Report

Town of La Ronge

Mowery Subdivision

Preliminary Design
Final Report

November 2009

ASSOCIATION OF PROFESSIONAL ENGINEERS
AND GEOSCIENTISTS OF SASKATCHEWAN
CERTIFICATE OF AUTHORIZATION
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Executive Summary

Associated Engineering was authorized by the Town of La Ronge, through the project manager Glen Gillis of SaskWater Corporation, to undertake a study of Parcel J, which is the area bounded by Boardman Street, Studer Street, Bedford Street, Cook Crescent and Lawton Crescent. The purpose of the study was to assess feasibility of development of the area to residential subdivision (ie: the Mowery Subdivision) and to complete preliminary design of the first phase of development.

The work involved compiling records of the Town's water and sewer infrastructure and previous geotechnical information, coordination of a new geotechnical investigation and analysis of the data, assessment of the existing infrastructure's ability to service the new subdivision, and developing options for upgrading and servicing, including order-of-magnitude cost estimates.

A conceptual lot layout for over 200 lots developed including two options for the first phase of development. Analysis at the existing infrastructure showed that it is likely possible to service the entire new subdivision by upgrading and expanding the existing water and sewer network, although a number of issues with existing were identified, including lack of capacity at Sewage Pump Stations (SPS) No. 8, 7 and 2 due to suspected high infiltration and wet inflows, and uncertainty around the configuration of the existing water loop no. 3. It is recommended that these issues be investigated further prior to construction of development beyond phase one.

The cost for Phase One Option 2 is estimated at $1.68 million for 32 lots, or $53,000 per lot not including costs of potential upgrades to SPS 8 or development type levies collected to pay for upgrades to common infrastructure that would be required to service future phases.

Table of Contents

1 Introduction

The Town of La Ronge (the Town) has authorized Associated Engineering (AE) to undertake preliminary design of the proposed Mowery subdivision. With the growing population and new housing demands there is a need to expand the existing infrastructure to accommodate the town's population growth. The area surrounding the proposed Mowery subdivision has been developed over several phases dating back to the 1970's. The future Mowery subdivision is the next phase of development in this area and will include several streets, crescents and cul-de-sacs, with an excess of 200 lots likely to be developed within a total area of 36.5 hectares.

Figure 1 in Appendix A shows the study area and the relative location for this development. This report is a summary of our preliminary findings regarding municipal servicing and development of this concept.

1.1 OBJECTIVE

The main objectives in this report are as follows:

1. To summarize previously gathered data along with new data obtained during this project;
2. Examine the feasibility of this subdivision and the compatibility with existing infrastructure in the area;
3. Provide and evaluate infrastructure options for servicing the new development with water, sanitary sewer and drainage along with phasing of potential construction; and
4. Provide recommendations and estimated costs for the first phase.

1.2 BACKGROUND

Information and data used in preparation of this report includes:

- Record drawings of subdivision development obtained from the Town
- The EPA Net water system hydraulic model obtained from UMA/AECOM for the 2008 La Ronge Regional Project.
- The 1999 *Saskatchewan Municipal Affairs, Culture and Housing Northern Infrastructure Study*, AE/UMA (the SMACH report)
- The 2005 *Waterworks System Assessment* (2005 WSA) report by UMA/AECOM
- The May 2002 *Sewage Pump Station Analysis* draft report by UMA (2002 UMA report)
- Lift station pump hour records obtained from the Town
- Geotechnical information from subdivision record drawings
- Geotechnical information from a January 2009 test hole drilling program conducted by P. Machibroda Engineering Ltd. (PMEL)
- Geotechnical information from a January 2009 test pit excavation program conducted by AE.
- Cut line clearing and topographic surveys in 2008 and 2009 conducted by AE
- Site visits by AE design staff on December 10, 2008 and January 12, 2009.

2 Existing Conditions

2.1 AREA TOPOGRAPHY AND DRAINAGE

Contours of the study area were generated from survey data (using Autodesk Land Development Desktop) and are shown in Figure 2 in Appendix A. An overall drainage concept for the area is also included. It is notable that the contours and drainage are approximate only, due to the density and spacing of survey data (25 meter grid) which is limited due to the extensive tree cover.

In general terms the area appears to drain from the north and west (near Mowery Place and Studer Street) south to the area between Kowalski Place and Cook Crescent. Drainage in the area is currently accommodated via a system of culverts alongside and perpendicular to Bedford Drive directing flows to the southeast of Bedford and Louis Road.

2.2 GEOTECHNICAL

Geotechnical information was obtained from previous subdivision record drawings as well as the 2009 drilling and test pit excavation programs. Figure 3 in Appendix A shows the location of all test holes which have been drilled in the area; Figure 4 shows test holes data from the 1974 drawings, Figure 5 shows test holes data from the 1976 drawings and Figure 6 shows test holes and test pits data from the 2009 program.

Generally the soils consist of organic peat overlying variable deposits of sand, silt, clay and glacial till. Auger refusal was encountered in several holes indicating the presence of a highly variable bedrock layer. Groundwater was also encountered in several holes. Locations with bedrock less than 2.5 m from surface and groundwater less than 1.0 m from surface are highlighted in Figure 3. Areas with multiple occurrences as such have been shown as “difficult to service due to soil conditions.”

Appendix B contains the 2009 geotechnical report prepared by PMEL as well as the AE test pit records.

2.3 POTABLE WATER

Potable water for the Town comes from the water treatment plant located on the shore of Lac La Ronge, and is distributed via three reservoirs and three potable water loops. Figure 7 – Existing Water System shows the existing distribution loops around the new subdivision.

According to the 1999 SMACH report, the distribution system consists of epoxy coated steel, high density polyethylene (HDPE) and polyvinyl chloride (PVC) pipe. According to record drawings, some older mains are Ductile Iron (DI). The water mains are typically 150 mm diameter while some side streets are serviced by 50 mm diameter pipe. Most structures are serviced with 20 mm diameter heat traced or thaw wire copper service pipe from the mains.

The water treatment plant has two reservoirs, 1A and 1B, that feed Loop No. 1, which services the area from La Ronge Ave to Hildebrand Drive, Bedford Drive and Lawton Crescent. Reservoirs No. 2 and No. 3 are also supplied by Loop No. 1. Reservoir No. 2 feeds Loop No. 2 which supplies Studer Street from Lawton Crescent to the south end of Quandt Crescent, Riese Drive, Dalby Crescent, and Mowery Place. Reservoir No. 3 feeds Loop No. 3 which supplies Studer Street from the south end of Quandt Crescent to Boardman Street, Sinotte Crescent, Aronec Place, Thompson Crescent, Sewell Place and the industrial area west of Highway No. 2. Table 2-1 below, from the 2005 WSA, summarizes the potable water infrastructure for La Ronge.

