is" and The Water Security Agency makes no representations or warranties in relation to the accuracy or completeness of the information found on it.

Nothing in this report should be taken to constitute professional advice or a formal recommendation and we exclude all representations and warranties relating to the content and use of this report.

In no event will The Water Security Agency be liable for any direct or incidental, indirect, consequential or special damages of any kind, or any damages whatsoever, including, without limitation, those resulting from loss of profit, loss of contracts, goodwill, data, information, income, anticipated savings or business relationships, whether or not advised of the possibility of such damage, arising out of or in connection with the use of this report.

INTRODUCTION

This document contains water consumption data for the majority of Saskatchewan municipalities from 2005 to 2019. The information includes Saskatchewan Ministry of Health population statistics, monthly, yearly, average day, peak day and per capita water consumption as well as peak day factors. For 2019, The Water Security Agency obtained water consumption records from 562 communities or agencies. Data for most of the First Nations was not available for 2018 or 2019 at the time of publication. This document will be updated on an annual basis.

Where community water usage records do not exist or data as obtained appears to be erroneous, the average water demands of between 230 and 450 litres/capita/day (LCD) could be used. Comments on questionable and/or missing data are included in Appendices A and B. The provincial weighted average and provincial average as shown are based on communities having water consumption in the range of 130 to 750 LCD.

It should be noted that the litres/capita/day (LCD) for some communities such as Resort Villages and First Nations may be misleading because the population may not reflect the actual users. It should also be noted that the peak day water consumption shown for each community (one-day occurrence in a year) is for information only and may be used for design purposes with caution. Sometimes this peak day water consumption data could be exaggerated due to a water main break, fire demand or other problem with the treatment or distribution system.

When communities report their peak day consumptions, occasionally questionable data is reported; this can be due to various reasons. For example, some communities may have more than one water meter. Other communities do not monitor their daily consumption. Such examples illustrate why peak day consumptions to be used in design purposes should be used with caution, as questions can be raised as to where the peak day data originates. In the case where questionable peak day data exists, the table located on the next page has been provided for reference.

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Further to this, some communities continue to report very low peak day consumptions. The reason for this low reporting has not yet been discovered. For this reason, some low peak day factors may still be found within this document but these data remain questionable.

For design purposes, water supply and treatment facilities serving municipalities with a population under 10,000, the peak day demand may be taken as two to three times the average rate. A peak hour factor of three to four would apply for distribution pumps. The table on the following page may be utilized for peaking factors for various sized communities.

The water type (R/T/U for Raw/Treated/Unknown) shown in this report next to the year for each record, indicates the type of water measured for that year. The population statistics for First Nations from 1998 onward have been provided by the Saskatchewan Ministry of Health where the statistics are available. A zero (0) does not necessarily indicate that no water was consumed, but that water records may not have been available.

For this publication, we have placed the usage records for pipeline-served utility groups after the named communities in the main report.

RECOMMENDED PEAKING FACTORS FOR MUNICIPAL WATER SUPPLY SYSTEMS		
Population Range	Peak Day Factor (For Supply and Treatment)	Peak Hour Factor (For Distribution)
Up to 500	3.00	4.50
501 - 1,000	2.75	4.13
1,001 - 2,000	2.50	3.75
2,001 - 3,000	2.25	3.38
3,001 - 10,000	2.00	3.00
10,001 - 25,000	1.90	2.85
25,001 - 50,000	1.80	2.70
50,001 - 75,000	1.75	2.62
75,001 - 150,000	1.65	2.48
Greater than 150,000	1.50	2.25

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PROVINCIAL AVERAGES Litres/Capita/Day (LCD) between 130-750			
Year	LCD[1]	Weighted LCD[2]	Communities used in Average
2019	262	341	335
2018	269	356	353
2017	276	375	340
2016	268	343	339
2015	293	367	388
2014	289	368	390
2013	284	387	411
2012	279	386	418
2011	275	383	405
2010	295	362	486
2009	328	408	523
2008	324	417	519
2007	331	438	516
2006	338	446	519
2005	320	420	543

[1]The LCD (litres/capita/day) is computed by summing the LCD for each community and dividing by the number of communities and 365 (days).
[2]The Weighted LCD (litres/capita/day) is computed by summing the yearly water consumption for each community and dividing by the total population and 365 (days).

If there are any questions or comments, please contact:

Communications Branch
The Water Security Agency
111 Fairford Street East
Moose Jaw SK S6H 7X9
Phone: (306) 694-3900
Email: comm.@wsask.ca

Water Security Agency

Community Water Consumption (Cubic Metres)

Year	Type	SH Pop	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year Total	Avg Day	Peak Day	P.D. Factor	Per Capita
Osler																			
2019	T	1889	5846	5521	5078	6074	6682	8378	6601	6733	6580	5671	5527	5583	74274	203	0	0	107
2018	T	1844	7014	6084	6671	6822	8384	10773	9501	8903	6934	6548	6225	6524	90383	247	584	2.36	134
2017	T	1895	6651	6095	6636	6397	8009	10391	13218	9527	8385	7795	7168	7351	97623	267	587	2.2	141
2016	T	1875	8210	7049	7694	7199	10153	11605	8792	7698	7597	8955	7416	7328	99696	272	0	0	145
2015	T	1840	6775	5505	5480	6168	6871	9711	8839	7495	6205	5983	5697	5650	80379	220	0	0	119
2014	T	1795	7003	5695	5683	5902	6723	6675	7315	7707	6636	5710	6090	4957	76096	208	0	0	116
2013	R	1728	7233	6044	5131	8331	7651	9802	8161	9808	10608	6860	7101	4761	91491	250	434	1.73	145
2012	T	1670	5046	4655	5150	5247	5769	5657	6142	6011	6230	5729	5610	5493	66739	182	311	1.7	109
2011	T	1697	5196	4189	5394	5301	5561	8027	5904	7662	5517	5815	5450	4541	68557	187	352	1.87	110
2010	T	1372	5520	4625	5967	4486	5383	5627	6051	6157	4604	4555	5900	3470	62345	170	358	2.1	124
2009	T	1419	5725	4935	5440	5545	7380	9415	7965	6260	7000	6035	4950	4735	75385	206	475	2.3	145
2008	T	1360	5375	5195	5215	5395	7995	8620	7655	8905	6180	6230	4690	5910	77365	211	523	2.47	155
2007	T	1303	6055	4015	5640	4830	5675	6800	8180	7215	4990	5690	5685	5655	70430	192	638	3.31	148
2006	T	1239	4550	4525	5480	5095	6140	7460	6880	7105	6195	6310	4745	5340	69825	191	690	3.61	154
2005	T	1176	4676	4190	4840	5385	6060	5790	6320	6020	5315	4830	4755	5417	63598	174	382	2.19	148
Outlook																			
2019	R	3006	16115	13684	19840	21503	29553	32767	28107	34301	20085	18768	18051	18178	270953	742	2970	4	246
2018	R	2969	17033	15028	19026	18795	31232	44320	40676	43855	21196	19735	18668	20030	309593	848	1957	2.31	285
2017	T	3038	28858	25425	29213	30648	40541	53931	59937	52735	44062	34409	31603	29891	461252	1263	3766	2.98	415
2016	T	2963	28735	27408	30741	33842	45917	49971	37274	36244	34852	30264	28693	30461	414402	1132	2637	2.33	382
2015	R	2857	27856	25070	30322	29158	45839	54407	43392	37294	33652	28782	26188	27179	409139	1120	2848	2.54	392
2014	R	2843	27286	24549	30191	29613	37299	36754	45232	45026	34451	31706	27296	29462	398865	1092	2513	2.3	384
2013	R	2838	28004 *	25081 *	28201 *	28248 *	39788 *	41153 *	46549 *	51106 *	36762 *	29519 *	26803 *	26711 *	407925	1117	2417	2.16	393
2012	R	2807	29474 *	29268 *	26782 *	28557 *	34916 *	36460 *	42801 *	42136 *	39611 *	28981 *	26105 *	28086 *	393177	1074	2418	2.24	382
2011	R	2801	30216 *	26833 *	32776 *	31945 *	39024 *	51505 *	53135 *	54963 *	40241 *	30856 *	28192 *	29474 *	449160	1230	2900	2.36	439
2010	R	2554	31270	29137	34190	34723	40125	37875	50438	45642	35034	33983	31647	32823	436887	1196	2427	2.03	468
2009	R	2476	32578	28565	34257	34541	51032	62051	51975	52310	55912	38780	32596	33628	508225	1392	2904	2.09	562
2008	R	2433	33933	30561	34922	33411	54577	54912	63992	59609	39439	34611	30081	33399	503447	1375	3129	2.27	565
2007	R	2338	31902	27631	32519	32695	44937	48071	70984	43888	40058	37700	33226	35059	478670	1311	3234	2.47	560
2006	R	2295	32698	29053	33286	35136	45885	45483	83258	62971	39840	33007	30484	31783	502884	1377	3611	2.62	600
2005	R	2348	29849	28031	33260	33570	47978	40779	63821	54798	37635	34119	30855	32052	466747	1278	3124	2.44	544
Oxbow																			
2019	R	1700	17298	15405	16326	15213	16533	9204	18820	14947	9299	9901	10962	12034	165942	454	936	2.06	267
2018	R	1634	21205	15442	16233	18137	23501	20567	23528	24226	16477	18539	18145	17499	233499	639	1187	1.86	391
2017	R	1661	17813	20130	19793	19460	23763	22538	25733	23969	24371	23849	22980	21169	265568	727	1351	1.86	438
2016	R	1668	16783	14328	15905	15896	23435	21046	20273	23686	21233	20071	16329	17751	226736	619	1168	1.89	371
2015	R	1706	19066 *	19921 *	28195 *	26154 *	24511	25925	25550	23107	17785	17015	16790	16834	260853	714	1051	1.47	418
2014	R	1715	22681	24429	28138	26655	25321	24490	25615	25158	25781	27317	25145	16046	296776	813	1773	2.18	474
2013	R	1688	17714	20629	21402	22444	25738	28484	30578	26482	23641	24800	22074	20839	284825	780	1124	1.44	462
2012	R	1610	17984	16409	17711	17114	21886	23306	26338	24952	53973	20470	17648	17000	274791	750	1152	1.53	466

Water Security Agency

Community Water Consumption (Cubic Metres)

Year	Type	SH Pop	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year Total	Avg Day	Peak Day	P.D. Factor	Per Capita
Notekue Water Inc.																			
2019	T	0	33	30	33	65	67	65	98	98	94	25	24	25	657	1	0	0	0
2018	T	0	37	33	37	90	93	90	82	82	80	32	31	32	719	1	0	0	0
2017	T	0	39	35	39	88	91	88	125	125	121	54	53	54	913	2	0	0	0
2014	T	0	38	35	38	72	75	72	96	96	93	48	47	48	758	2	0	0	0
Outlook West Regional Pipeline Association																			
2019	R	0	7373	6781	6612	7658	9563	8511	8714	8991	7838	7950	7156	7610	94757	259	0	0	0
2018	R	0	9870	8974	9263	10021	13120	12864	11599	13103	9042	9118	8943	8703	124620	341	742	2.18	0
2017	T	0	8296	8053	8883	9004	11453	12400	12360	11357	9919	9208	9639	9808	120380	329	0	0	0
2016	R	0	8764	8099	9659	9502	10575	12435	10281	9636	9054	8373	8305	8912	113595	310	751	2.42	0
2015	T	0	8265	7207	8755	7911	12739	9863	8656	8392	7628	6563	6785	7351	100115	274	717	2.61	0
2014	T	0	9434	8085	9300	8968	10885	10448	12075	11367	9280	9030	8780	9196	116848	320	731	2.28	0
2013	R	0	119973	0	0	0	0	0	0	0	0	0	0	0	119973	328	0	0	0
2012	R	0	131968	0	0	0	0	0	0	0	0	0	0	0	131968	360	0	0	0
2011	R	0	115857	0	0	0	0	0	0	0	0	0	0	0	115857	317	0	0	0
2010	R	0	114628	0	0	0	0	0	0	0	0	0	0	0	114628	314	0	0	0
2009	R	0	114516	0	0	0	0	0	0	0	0	0	0	0	114516	313	0	0	0
2008	R	0	0	0	0	0	0	0	0	0	0	0	0	95621	95621	261	0	0	0
2007	R	0	0	0	0	0	0	0	0	0	0	0	0	104545	104545	286	0	0	0
2006	R	0	0	0	0	0	0	0	0	0	0	0	0	87355	87355	239	0	0	0
2005	R	0	0	0	0	0	0	0	0	0	0	0	0	89932	89932	246	0	0	0
Parklane Waterline Inc.																			
2019	T	0	77	79	64	76	95	132	117	105	88	76	71	75	1055	2	0	0	0
2017	T	0	81	64	66	89	109	119	200	141	107	93	76	74	1219	3	0	0	0
2016	T	0	81	68	69	88	201	329	344	170	139	67	67	73	1696	4	0	0	0
2015	T	0	79	58	60	100	228	264	142	152	123	73	65	71	1415	3	0	0	0
2014	T	0	112	86	69	78	193	131	325	169	130	68	60	59	1480	4	0	0	0
2013	T	0	71	70	59	55	185	188	279	166	202	107	109	86	1577	4	0	0	0
Parkview Water Users Inc.																			
2019	T	0	373	285	300	427	643	1011	581	1464	1128	987	457	232	7888	21	0	0	0
2018	T	0	549	467	549	240	1300	1353	720	2499	1848	1677	1591	312	13105	35	0	0	0
2017	T	0	997	132	135	256	221	294	456	419	370	378	373	381	4412	12	0	0	0
2016	T	0	153	137	147	204	352	289	568	318	425	424	493	596	4106	11	0	0	0
2015	T	0	146	110	107	261	289	490	537	468	466	248	148	125	3395	9	0	0	0
2014	T	0	180	147	146	304	330	206	473	374	182	169	117	117	2745	7	0	0	0
2013	T	0	311	107	80	153	460	326	452	617	571	774	150	123	4124	11	0	0	0

ONE ARROW RESERVE #95

2005 forward: Data supplied by SaskWater. Backwash not metered through truck fill.

2005 Truck Fill 8,933 m3
2006 Truck Fill 9,174 m3
2007 Truck Fill 8,987 m3
2008 Truck Fill 8,421 m3
2009 Truck Fill 7,629 m3
2010 Truck Fill 7,387 m3
2011 Truck Fill 6,473 m3

ORMISTON

2006 Dec - Water Break
2007 Water line leak in April. RM states population = 20. High consumption due to major leaks which cannot be located due to sandy soils.
2010 Water main breaks in January and August.
2013 Line breaks fixed, new pump installed resulting in lower usage.
2016 Meter broken early in year.
2019 Water leak Jan - Apr.

OSAGE

2010 New meter in July. Flooded ice in December.

OSLER

2008 Peak day from Community of Osler water use card.

OUTLOOK

2011 forward: Pipeline use is not included.

PALMER

2011 water line break in November
2012 Data provided in cubic feet.

PAMBRUN

2010 Water leak in January.
2011 Previous report was a tankload.
2015 System leak in January.

PANGMAN

2013 Estimates based on 2012 and 2014 monthly percentages. Total yearly use is accurate.

PASQUA RESERVE #79

2005 forward: Data supplied by SaskWater. Truck fill not metered.

PAYNTON

2007 confirmed that it is raw water
2009 Cleaned reservoir & lines in November and line break in December.

PEEBLES

2008 Didn't receive card so took average using meter readings.

NORTHEAST INDIAN HEAD RURAL PIPELINE
2015 Quarterly reports.

OUTLOOK WEST REGIONAL PIPELINE ASSOCIATION
2015 forward: Excludes Milden usage.

PARKVIEW WATER USERS INC.
2018 Line breaks Sept. to Nov.

PASQUA WATER USERS COOPERATIVE LTD.
2013 Monthly totals estimated from Moose Jaw usage.

PEACEFUL SPRINGS WATER USERS INC.
2018 Meter change in May.

PRINCE ALBERT RURAL WATER UTILITY
2015 forward: Excludes water sold to Muskoday FN.

SHAMROCK WATER PIPELINE PROJECT
2005 forward: Pipeline usage only - average of 2 month meter readings.
2005 No explanation as to why lower usage
2008 forward: December values added from subsequent year's records. Usage doesn't include Shamrock

SWIFT CURRENT RURAL PIPELINE ASSOCIATION
2005 forward: All three municipal lines.

VALLEY RIDGE COMMUNITY COOPERATIVE
2013 forward: Monthly totals estimated from Moose Jaw usage.

WOOD DALE WATER CO-OP
2005 forward: Pipeline usage only, Village of Woodrow usage has been removed. Average of 2-month readings.
2010 Averaged amounts (did not send in data)
2012 Estimated data; reports not received.
2015 Bi-monthly usage provided.

YORKVILLE PUBLIC UTILITY BOARD-MELVILLE
2005 forward: Total pipeline usage; obtains treated water from Melville.
2013 Records from Melville. YPUB records show consumption at 108,672 m3.
2015 Peak day estimated.