Table 2-1
Water System Capacities

Equipment	**Rated Capacity** (1)	**Demand Condition**	**Required Capacity**	
			Current Demand	2015 Demand (2)
Raw Water Intake Structure	Unknown	Peak Day + 12% backwash	32.1 L/s	39.2 L/s
Raw Water Intake Line	Unknown(3)	Peak Day + 12% backwash	32.1 L/s	39.2 L/s
Raw Water Pumps	53.6 L/s @ 23.8 m TDH	Peak Day + 12% backwash	32.1 L/s	39.2 L/s
Water Treatment Units	50 L/s(4)	Peak Day + 12% backwash	32.1 L/s	39.2 L/s
Treated Water Reservoir 1A & 1B	2,750 m^3	2 days at average day demand	2,640 m^3 (5)	2,640 m^3 (5)
Backwash Pump	72.0 L/s @ 13.7 m TDH	16 USGPM/ft^2 + 53 USGPM surface wash	72 L/s	72 L/s
Backwash Sump	200 m^3	One backwash cycle per filter	82 m^3	82 m^3
WTP Distribution Pumps (Loop 1)	76 L/s @ 56 m TDH	Peak Hour (6)	59.9 L/s	73.0 L/s
WTP Standby Pump (Loop 1)	31.5 L/s @ 56 m TDH			
Treated Water	617 m^3	2 days at	172 m^3 (7)	535 m^3 (7)

Equipment	**Rated Capacity** (1)	**Demand Condition**	**Required Capacity**	
			Current Demand	2015 Demand (2)
Reservoir No. 2 (Loop 2)		average day demand		
Reservoir No. 2 Distribution Pumps (Loop 2)	19 L/s (8) @ 46 m TDH	Peak Hour (6)	4.5 L/s	14.0 L/s
Reservoir No. 2 Propane Engine Driven Standby Pump (Loop 2)	50.8 L/s @ 54 m TDH			
Treated Water Reservoir No. 3 (Loop 3)	2,700 m^3	2 days at average day demand	450 m^3 (7)	812 m^3 (7)
Reservoir No. 3 Distribution Pumps (Loop 3)	27.0 L/s @ 53 m TDH	Peak Hour (6)	11.7 L/s	21.2 L/s
Reservoir No. 3 Propane Engine Driven Standby Pump (Loop 3)	50.8 L/s @ 54 m TDH			

(1) Information from Water Treatment Plant record drawings or operation and maintenance manuals.
(2) Current demands + 2% annual growth. Future growth will be directly from Loop No. 2 and Loop No. 3.
(3) Maximum capacity limited by required net positive suction head (NPSH) of raw water pumps.
(4) Currently operating at 33 L/s.
(5) Distribution Loop No. 1 services 80% of current distribution system.
(6) With both distribution pumps running.
(7) For area serviced. See description under section 2.1 of WSA.
(8) Installed in 2006.

2.3.1 Regional System Upgraded Water Capacities

The Town is a partner in the Lac La Ronge Regional Water Corporation, which is in the process of upgrading the water supply, treatment and distribution for the area. As part of this project, the raw water supply, water treatment and Loop No. 1 capacities are all being upgraded, with completion expected mid 2010. The future capacity for La Ronge that has been accommodated in the design allows for a future La Ronge population of 4,147 people (20 years at 2% growth using 2006 base population estimated at 2,725) or an additional 1,400 people approximately. This Mowery Subdivision will likely be the major growth area for the Town and the future capacity (for supply) from the regional system should be enough to supply the new Mowery Subdivision provided the distribution system loops can be upgraded (to be discussed later in this report).

2.4 SANITARY SEWER

The existing sanitary sewer network for the entire town is shown in Figure 8. Information was not available for all areas, however the network and contributing areas for the Town's sewage pump stations are shown approximately.

The sewage system surrounding the proposed subdivision consists of gravity sanitary sewer mains which collect sewage from adjacent lots and deliver it to one of three pumping stations. See Figure 9 for SPS collection areas for the study area. The gravity sewer is typically made of 200 mm diameter vitrified clay tile or PVC pipe. Service connections are typically 100 mm diameter clay tile or PVC pipe.

2.5 SEWAGE PUMPING STATIONS

La Ronge has 12 sewage pumping stations which collect and pump sewage from the community to the sewage treatment plant. The area surrounding the proposed subdivision development is serviced by three Sewage Pump Stations (SPS): No. 2, No. 7 and No. 8.

SPS No. 8 (commissioned in April 1978) collects from the industrial area west of Highway 2 plus the west side of Studer, south of Quandt Crescent (247 lots) and pumps through a 150 mm force main which discharges to MH-10 near the north entrance to Quandt Crescent. It then gravity flows to SPS No. 7.

SPS No. 7 (commissioned in 1976) collects from the north area of Studer north of Quandt Crescent including all of Reese Drive and its connecting roads (147 lots) and pumps via a 150 mm cast iron force main east on Studer connecting to the 150 mm force main at Bedford and Studer. From there, the sewage flows to the sewage and treatment plant via a 250 mm force main.

SPS No. 2 (commissioned in 1974) collects from the portion of Studer Street east of Riese and the area along Bedford south of Studer Street (135 lots); from Diefenbaker, Kowalski and Guy Place (44 lots); as well as servicing the flows from SPS No. 1 and SPS No. 10 (approximately 96 units total). SPS No. 2 discharges via a 150 mm forcemain that is twinned with a 200 mm forcemain running on the east side of Bedford Drive (installed in 1994). The forcemain from SPS No. 3 (200 mm) ties into this forcemain at a point just north of Louis Road on Bedford Drive. At a point just north of Bedford and Studer this 200 mm forcemain connects to the 250 mm forcemain running to the STP. The 150 mm forcemain from SPS No. 1 joins the original 150 mm forcemain from SPS No. 2 just north of SPS No. 2. This original forcemain continues on the west side of Bedford and joins the 150 mm forcemain from SPS No. 7 at Bedford and Studer where it becomes a 250 mm forcemain.