APPENDIX D - FIGURES

IF NOT 25 mm ADJUST SCALES

SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE

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Associated Engineering

BEST MANAGED COMPANIES
Platinum member

AE PROJECT No.	20214675-00
SCALE	1:40000
APPROVED	R. AHMAD
DATE	2021DEC08
REV	A
DESCRIPTION	ISSUED FOR REVIEW

PRELIMINARY/ FOR DISCUSSION
NOT FOR CONSTRUCTION

FIGURE C-701

TOWN OF OUTLOOK
RAW WATER SUPPLY STUDY

CIVIL
BRODERICK RESERVOUR INTAKE

AE PROJECT No.	20214675-00
SCALE	1:10000
APPROVED	R. AHMAD
DATE	2021DEC08
REV	A
DESCRIPTION	ISSUED FOR REVIEW

FIGURE C-702

TOWN OF OUTLOOK
RAW WATER SUPPLY STUDY

CIVIL
B2 CANAL INTAKE TURNOUT

PRELIMINARY/
FOR DISCUSSION
NOT FOR CONSTRUCTION

Associated Engineering

BEST MANAGED COMPANIES
Platinum member

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IF NOT 25 mm ADJUST SCALES

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BEST MANAGED COMPANIES

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SCALE	1:10000
APPROVED	R. AHMAD
DATE	2021DEC08
REV	A
DESCRIPTION	ISSUED FOR REVIEW

PRELIMINARY/ FOR DISCUSSION NOT FOR CONSTRUCTION

FIGURE C-703

TOWN OF OUTLOOK
RAW WATER SUPPLY STUDY

CIVIL
INDUCED SURFACE WATER INFILTRATION WELLS

IF NOT 25 mm ADJUST SCALES

SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE

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AE PROJECT No.	20214675-00
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APPROVED	R. AHMAD
DATE	2021DEC08
REV	A
DESCRIPTION	ISSUED FOR REVIEW

PRELIMINARY/ FOR DISCUSSION NOT FOR CONSTRUCTION

FIGURE C-704

TOWN OF OUTLOOK
RAW WATER SUPPLY STUDY

CIVIL
DIRECT RIVER INTAKE

IF NOT 25 mm ADJUST SCALES

SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE

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Associated Engineering

BEST MANAGED COMPANIES
Platinum member

AE PROJECT No.	20214675-00
SCALE	1:10000
APPROVED	R. AHMAD
DATE	2021DEC08
REV	A
DESCRIPTION	ISSUED FOR REVIEW

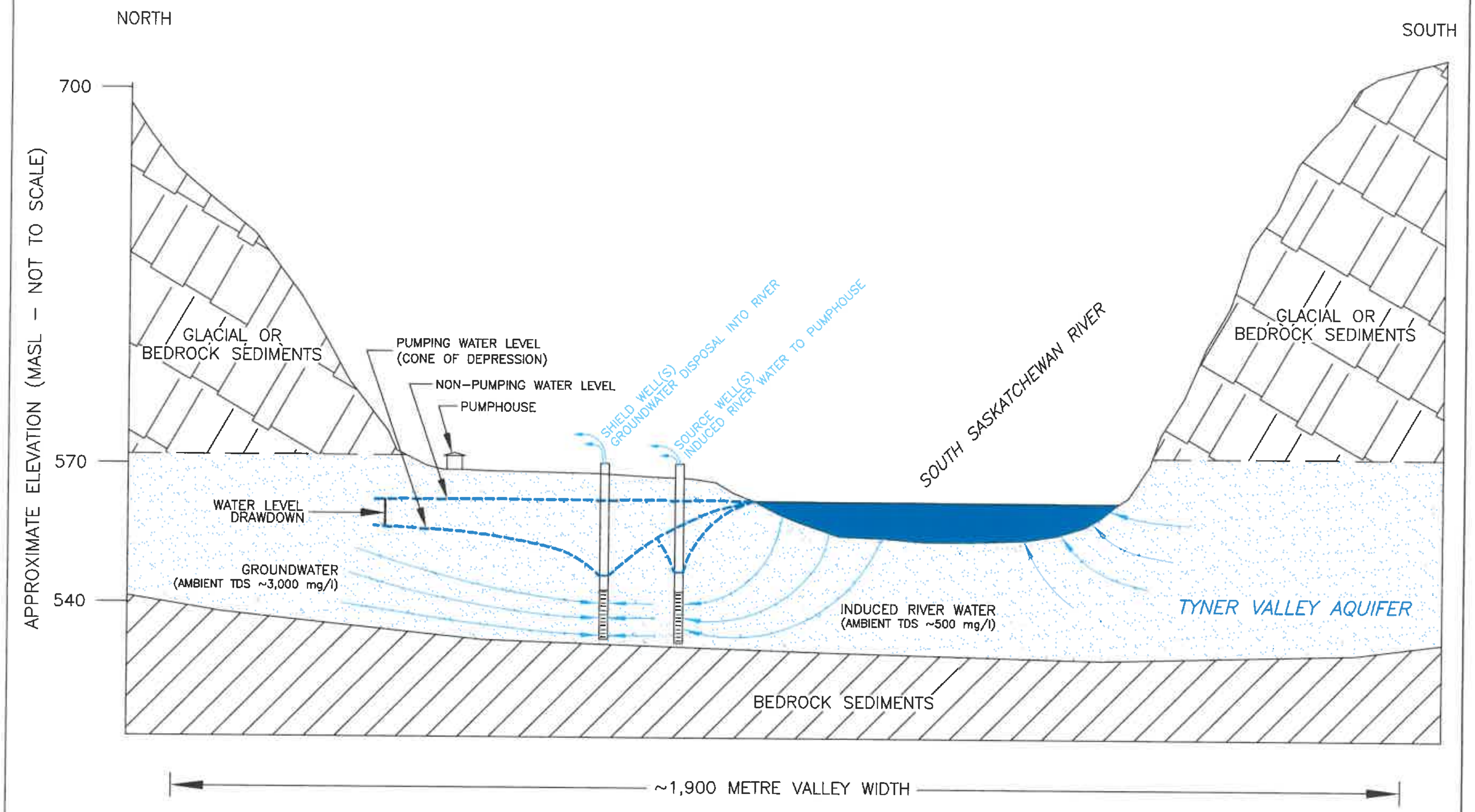
PRELIMINARY/ FOR DISCUSSION
NOT FOR CONSTRUCTION

FIGURE C-705

TOWN OF OUTLOOK
RAW WATER SUPPLY STUDY

CIVIL
CONCEPTUAL RAW WATERMAIN ROUTING FROM POTENTIAL WELL FIELDS

Ice Breaker



LEGEND

INFERRED DIRECTION OF GROUNDWATER FLOW

BECKIE HYDROGEOLOGISTS (1990) LTD.
CONSULTING ENGINEERS AND GEOSCIENTISTS
APEGS CERTIFICATE OF AUTHORIZATION No. 664

DRAWN BY MUTSCHLER
APPROVED BY FAMULAK
DATE JUNE 4, 2013

DESCRIPTION:
RECORD DRAWING

E–K WATER ADMINISTRATION BOARD

SCHEMATIC CROSS SECTION SHOWING INDUCED INFILTRATION AND SHIELD WELL INTERCEPTION PROCESSES

ACAD FILE: INDINF.DWG
SCALE: NOT TO SCALE
DRAWING NO. 1
PLOT DATE: JUNE 5, 2013

APPENDIX E – PMEL REPORT

P.MACHIBRODA
ENGINEERING LTD.
CONSULTING GEOTECHNICAL AND GEOENVIRONMENTAL ENGINEERS AND GEOSCIENTISTS

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- Piezocone (CPTu) Testing
- Soils Testing
- Concrete Testing
- Asphalt Testing
- Pile Driver Analyzer (PDA Testing)

Member of the Association of Consulting Engineering Companies/Canada

November 30, 2021

Associated Engineering (Sask.) Ltd.
1 – 2225 Northridge Drive
Saskatoon, Saskatchewan
S7L 6X7

**ATTENTION: RAHIM AHMAND, P.ENG.
PROJECT MANAGER, WATER DIVISION**

**RE: GEOTECHNICAL REVIEW
NEW RAW WATER SUPPLY STUDY
EAST BANK OF SOUTH SASKATCHEWAN RIVER
TOWN OF OUTLOOK, SASKATCHEWAN
PMEL PROJECT NO. 18533**

1 INTRODUCTION

1.1 GENERAL

The following letter has been prepared to summarize our geotechnical review of the proposed new raw water supply options for the Town of Outlook.

The raw water supply options part of the study that require the geotechnical review includes a new direct river intake (near pier 3 of the abandoned rail bridge), induced surface water infiltration wells (near existing direct river intake) and groundwater wells in the Tyner Valley Aquiver north of town.

The terms of reference for this assessment were presented in P. Machibroda Engineering Ltd. (PMEL) Proposal No. 18533, dated September 16, 2021. Authorization to proceed with the review was vis Associated Engineering (Sask) Ltd. Prime Consultant and Subconsultant Agreement, dated September 29, 2021.

2 BACKGROUND INFORMATION

PMEL previously completed the following slope stability studies for the Town of Outlook in regards to the East Bank of the South Saskatchewan River

- PMEL Report No. S08-6559, dated November 19, 2008;
- PMEL Report No. 9551, dated August 31, 2015;
- PMEL Report No. 12451, dated October 26, 2017; and
- PMEL Letter No. 16030, dated May 7, 2020.

In general, the existing slopes along the East Bank of the South Saskatchewan River within the Town of Outlook is an area of known historical/active landslide activity. The head scarp of the overall landslide was at or just upslope of the crest slope, with intermediate scarps located in areas along the mid-slope (indicating multiple landslide blocks). The toe of the overall landslide is hard to distinguish but is theorized to be at the toe of the slope (approximate shoreline of river) and/or under the river.

The ongoing slope movement has damaged the abandoned bridge, residential homes and other infrastructure (i.e., storm water pipes, existing sanitary pump station, etc.) on the slope or near the crest of slope. Based on PMEL past investigations the general soil conditions predominately consist of glacial till with intermittent sand deposits overlying clay shale. The clay shale was typically encountered at an elevation between approximately 485 to 475 m (approximately 25 to 35 m below existing grade) along the lower third of the slope, and at an elevation of between approximately 500 to 495 m below the crest of the slope (approximately 35 m below existing grade).

Based on E.A. Christiansen Consulting Ltd., *Report on the Geology of the Outlook Bridge, Saskatchewan* (Report No. 0150-002, dated September 15, 1994), the soil conditions within the South Saskatchewan River flood plain consists of alluvium sand and silt that is up to 25 m thick under the main river channel.

3 Site Review and Monitoring

A visual site review of the study sites and updated monitoring of the existing slope inclinometers and groundwater monitoring wells was completed on October 20, 2021. Select photographs taken during the site visit has been included in Appendix A.

3.1 Slope around Abandoned Rail Bridge

Based on the review of the slope in the area of the abandoned rail bridge, the slope has numerous signs of ongoing slope movement (refer to Photograph Nos. 1,2,3,6,7 and 8), but does not appear to have moved significantly since PMEL's previous monitoring (i.e., 2019). Some tension cracking appears to have progressed near the bridge (refer to Photograph Nos. 6 and 7). The existing raw water pumphouse does not appear to have experienced differential movement (possible indication of slope movement), but the power lines between the power poles that go to the pumphouse were tight (refer to Photograph No. 3). This may indicate that the power poles are moving and may be caused by slope movement. In addition, Pier 1 at the toe of the slope appears to have heaved upwards (refer to Photograph No. 4), likely due to slope movement (bulging at toe of landslide block).

3.2 Slope 1.5 km N of Abandoned Rail Bridge

The area where the Tyner Valley Aquifer crosses the South Saskatchewan River valley is approximately 1.5 km north of the rail bridge. Based on the review of the slope in this area revealed that the slopes have experienced significant slope movement as evidenced by deep tension cracking at or near the crest of slope and large slump blocks (Photograph No. 9). Groundwater seepage was also noted along the slope (Photograph Nos. 10 and 12).

3.3 Slope Inclinometers

Slope inclinometer readings were collected using a Digitilt Inclinometer Probe and Digitilt Datamate II data collector, owned and operated by PMEL. The results of the slope indicator readings presented as cumulative displacement, incremental change and cumulative change over time, have been shown plotted in Appendix B. A site plan showing the location of the SI's have been attached in Appendix C. An examination of the slope inclinometer plots revealed the following:

SI 14-1

- Negligible lateral slope movement was measured between August 29, 2019 and October 20, 2021.
- Since the initial measurements on April 29, 2015, approximately 15 mm of lateral slope movement has been measured at this location.
- As noted in the 2015 Geotechnical Report, this SI was likely not installed deep enough to fully capture the slip plane. However, based on the SI plots, it appears that a slip plane is located at approximately 19.5 m below existing grade in the glacial till and at 33 m below existing grade near the glacial till and clay shale interface.

SI 17-1

- Approximately 4 mm of lateral slope movement (total cumulative displacement) was measured between September 28, 2017 and August 29, 2019.
- Since the initial measurements on April 17, 2017, approximately 15 mm of lateral slope movement has been measured at this location.
- Two slip planes were identified, with an upper slip lane located at approximately 36.7 m below existing grade (Geodetic Elevation of approximately 486.4 m) and a lower plane located at approximately 47 m below existing grade (Geodetic Elevation of 476.1 m).

SI 15-1 and SI 15-2

- No measurements were obtained in SI 15-1 and SI 15-2 due to the casings having been previously sheared off.
- SI 15-1 sheared off sometime between August 2019 and September 2017 at approximately 32 m below existing grade (Elevation of 475.7 m).
- SI 15-2 sheared off sometime between September 2017 and May 2016 at approximately 35 m below existing grade (Elevation 476.5 m).

3.4 Groundwater Monitoring

A summary of the groundwater levels recorded in the standpipe piezometers has been presented in Table I. A site plan showing the location of the monitoring wells have been attached in Appendix C.

An examination of Table I shows that the change in groundwater levels varied significantly across the site. The groundwater levels measured in the piezometers in Test Hole Nos. 08-4, 17-1B, 17-2A, 17-2B, 17-3B and 17-5B were approximately 0.3 to 3.8 m lower since the last readings. The groundwater levels measured in the piezometers in Test Hole Nos. 17-1A, 17-3A and 17-5A were approximately 0.3 m higher since the last readings.

On average, the groundwater level was approximately 0.7m lower in comparison to the last monitoring period.

TABLE I RECORDED GROUNDWATER LEVELS

Test Hole No.	Piezometer Rim Elevation (metres)	Piezometer Tip Elevation (metres)	Ground Surface Elevation (metres)	Groundwater Elevation (metres)		
				September 28, 2017	August 29, 2019	October 20, 2021
08-1	513.0	471.3	512.0	505.9	505.2	--[1]
08-4	507.5	477.9	506.7	499.3	499.1	498.8
14-1A	537.7	528.2	536.7	532.5	532.2	--[2]
17-1A	536.9	515.3	535.8	520.1	527.4	528.0
17-1B	536.9	523.9	535.9	528.3	531.4	527.62
17-1C	536.7	529.9	535.8	Dry	Dry	Dry
17-2A	512.3	491.4	511.4	508.1	509.7	508.9
17-2B	512.5	501.0	511.4	509.8	510.1	509.3
17-3A	538.2	517.3	537.3	523.7	527.2	527.5
17-3B	538.2	526.8	537.3	528.7	527.9	Dry
17-4A*	532.3	511.3	531.3	524.6	NR**	--[3]
17-4B*	532.3	521.1	531.3	525.3	524.7	--[3]
17-5A	522.6	506.6	521.7	512.9	518.1	518.4
17-5B	522.7	517.3	521.7	518.5	518.3	517.7

[1] Obstruction blocking pipe at 6.2 m below top of pipe
[2] Obstruction blocking pipe at 3.7m below top of pipe
[3] Could not locate monitoring wells.

4 STABILITY REVIEW

4.1 DIRECT RIVER INTAKE

It is understood the proposed new intake would be situated in the middle of the river near Pier 3 of the abandoned rail bridge and will be connected to the existing raw water pump house and water line. It is also understood that a cofferdam will be constructed to install the new intake. The geotechnical issues with the new direct river intake would be related to construction in the river and long-term impacts of infrastructure on the slope.

The soil conditions within the river channel are anticipated to consist of a thick layer of sand and silt. The construction of a coffer dam in sandy soils will pose difficulties related to seepage flow that could occur around/under the coffer dam, which would impact the stability of the coffer dam. A geotechnical investigation and design should be undertaken by a qualified geotechnical engineer with experience in coffer dam designs.

The long-term issues with the new direct river intake would relate to slope movement impacting the existing and any new infrastructure on the slope. The slope is experiencing ongoing slope movement (though appears to have been minimal in the last couple years) and any infrastructure on the slope is at high risk of experiencing distress and damage. Structures based at grade would likely experience differential movement (i.e., tilting) and damage, as seen with the rail bridge. Buried utilities, like the water line, would experience breakages, particularly where a buried line crosses between different landslide blocks (i.e., locations with tension cracking). Since it isn't feasible to stabilize the slope (due to the deep slip plane and large size of the landslide), any infrastructure on the slope should be monitored closely and designed to accommodate slope movements (as well as possible). For buildings this could include using a rigid raft foundation, for buried utilities they could consist of installing instrumentation to monitor the slope movement and stresses on the pipes. This may enable repairs to the lines prior to breaks occurring.

4.2 Surface Water Induction Wells

It is understood that surface water induction well locations being considered are near the abandoned rail bridge. Considering the soil conditions on the slope near the river predominately consist of low permeable glacial till, the surface water induction wells will likely need to be based within the flood plain of the river. The geotechnical issues would be similar to the new intake in regard to impacts from slope movement.

Depending on the location of the wells in relation to the toe of the landslide, the wells may experience distress. This distress could consist of shearing off of the wells (if they intersect the slip surface) and heaving/tilting (from upward movement at toe of landslide). In addition, infrastructure along the slope will experience distress from slope movement (as described above for the direct water intake). Depending on the preferred location of the wells, test drilling and installation of slope inclinometers may be beneficial to determine the soil conditions and monitor for slope movement (to establish the elevation of the slip plane).