SPS 8 Forcemain – Plugging Issues

Discussions with Wally Perada of the town's Public Works department have revealed operational problems with the force main servicing SPS No. 7. Along Studer Street, south of Lawton Crescent, the force main has become plugged and flow is restricted, likely at the low point where a "flush out" was installed. This repeated plugging required flows from SPS No. 7 to be diverted during the study period to the gravity sewer (MH-1) at Lawton Crescent and Studer Street, which flows to SPS No. 2. As a result, all flows from SPS No. 7 and No. 8 during the period of this study are reported to be pumped through SPS No. 2. This places an unnecessary strain on the pumps at SPS No. 2. From discussions with the Town staff, the Town corrected this problem during the summer of 2009.

Data from the 2002 Sewage Pumping Station Analysis report by UMA, (contained in Appendix C) and the 2005 WSA by AECOM is summarized below in Table 2-2.

Table 2-2
Sewage Pumping Station Capacity and Flow Estimates

Sewage Pumping Station Number[4]	**Pumps To**	**Pump Hp and Manufacturer**[1]	**Pumping Capacity L/s (@ m TDH)** [1]	**Existing Wet Well Size (L)**	**Recom. Wet Well Size for pump (L)**[2]	**Average Model Inflow (L/s)**[3]	**Peak Model Inflow (L/s)**[3]
1	2	15/Morris	18.2/9.1	850	3030	2.5	4.9
2	STP	30/Morris	21.6/15.2	3000	4000	4.9	9.8
3	STP	30/Flygt	35.0/?	4600	4755	18.2	34.5
4	3	7.5/Morris	22.0/?	3700	3300	9.9	19.8
5	4	15/Gourds	5.7/?	600	1600	2.5	5.0
7	STP	7.5/Gourds	19.2/7.3	2700	3300	9.0	18.0
8	7	7.5/Morris	14.2/11	2000	2550	5.2	10.4
9	3	2/Aurora-P.	5.7/12.2	1600	n/a	2.0 (est.)	4.0 (est.)
10	2	7.5/Morris	25.6/5.2	700	n/a	n/a	n/a

Notes:

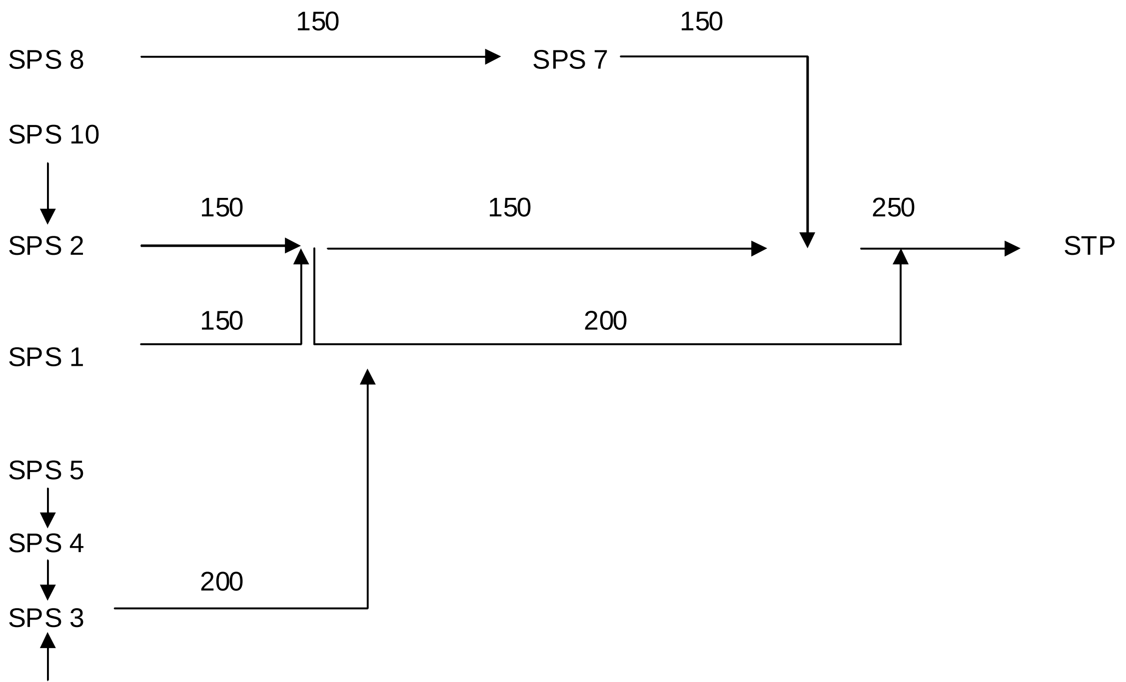
(1) The pump sizes and capacities listed are from the 2002 UMA "Sewage Pump Station Analysis" and the 2005 AECOM WSA and should be confirmed. The capacities appear to be based on 90 percent of the manufacturer ratings. Pumps in SPS No. 3, 4 and 5 were changed between the 2002 UMA study and the 2005 WSA and the discharge head at duty points (TDH) was not available for these pumps.

(2) The 2002 UMA report did not provide a rationale for the "Recommended Wet Well Size".

(3) Origin of the "model" flows was not fully explained in the 2002 UMA report.
(4) There is no SPS No. 6. There are three other sewage pumping stations that were not studied in the 2002 UMA report: Police Point SPS 11, East La Ronge Ave SPS 12, and the Senior Citizen's pumpstations. The 2005 WSA reported they each pump less than one hour per day, capacity unknown, all pump to SPS 3.

2.6 SEQUENCE OF WASTE WATER FLOWS

Three sewage pump stations discharge directly to the STP via the 250 mm force main starting at the intersection of Bedford and Studer: SPS 2, 3, and 7. All the other flows from the remaining SPS's are directed to the gravity systems for these three or to their forcemains as shown below:



SPS 9, Senior Citizens SPS, Police Point SPS 11 and East La Ronge Avenue SPS 12 (configuration unclear and not part of this study).

Note: This sequence of wastewater flows has been determined from the 2002 UMA Study, 1994 record drawings from Bullee Consulting Ltd., and 2007 records from AECOM.