Alternately, the surface water induction wells could be located in areas that could lessen the risk for slope movement impacting the wells, as shown in Figure No. 1. The areas shown consist of the following:

A. On river bar on west half of the river behind the existing berm in the river. Though the wells would likely be stable, a new raw water line would need to cross the river and to connect to the existing water line. The same risks with infrastructure on the landslide would still apply.

B. Along the edge of the golf course is anticipated to be stable. Similar to above, risks with slope movement for the new raw water line would apply.

C. South of Hwy SK-15 bridge could be another option. The river bar along the east bank appears stable, though further investigation would need to be completed to determine the stability of the slope in this area.

FIGURE NO. 1 ALTERNATE LOCATIONS FOR SURFACE WATER INDUCTION WELLS

5 Induced Surface Water Induction Wells Tied to Tyner Valley Aquifer

The Tyner Valley Aquifer crosses the river valley approximately at the north boundary of the town limits (approximately 1.5 km north of the abandoned rail bridge). The geotechnical issues with this area are similar to what has been described above. Though the slope appears to be more active (i.e., greater magnitude of lateral slope movement) due to the large slump blocks and deep tension cracking present on the slope (refer to Figure No. 2). As such, risk of damage to infrastructure crossing the landslide blocks would be greater due to greater magnitude of slope movement. This would result in a greater frequency of infrastructure needing repair.

The one possible benefit would be that installing wells in the aquifer in this area may lower the pore pressures in the slope (i.e., lower groundwater table) which may help stabilize the slope or at the very least slow the slope movement. Further investigation with test drilling and installation of instrumentation (i.e., slope inclinometer, monitoring wells, etc.) and slope stability analysis would be required to determine the feasibility of groundwater wells in this area.

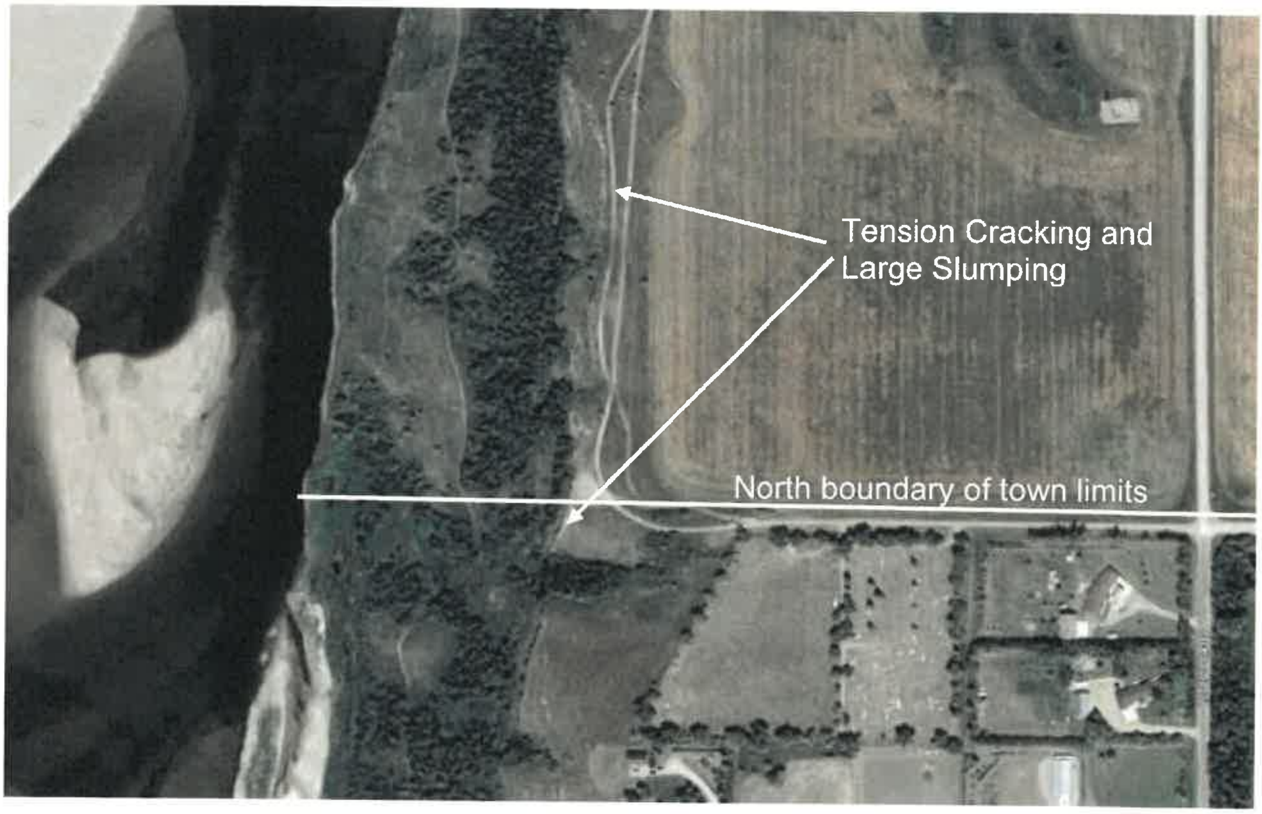


Figure No. 2 1.5 km North of Abandoned Rail Bridge

6 Conclusion

The desktop/visual slope assessment has been completed as authorized. This report has been prepared for the exclusive use of Associated Engineering (Sask) Ltd. and their agents for specific application to the Raw Water Supply Study for the Town of Outlook, Saskatchewan. Any use which a Third Party makes of this report, or any reliance on decisions to be made based on it, is the responsibility of such Third Party. PMEL accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

It is acknowledged that the Town of Outlook will be relying on this report with respect to the development and/or construction of the facilities described herein. Any use which a Third Party, other than the Town of Outlook, makes of this report, or any reliance on decisions to be made based on it, is the responsibility of such Third Party. Any other unspecified subsequent development would be considered Third Party and would, therefore, require prior review by PMEL. PMEL accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

If this report has been transmitted electronically, it has been secured with personal passwords to lock the document. Due to the possibility of digital modification, only originally signed reports and those reports sent directly by PMEL can be relied upon without fault.

We trust that this report fulfills your requirements for this project. Should you require additional information, please contact us.

P. MACHIBRODA ENGINEERING LTD.

Graham Baxter, P. Eng.

K. Pardoski

Kelly Pardoski, P. Eng.