2.7 ANALYSIS OF SANITARY FLOWS

The Town has provided log sheets of daily pump hour meter readings for SPS No. 2, No. 7 and No. 8 for the 2008 calendar year. An estimate of sewage flows was made from pump run times and the original pump capacity (ie: worst case versus using the 90 percent efficient capacities assumed in the 2005 WSA) with the results summarized in Appendix C. Flows are estimated assuming 24 hour between hour meter readings. Analysis showed the following:

- SPS No. 8 (original Capacity 15.8 L/s; reported capacity 14.2 L/s)
    - Dry weather months (November through February) averaged 4.4 L/s with maximum day of these months at 7.9 L/s;
    - Wet weather months saw maximum day flows of 21.1 L/s; wet in-flow estimated at 0 to 13.2 L/s (ie. 21.1 minus 7.9)
    - Over the entire period the average day was 4.9 L/s;

- SPS No. 7 (Original Capacity 22.1 L/s; reported capacity 19.2 L/s)
    - Dry weather months (November through February) averaged 12.1 L/s with maximum day of these months at 16.6 L/s;
    - Wet weather months saw pump hours in excess of 25 per day; estimated maximum day flows of 23.0 L/s; wet in-flow estimated at 6.4 L/s (ie. 23.0 minus 16.6);
    - Over the entire period the average day was 11.6 L/s;

- SPS No. 2 (Original Capacity 24.0 L/s; reported capacity 21.6 L/s)
    - Dry weather months (November through February) averaged 7.6 L/s with maximum day of these months at 12.0 L/s;
    - Wet weather months saw pump hours in excess of 30 per day; estimated maximum day flows of 30.0 L/s; wet in-flow estimated at 0 to 18 L/s (ie. 30 minus 12)
    - Over the entire period the average day was 7.8 L/s;
    - *Note: Over the one year period there were a number of high pump hour readings, some of which corresponded to operator notes about problems (plugging, debris in pumps, etc).*
    - *Note: Pump hour records indicate that Pump No. 1 runs more frequently than Pump No. 2. Records also show that total hours combined often exceeds 24 hours suggesting both pumps must operate in tandem excessively. This indicates there may be a problem with Pump No. 1, perhaps due to a poor impeller, debris, plugging or other factors. It is recommended that a pump down test be performed to help identify problems with Pump No. 1.*

The SPS data analysis is contained in Appendix C. The data for the lift stations is not detailed enough to estimate a peak hour factor so a typical value of 2.0 X max dry day plus wet in-flow will be used to calculate the current peak as follows:

Current Peak flow = (2 x max day) plus Extraneous flow

SPS 8 Peak flow = (2 x 7.9) + (0 to 13.2) = 15.8 to 29.0 L/s
SPS 7 Peak flow = (2 x 16.6) + (0 to 6.4) = 33.2 to 39.6 L/s
SPS 2 Peak flow = (2 x 12.0) + (0 to 22.2) = 24.0 to 46.2 L/s

Note: These estimates of current peak flow or "worst case" all exceed the rated capacities of the installed pumps. Further investigation is recommended.

2.8 SEWAGE TREATMENT

The sewage treatment plant is located at the north end of Bedford Drive. Sewage flows to the plant via a single gravity main.

Sewage treatment is accomplished through a sequencing batch reactor process. According to the 2005 WSA the sewage treatment plant has two units with a combined capacity to treat 28.1 L/s (2420 m^3/day) of raw sanitary wastewater and a peak flow capacity of 80.1 L/s. Effluent is discharged into a muskeg area located to the north of the plant. The muskeg drains to McGibbon Bay on Lac La Ronge. The sewage treatment plant was last upgraded in 2003/04. According to the 2005 WSA the plant is currently meeting effluent limits set by the town's operating permit.

Assuming a current (2006) population of 2725 and water demand of 490 L/c/day the water demand is estimated at 1335 m^3/day. Assuming 100% of water directed to the sanitary system, the plant has excess capacity of (2420-1335) = 1084 people on an average day. This should provide enough capacity for all the population growth of the proposed subdivision.

2.9 STORM DRAINAGE

Drainage in the town is achieved through overland drainage to lower areas. The drainage plan is shown on drawing 4412-100 – Town of La Ronge Drainage Plan in Appendix A. The proposed development area drains over land to the south. Water is conveyed through several culverts under Bedford Drive and eventually toward Lac La Ronge. This drainage pattern should be maintained during development.

Development of the Mowery Subdivision may require the incorporation of retention ponds and/or the upgrading of downstream culverts and ditches. Further investigation is recommended once the development plan is adopted.

3 Subdivision Design Criteria

Preliminary design for the new subdivision has been based on a concept plan developed by AE. Review of this plan by an Urban Planner would be beneficial and is highly recommended. The town has not adopted a complete set of subdivision design standards at this time so this conceptual design has been based on:

- Specific design details as provided by the Town;
- Conformance with previous phases (determined from record drawings);
- The Province of Saskatchewan Subdivision Regulations, where La Ronge standards were not available;
- Saskatchewan Environment Standards and Guidelines; and
- Standard engineering practice.

In addition to the standards described, the preliminary design has been prepared based on GPS field survey gathered by AE, on records provided by the Town, and borehole logs from a geotechnical investigation conducted by P. Machibroda Engineering Ltd in February 2009. The geotechnical report is included in Appendix B.

3.1 FLOW ESTIMATES

Future flows were estimated for the proposed subdivision by the following:

- Water consumption – 490 lpcd (from 2005 WSA);
- Sewage generation estimated at 100% of water use;
- 2.9 persons per dwelling unit (2006 Canada census data for La Ronge);
- Approximately 670 new residents in the subdivision and build out; and
- Peak hour to average day peaking factor = 3.4 (using Harman's formula based on population).

Note: The 2002 UMA Study used a peaking factor of 2 times the average flow over an 18 hour period for estimating peak sewage flow. For this study we chose the Harmon's formula for ease of estimation.

Water Demand - The 2005 WSA indicates the average per capita day demand for 2004 was 490 lpcd. This value is higher than for other communities of similar size, however there is bleeding during winter months to keep lines from freezing. The addition of new lots is expected to add 0.5 L/s to the average day demand and 1.75 L/s peak hour for each 30 lot phase. Total new water demands for the entire proposed subdivision were estimated at 3.8 L/s average day and 13.3 L/s peak hour.

Sewage Generation - The resulting sanitary dry-weather flows for Phase 1 of the proposed subdivision were estimated at 0.5 L/s for an average day and 1.75 L/s peak hour for approximately 90 people (30 lots). The resulting sanitary dry-weather flows for the entire proposed subdivision were estimated at 3.8 L/s for an average day and 13.3 L/s peak hour for approximately 670 people.