GB:KP:zz

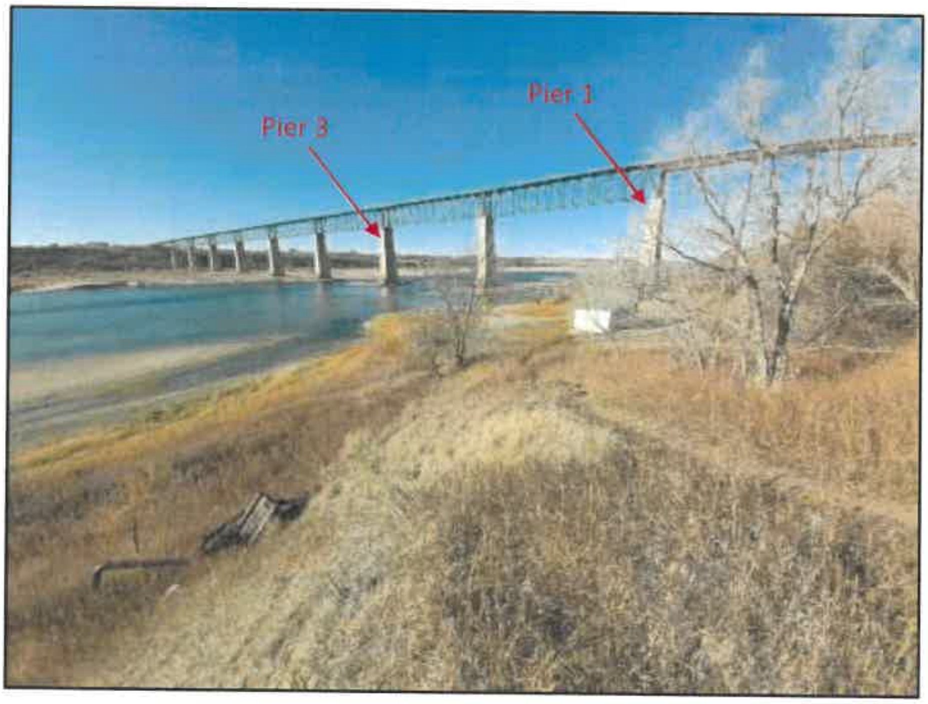
APPENDIX A

Photographs

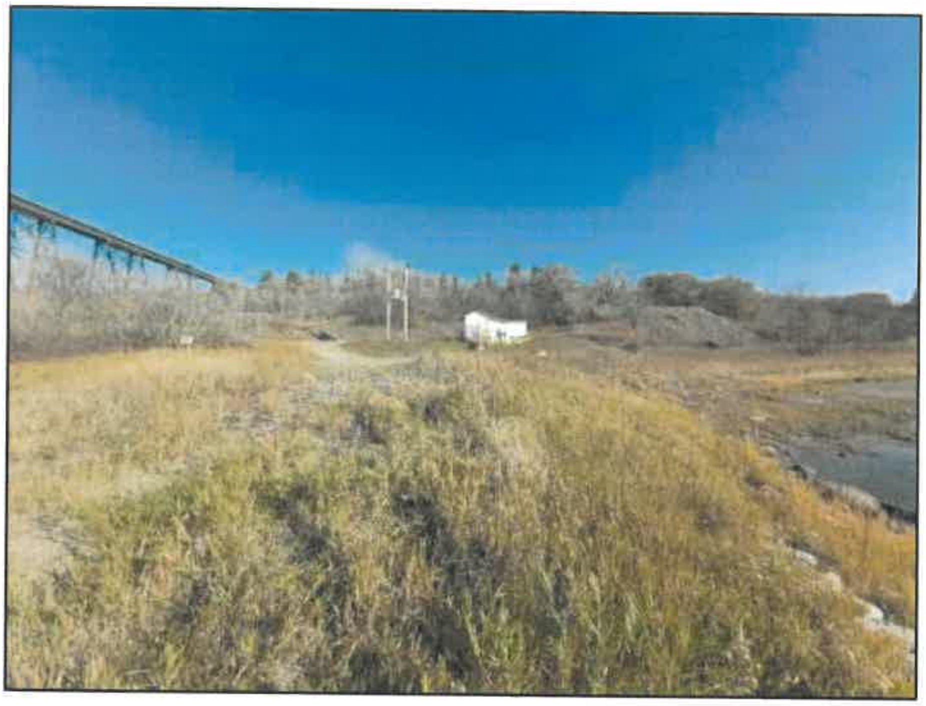
PHOTOGRAPH NO. 18533-01:
Buckling of truss near the abandoned rail bridge pier along the east shore of the river.

PHOTOGRAPH NO. 18533-02:
Gap under bridge pier on east slope.

PHOTOGRAPH NO. 18533-03:
Existing raw water pumphouse at toe of east slope. Power line was tight.



PHOTOGRAPH NO. 18533-04:
Looking northwest towards proposed new raw water intake location (next to Pier 3). Pier 1 at toe of slope appears to have heaved upwards.



PHOTOGRAPH NO. 18533-05:
Looking east towards existing pumphouse and east slope.

PHOTOGRAPH NO. 18533-06:
Tension crack in roadway along east slope. Location approximately 300 m south of rail bridge.

PHOTOGRAPH NO. 18533-07:
Tension cracking around existing sewage pumpstation, located north of rail bridge east abutment.

PHOTOGRAPH NO. 18533-08:
Looking west from east abutment of rail bridge.

PHOTOGRAPH NO. 18533-09:
Slumping and wide deep tension cracking along east slope, approximately 1.5 km north of rail bridge.

PHOTOGRAPH NO. 18533-10:
Groundwater seepage face along slope, downslope of area shown in Photo No. 18533-09

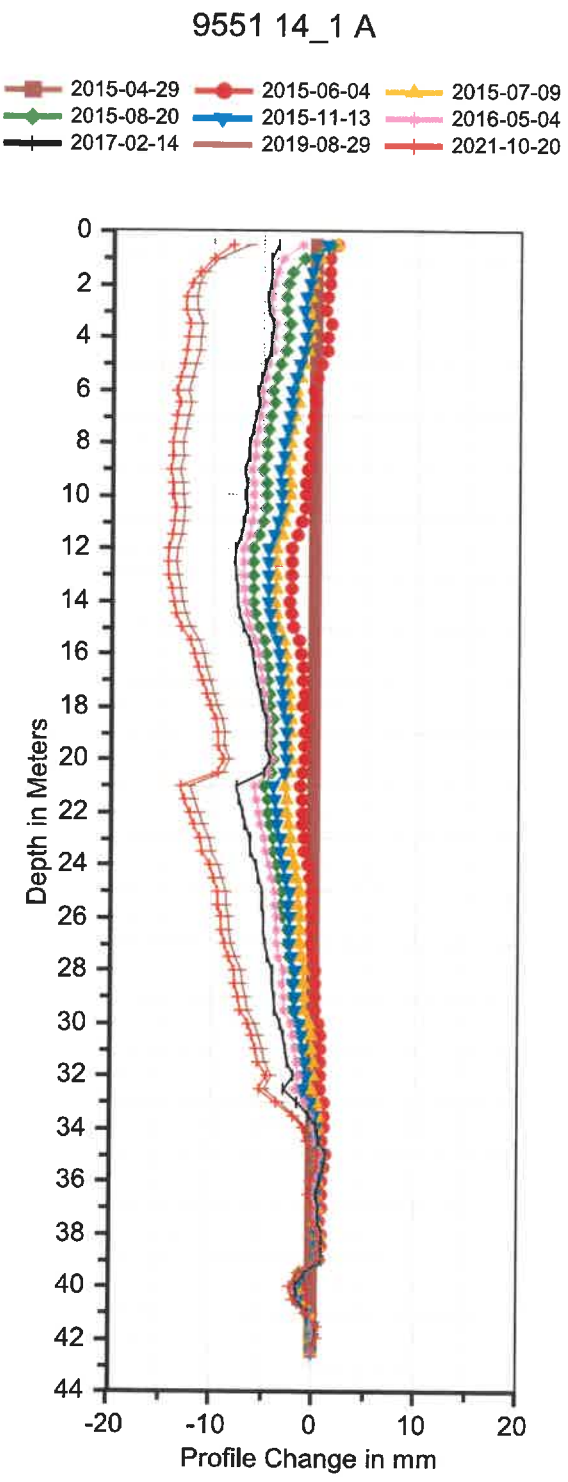
PHOTOGRAPH NO. 18533-11:
Midslope of east slope approximately 1.5 km north of rail bridge.

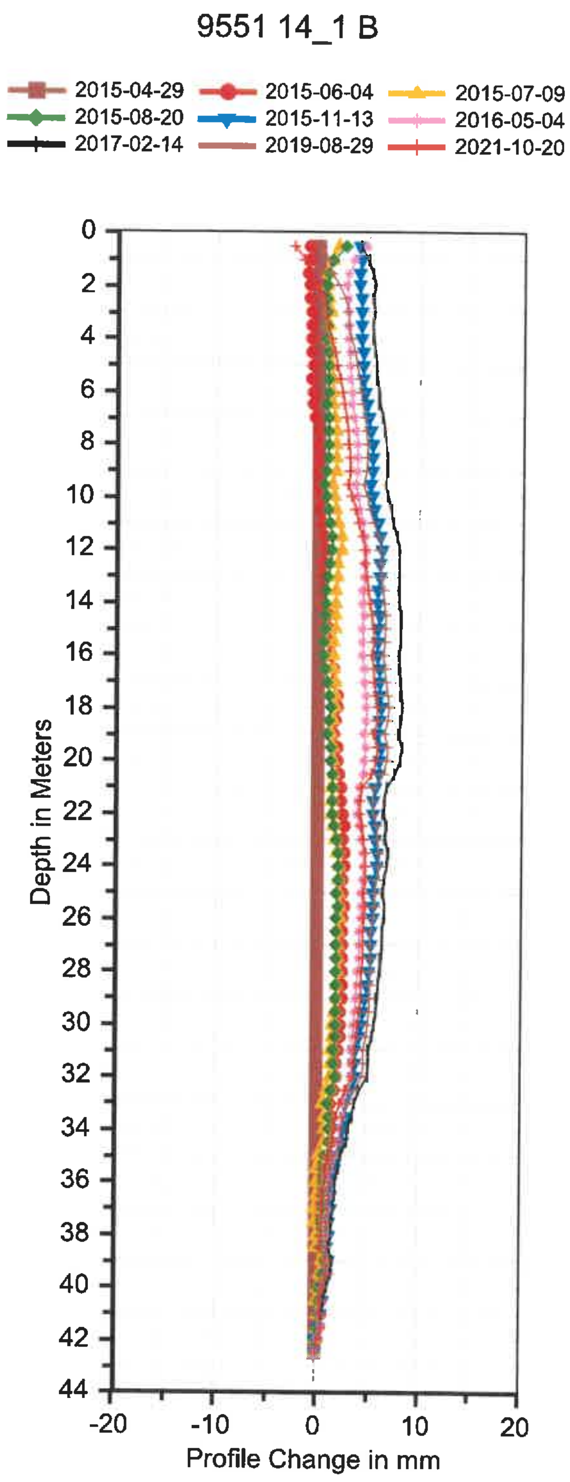
PHOTOGRAPH NO. 18533-12:
Near toe of slope near area shown in Photo No. 18533-11. Bare ground area (circled) had salt deposits on surface.