3.2 POTABLE WATER DISTRIBUTION

Mains - It is recommended that a minimum 150 mm diameter PVC C900 or HDPE DR 17 water mains be used for potable water transmission. These are the most suitable material for its ease of installation, cost effectiveness and long life span. Larger 200 mm mains may be required (or recommended) to provide higher flows with lower pressure drops as in emergency/fire flow situations. This need will have to be further evaluated as development proceeds.

Fire hydrants are recommended to be provided along the mains at spacing no greater than 150 m and so that the distance to any building entrance is no greater than 75 m. Hydraulic model analysis is recommended as part of the detailed design in order to confirm the size of mains required through the new system.

Building Services - Recommend 19 mm copper or HDPE DR11, insulated and heat traced. Services to be extended to curb stop valve set on or near property line with adequate line and heat trace cable to reach future house. Main connection to include corporation stop and service saddle.

3.3 SANITARY SEWER NETWORK

Recommendations for the design of the sanitary system are as follows:

Mains – All sanitary mains are to be minimum 200 mm diameter. For pre-design it is assumed that mains should be buried to a minimum 3.0 m depth below frost line where possible. In reality, this may not always be possible although this depth is desirable. The Town indicates some lines are buried as shallow as 1.5 m and don’t give any significant troubles. If mains are installed less than minimum for frost protection they should be insulated Insulated lines will allow for reduced cover on services going below ditches while still maintaining frost protection. Minimum slope of 0.4% should be maintained with a maximum velocity not greater than 3.0 m/s. Manhole spacing should not exceed 120 m.

Building Services - Minimum size 100 mm. Minimum slope on service lines 2.0%. Under no circumstances will weeping tile, roof or surface drainage from buildings be permitted into the service connection of the sanitary sewer system. All gravity, sanitary sewer service pipes shall be PVC ASTM D3034 DR28 Municipal Service Pipe.

3.4 STORM WATER DRAINAGE

Storm water drainage systems are to meet municipal bylaws and provincial regulatory authority having jurisdiction. Drainage should generally follow the pattern identified in the Towns drainage plan, prepared by AE in 1997 (see Drawing 4412-100 in Appendix A).

Overall drainage will consist of a major and minor system. The major system consists of streets, detention facilities, parkland and other land which can convey run-off to prevent significant property damage. The minor system consists of manholes, catch basins and outfall structures. The minor system shall convey run-of from snow melt and rain fall without sustaining surface ponding or excessive surface flows. The major system should be sized for a 1 in 100 year event while the minor system shall be sized for a 1 in 5 year event.

3.5 ROADS

The design criteria for the new subdivision roads, matching the previous development, are recommended as follows:

- Main roads are to be classified as local collectors, with a minimum paved width of 3.5 m driving lanes with 2.8 m parking lanes;
- Road structure, based on an assumed subgrade California Bearing Ratio (CBR) of 3 to 5, to conform with PMEL report, summarized below in Table 3-1. Asphalt surface is assumed.
- With the residential nature of the subdivision the roads will be designed to accommodate “Light Truck/Passenger vehicle” wheel loading.
- Sidewalks are rolled face curb and gutter, monolithic, 1.5 m width, provided on both sides of all roads.

Table 3-1
Thickness Design for Access Roads

Pavement/Granular Structure	**Heavy Truck Traffic Wheel Loading (5,400 kg) (mm)**		**Light Truck/Passenger Vehicle Traffic Wheel Loading (1,830 kg) (mm)**	
Surfacing Gravel	-	50	**-**	50
Asphalt Concrete	100	-	**65**	-
Granular Base (Min CBR = 65)	150	150	**125**	150
Granular Sub-Base (Min. CBR = 20)	250	400	**175**	225
Prepared Subgrade	(150)	(150)	**(150)**	(150)
Geotextile	*	*	*****	*
Total Thickness	**500**	**600**	**365**	**425**

*Geotextile will be required where soft subgrade soils are encountered. High-strength (1,300 Newtons minimum), permeable, woven geotextile is recommended.

3.6 SHALLOW UTILITIES

Preliminary design does not include provision of shallow utilities (gas, power, cable, telephone) as these will be designed by the utility providers. For this report, we have assumed that all shallow utilities will be underground and lots will be serviced from the rear with right-of-ways provided.

4 Phasing of Development

4.1 DEVELOPMENT CONCEPT

A proposed development concept is shown in Figure 10 showing approx 230 lots with 19 of the total 36 ha developed. In addition we present two options for the first phase of development, according to the request for around 30 lots and a capital cost of under $1.5 M (costs to be discussed later in this report. The base assumption is that the development will occur in stages. Assuming a growth rate of 2% per year, which is aggressive based on historical records, the new development of 230 lots will provide in excess of 11 years development, or 20 lots per year.

4.2 PHASE ONE - OPTION 1

4.2.1 Overview

The location of Phase One - Option 1 is to the south west of Cook Crescent. Servicing Option 1 with water, sewer, storm and roads is reasonably well defined based on existing conditions. The portion of Option 1 closest to Bedford Drive is shaded to indicate the need to fill lots to provide adequate pipe cover for frost protection. There is some uncertainty to the number of lots that could be constructed in this area and how close to Bedford the lots would start.

4.2.2 Water

Option 1 could be connected to Loop No. 1 near the intersection of Bedford Drive and Louis Road. Loop No. 1 would be extended through the entire Phase 1 development area and would be looped to provide continuous circulation. Hydraulic model analysis shows that there should be adequate pressure available during average demand. Due to the overall length of Loop No. 1 and the location of Phase 1 tie-in to the system, there may be reduced pressure available during peak demand.

4.2.3 Sanitary

Sanitary flows for Phase One Option 1 would be collected in gravity sanitary sewer mains and flow to a manhole east of SPS No. 2, with eventual drainage to SPS No. 2. Due to topography and construction phasing, it is expected that all sanitary flows generated by the entire new subdivision will flow to the area near SPS No. 2.

Note: From the pump hours and flow analysis in Appendix D (summarized in table 4-1), based on 2008 flows, SPS No. 2 is <u>undersized to meet current</u> peak hour demands. With the additional load provided by the new subdivision, the capacity of SPS No. 2 will quickly become inadequate to handle peak flows. The 2002 UMA report noted that the wet well for SPS no. 2 is undersized for current flow and the Town reports that the 30 hp Morris pumps are original from 1975 construction.

Replacing the pumps in SPS No. 2 and expanding the wet well during phase one construction would address these existing concerns. The impact of growth and the possibility and timing of an upgrade to the SPS pumps should be examined further in detailed design.

From the pump hour analysis (see Appendix D), we concluded previously in 2.7 that the existing flow to SPS No. 2 consists of two components, the residential generated flow and an extraneous flow, summarized in the following Table.

Table 4-1
Estimates of Generated and Extraneous Flows

	Flow (L/s)		
Description	**SPS No. 8**	**SPS No. 7**	**SPS No. 2**
Average day dry weather flow	4.4	12.1	7.6
Max day dry weather flow	7.9	16.6	12.0
Peak hour dry weather flow (estimated at 2 x max day)	15.8	33.2	24.0
Extraneous flow	0 to 13.2	0 to 6.4	0 to 22.2
Total Required Capacity	**15.8 to 29.0**	**33.2 to 39.6**	**24.0 to 46.2**
Required Capacity estimated to service phase 1 – option 2 (32 lots)	up to 30.8 option 2 (32 lots)	up to 39.6	up to 47.6 option 1 (25 lots)
Reported Pump Capacity (2005 WSA)	**14.2**	**19.2**	**21.6**

To service the entire subdivision, SPS No. 2 will see an additional 13.3 L/s (peak hour). The increased peak hour flow exceeds the capacity of a single operating pump in the pump station.

To service only Option 1 of the new subdivision, SPS. No. 2 will see approximately an additional 1.75 L/s (peak hour).

The possibility does exist to provide sanitary sewer service to approximately 50 of the new lots by directing flow to SPS No. 8. These lots are located parallel to Studer Street along the north-west side of the subdivision. This is discussed further under Option 2.

4.2.4 Storm Drainage

Lots will be graded to direct water away from foundations. Drainage from lots will typically be directed on to the street and to ditches. The streets and ditches will convey the runoff toward the south of the subdivision, or to adjacent municipal reserve areas.

4.3 PHASE ONE - OPTION 2

4.3.1 Overview

Phase 1 Option 2 is the area parallel to Studer Street between Mowery Place and Thomson Crescent. This area was investigated as an alternate to Option1 above and is shown on Figure 10. Servicing of the alternate Phase 1 area with water and sewer service is feasible. A number of lots north of the entrance opposite Quandt Crescent are shown as shaded. These lots are “optional” depending on the available budget and could be left to a future phase if desired.

4.3.2 Water

The alternate Phase One Option 2 development could be supplied with water by Reservoir No. 3. The hydraulic model provided by AECOM shows distribution Loop No. 3 as a continuous circulating loop and that it experiences significant pressure drops near the end of the loop during peak demand due to its long length. This is not supported by the record drawings (which were inconclusive) or by the information from the Town. This needs to be confirmed.

Assuming that loop 3 is as shown in the model, in order to service the new subdivision from Reservoir No. 3, modifications to the supply and return lines to Reservoir No. 3 at Boardman Street would be required to split the flow into two sub-loops, one feeding the industrial area and the other feeding the residences, and both circulating back to Reservoir 3 via the existing return line. The modifications would involve the addition of a small amount of pipe and several valves to separate flow between the industrial area, residences along and to the west of Studer Street and the new development. No immediate pump upgrade would be required for service to alternate Phase 1 only.

There is the potential that these upgrades may not be required if the Town can verify that loop 3 already operates in the way we propose.

4.3.3 Sanitary

Development of Option 2 would allow diversion of sewage flows from approximately 50 lots to SPS No. 8. While this may ease demand on SPS No. 2, it will add a significant demand to SPS No. 8. Pump hour analysis shows that all three pumping stations servicing the study area may not be capable of meeting the demand for peak flows (see Table 4-1). Addition of flow through the Phase 1 Option 2 development may require upgrades to SPS No. 8 and SPS No. 7, whereas development of Phase 1 Option 1 near Cook Crescent would only require upgrades to SPS No. 2. Elimination of a portion of the infiltration may be satisfactory to provide the needed capacity at all three lift stations.

From the pump hour analysis in Appendix D, we estimated the existing flow to SPS No. 8 and SPS No. 7. These estimates are summarized in Table 4-1 as well.

Confirmation of the sanitary flows to SPS's 7,8 and 2 is required before proceeding with any future development or planning of future phases.

4.3.4 Storm Drainage

Lots will be graded to direct water away from foundations. Drainage from lots will typically be directed on to the street and to ditches. The streets and ditches will convey the runoff toward the south of the subdivision, or to adjacent municipal reserve areas.

4.4 COST ESTIMATE

A break down of the estimated costs for the subdivision development was prepared and can be viewed in Appendix E. Costs are summarized in the following Table. Costs are based on contractor unit costs for similar work in the La Ronge area.

Table 4-2
Phase One Cost Comparisons

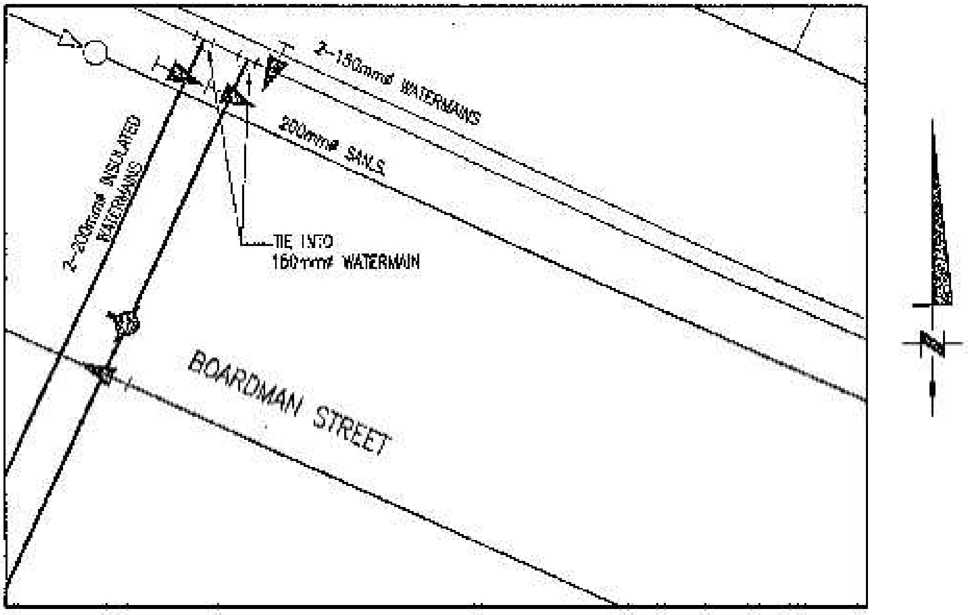
Description	Option 1 (near Cook Cres, approx 25 lots)	Option 2 (parallel to Studer St, approx 32 lots)
Estimated Cost	$1.67M	$1.60M
Estimated Cost / Lot	$67,000	$53,000

Note: No costs have been included for possible upgrades to the SPS's that may be required or the potential upgrades to the water loop no. 3.