APPENDIX B

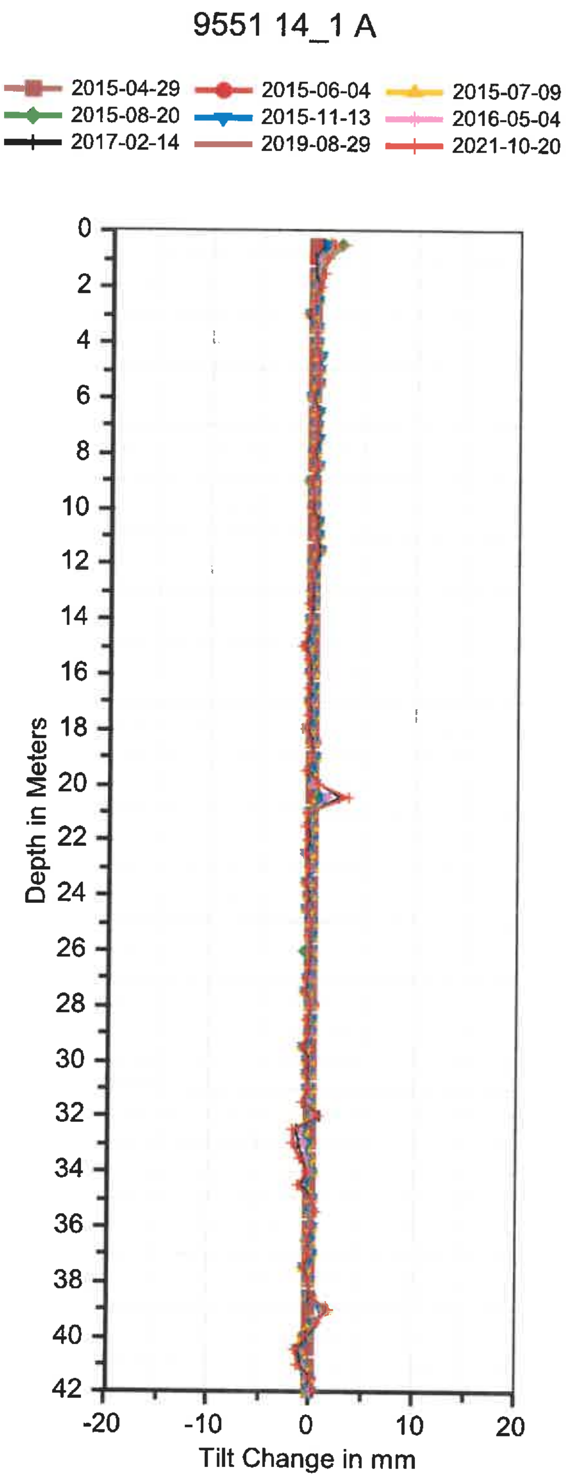
Slope Inclinometer Plots

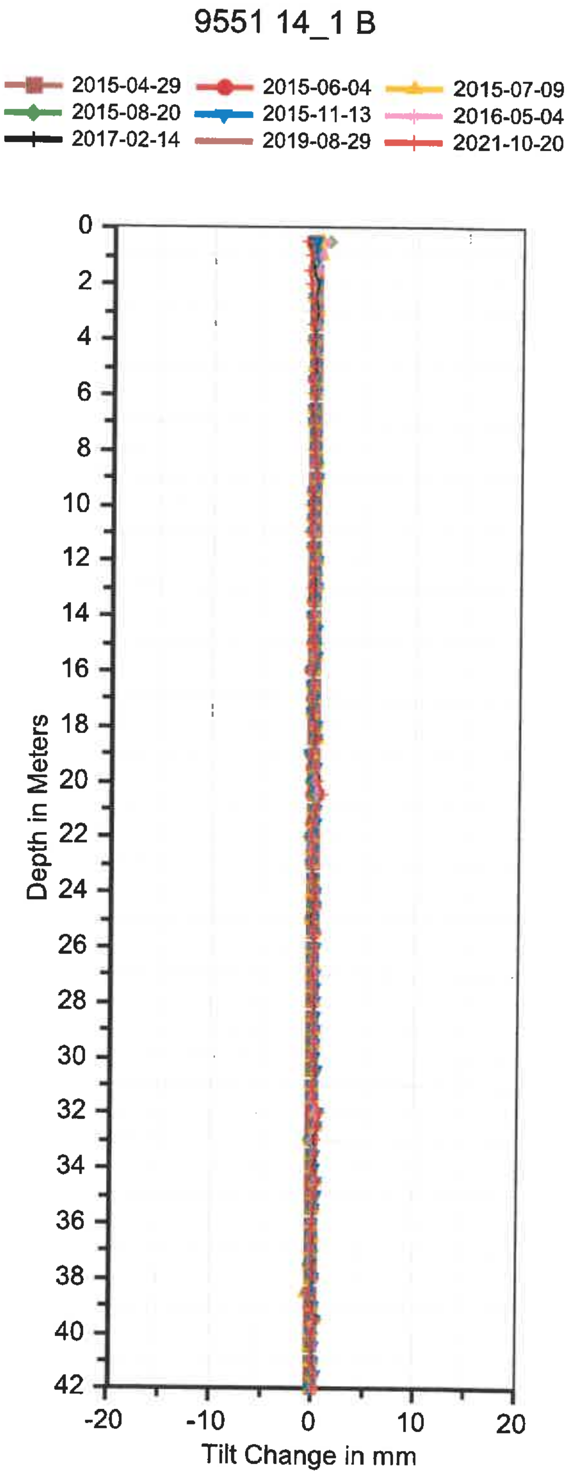
P.MACHIBRODA ENGINEERING LTD.