4.5 FUTURE PHASES

The remainder of the new subdivision development could be supplied with potable water by Reservoir No. 3. This reservoir has capacity to meet projected water demands however hydraulic model analysis shows the length and small diameter of Loop No. 3 causes excessive pressure loss at peak demand to provide the required pressure throughout the entire system. The distribution system would require upgrading if Loop No. 3 is to supply water to the proposed subdivision. (see Figure 4-1a for existing configuration)

Figure 4-1a
Existing Pipe Configuration from 2004 Records

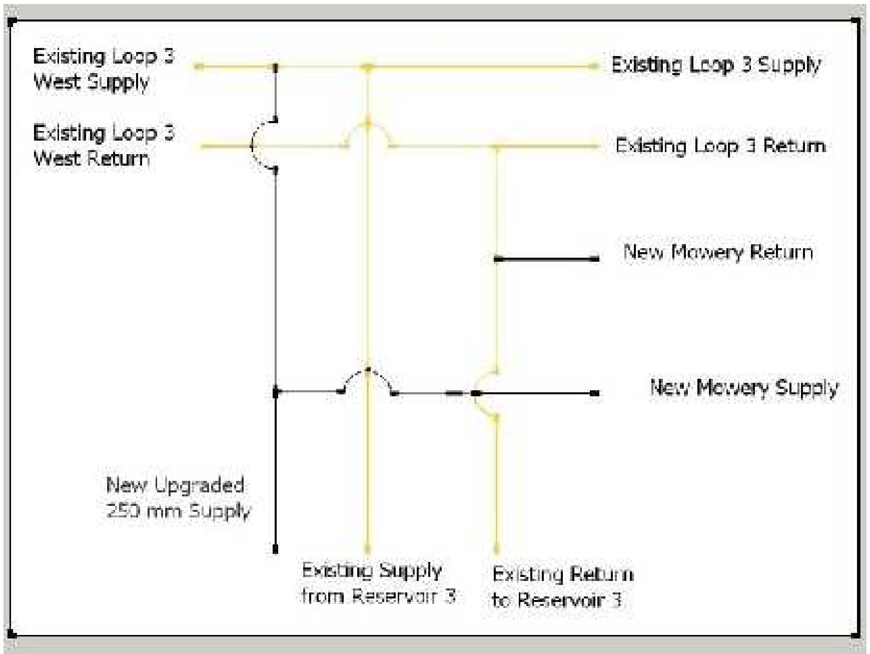


Proposed modifications to Loop No. 3 would ultimately split the loop into 3 loops with a common supply and return line to the reservoir. Initially, the addition of a small amount of pipe and several valves would be added to split the flow between the industrial area and the residential mains to the west of Studer Street. This would still be a circulating system but would proportion the flow between the two sides according to the demand.

To service the entire Mowery subdivision would require the addition of a third loop, supplied by an upgraded supply main (200mm or 250 mm) to parallel the existing main. Two new mains would be constructed from this point, running along Boardman and entering the subdivision creating a dedicated circulating loop. At this point, the connections to the existing Studer Street loop would be isolated and the mains installed in Phase One Option Two would become part of the new loop.

A sketch of the proposed piping modifications are shown in Figure 4-1b – Proposed Piping Modifications. No immediate pump upgrade would be required for Phase One if these modifications are implemented. Future Reservoir No. 3 pump upgrades would be required, however, when the new loop is created and the new subdivision is further developed.

Figure 4-1b
Proposed Piping Modifications



4.6 DEVELOPMENT LEVIES

The costs of lots for phase one (from 4.4) do not include costs of upgrades required for future full development, such as:

- Upgrades to water distribution loop no. 3
- Upgrades to SPS No. 2, 7, and/or 8
- Upgrades to the sewage treatment plant
- Upgrades to reservoirs and/or storage
- Drainage facilities
- Municipal and public buildings
- Fire protection

It is our understanding that under the Municipal Development Act, municipalities may elect to charge for these upgrades by applying and collecting Development Levies providing the amount of the levy is set by council based on some supporting study or professional advice. The typical value of these levies as reported to AE is in the range of $2,000 per lot to $4,000 per lot. The Town should consider adopting a Development Levy bylaw and collecting these fees on Phase One and future phases.

5 Conclusions

Based on the work completed in the preliminary design of the Mowery subdivision, we make the following conclusions:

- Municipal servicing of the 200+ lot proposed subdivision can be designed and constructed using conventional servicing methods compatible and consistent with previous development in the community;
- Topography and soil conditions are likely conducive to subdivision construction according to the conceptual development plan presented. Additional survey will be required, and additional bedrock investigation is recommended, for each phase of construction once the area is cleared.
- Significant uncertainty still exists regarding the sanitary network and pump stations.
- SPS No. 8 and SPS No. 2 could potentially be modified to handle the flows from a first phase of development. Additional investigation of existing flows and pump capacity is required.
- All three pump stations (SPS 2, 7 and 8) had significant infiltration events in 2008 according to the pump hour records analysis.
- The sewage treatment plant will likely accommodate all new flows from the full development.
- Water Loop 1 could be extended to phase one Option 1 and would likely provide adequate supply but an alternate supply would have to be provided for the full development.
- Water Loop 3 could be modified (or is presently adequate) to provide adequate supply to phase one, Option 2 and could also be upgraded to provide supply to the full development. The full development will require upgrading of pumps at Reservoir No. 3, twinning the supply main, and construction of a third branch to Loop No. 3.
- A surface drainage system for the new development could be developed to accommodate increased run-off from the Phase One development. Additional study is required to determine if the existing culverts on Bedford and the downstream drainage system can accommodate the increased flows from the full development. It is likely that the incorporation of ponding areas will be required to accommodate the flows from clearing and development.
- For both Option 1 and Option 2 of phase one, the cost is estimated at approximately $1.6 million, which is just over the Town budget of $1.4 million. Due to the topography, the number of lots in Option 1 is smaller which makes the cost per lot higher. Option 2 is the best option for servicing and the lower cost option. The number of lots can be adjusted somewhat to provide some flexibility in budgeting.