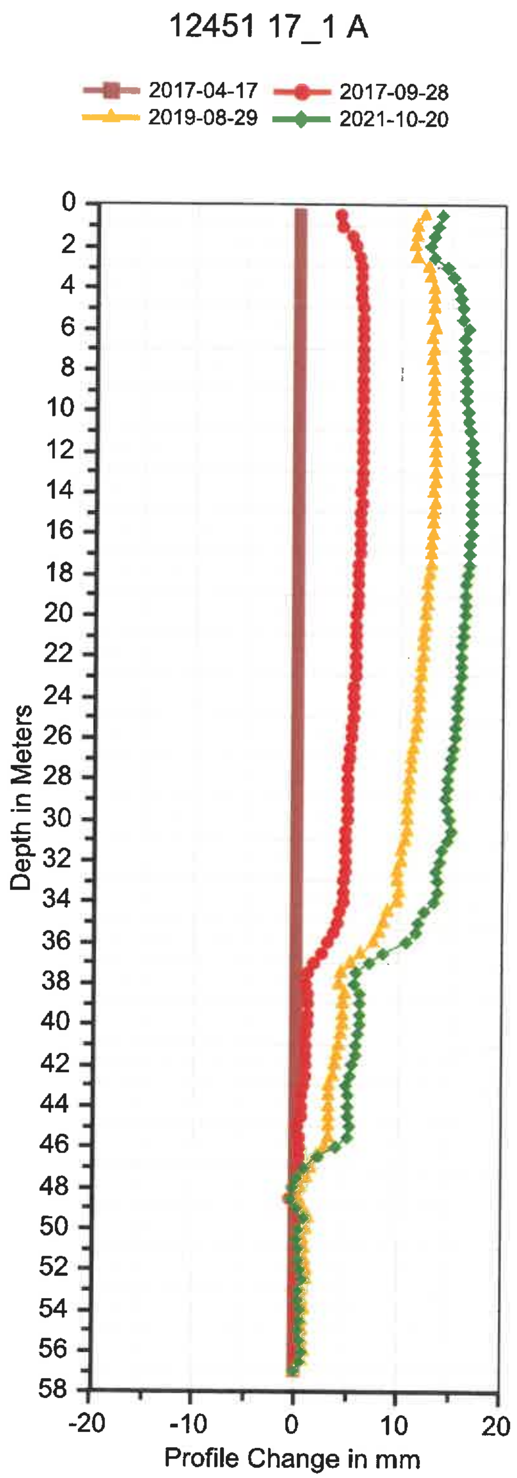


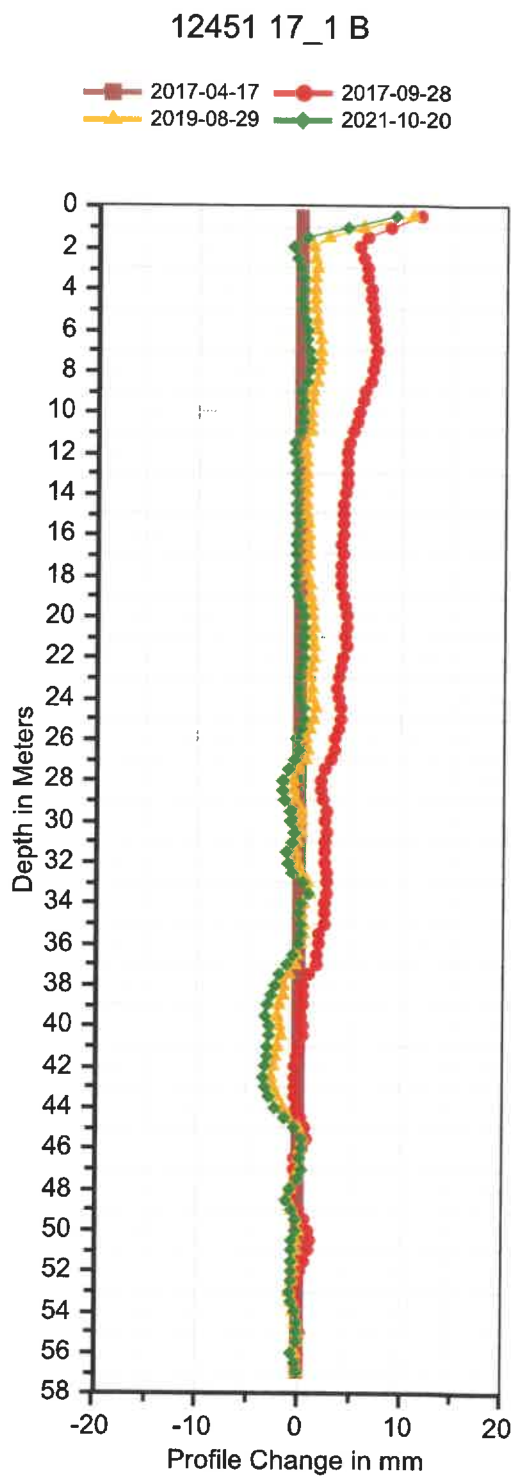
P. Machibroda Engineering Ltd.
Propoesd New Raw Water Intake
Outlook, Saskatchewan
18533
SI 9551 14-1



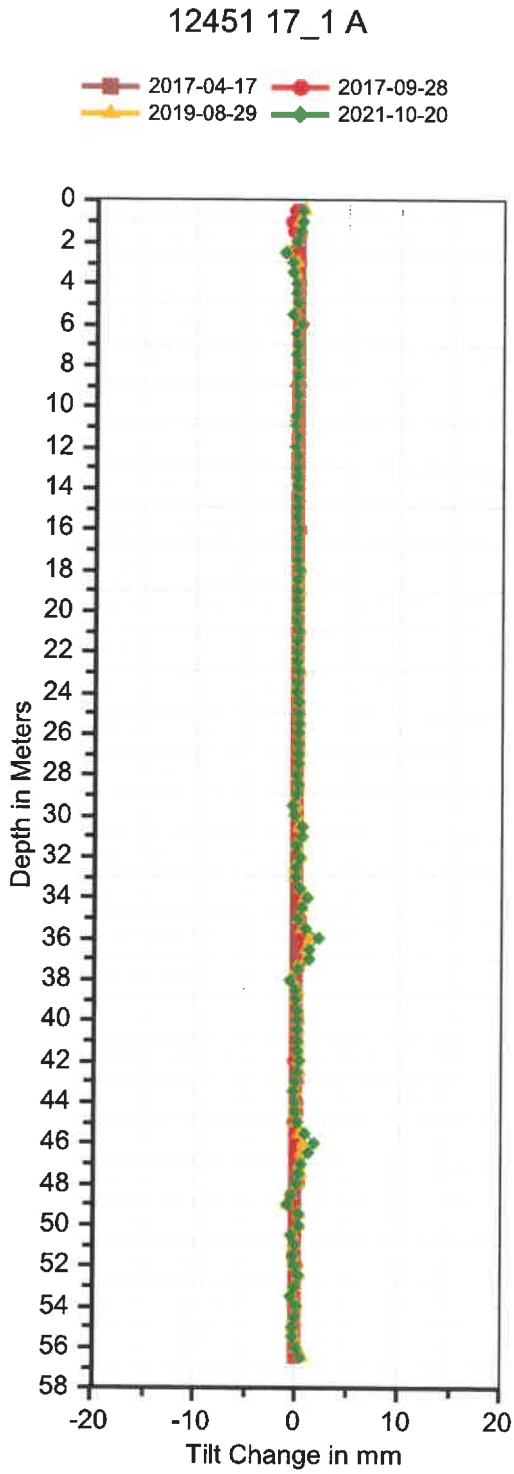


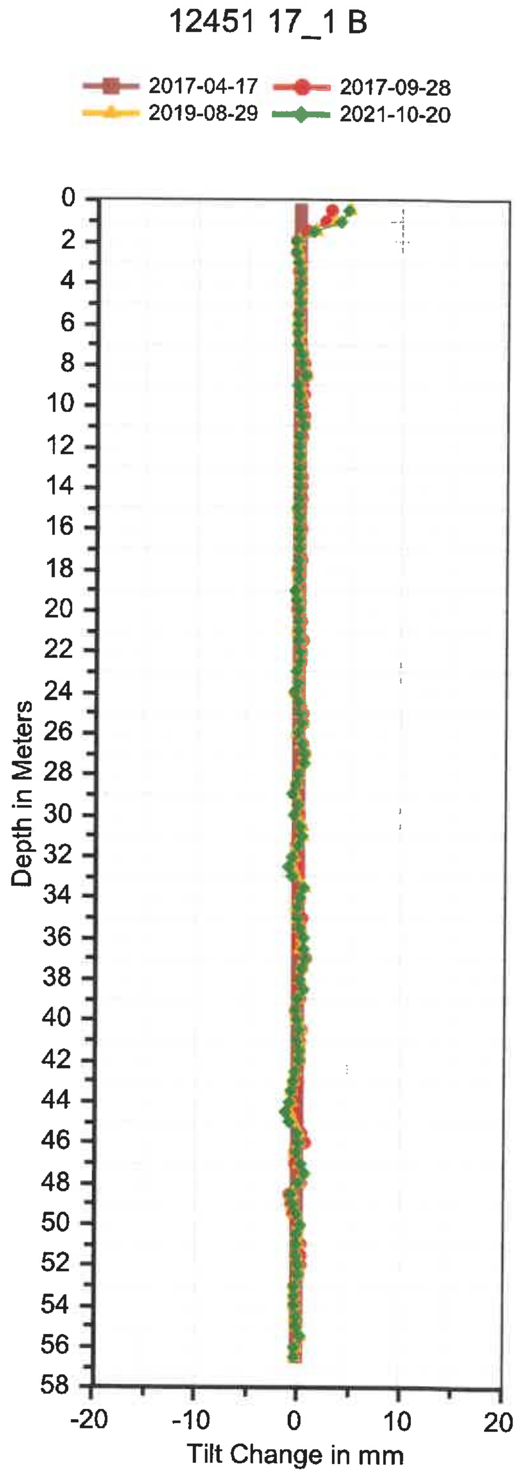
P. Machibroda Engineering Ltd.
Propoesd New Raw Water Intake
Outlook, Saskatchewan
18533
SI 9551 14-1





P. Machibroda Engineering Ltd.
Propoesd New Raw Water Intake
Outlook, Saskatchewan
18533
SI 12451 17-1





P. Machibroda Engineering Ltd.
Propoesd New Raw Water Intake
Outlook, Saskatchewan
18533
SI 12451 17-1

APPENDIX C

Site Plan, Drawing No. 12451-1

P.MACHIBRODA ENGINEERING LTD.

NOTE:
1. THIS DRAWING IS FOR CONCEPTUAL PURPOSES ONLY. ACTUAL LOCATIONS MAY VARY AND NOT ALL STRUCTURES ARE SHOWN.
2. THIS DRAWING WAS COMPILED USING AN AERIAL PHOTOGRAPH PROVIDED BY ISC.

LEGEND

- PMEL TEST HOLE
- PMEL TEST HOLE (PIEZOMETER INSTALLED)
- TENSION CRACKS
- BENCHMARK
- CREST OF SLOPE
- TOE OF SLOPE

P. MACHIBRODA ENGINEERING LTD.

CONSULTING
GEOENVIRONMENTAL
GEOTECHNICAL
ENGINEERS

806 - 48th STREET EAST
SASKATOON, SK
S7K 3Y4

DRAWING TITLE:
SITE PLAN - TEST HOLE LOCATIONS

PROJECT:
SLOPE STABILITY AND DEWATERING ASSESSMENT
EAST BANK OF SOUTH SASKATCHEWAN RIVER OUTLOOK, SK

APPROVED BY: GB	DRAWN BY: SD
DATE: MARCH, 2017	DRAWING NUMBER: 12451-1
SCALE: 1:5000	

APPENDIX F - OPINION OF PROBABLE COST