5.1 ISSUES TO BE RESOLVED

A number of issues require further investigation during detailed design of phase one including:

- Review of the concept plan by an urban planner.
- Additional survey of phase one once the site is cleared.
- Determination of flows to SPS No. 2, 7 and 8 and impact on future or immediate upgrading;

- Confirmation of storm drainage flows from on and off site that need to be conveyed through the proposed subdivision;
- Determine the actual remaining capacity of SPS No. 2, SPS No. 7 and SPS No. 8, and investigation of extraneous sanitary flows.
- Configuration of Loop 3 supply and correlation of the hydraulic model with hydrant flow testing and pressure readings.

6 Recommendations

It is recommended that the Town of La Ronge proceed with the development, given adequate fiscal capacity, with the following considerations:

Sanitary Sewer

For future planning it is recommended that a detailed analysis of the pumping capacity of SPS No. 2, SPS No. 7 and SPS No 8 be completed including authorization of a flow monitoring program to determine actual flows. This will help to plan future pump upgrades and ensure that the pumping stations are capable of handling peak flows. The ability to handle peak flows is important to prevent sewage back-ups in local residences. The sewage force mains should also be examined for available growth capacity.

Water Distribution

It is recommended that the piping modifications discussed in Section 4.3.2 be completed with the development of Phase 1. Reservoir No. 3 will supply water for future phases of development, however the distribution loop is long and experiences significant pressure drops near the end of the loop. The modifications will reduce the total length of distribution Loop No. 3 by splitting it into several smaller loops, resulting is lower pressure drops at the end of the loops. This ensures good service pressure to the new development and improved service to existing residences.

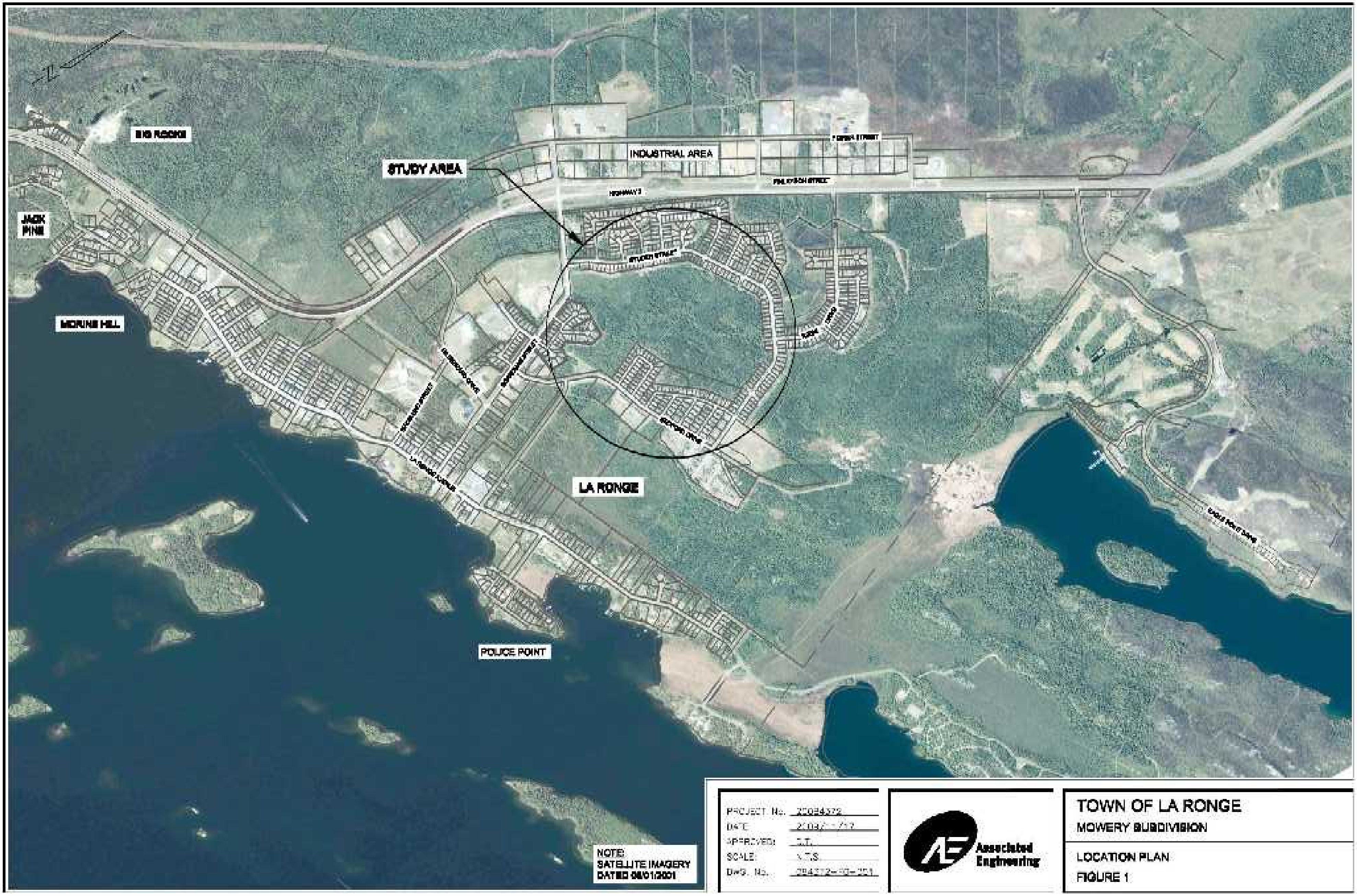
Preliminary hydraulic analysis indicates that future improvements to the distribution system may include a larger supply line from Reservoir No. 3 and upgraded pumps. Further hydraulic analysis is recommended to determine the exact type and timing of upgrades required.

Detailed Design

In addition, we recommend that La Ronge authorize detailed design of Phase One Option 2, including submission of the plan to Community Planning and to a legal surveyor for preparation of the proposed plan of subdivision.

A

Appendix A - Figures



NOTE:
SATELLITE IMAGERY
DATED 08/01/2001

PROJECT No. 20034372
DATE 2003/[illegible]/17
APPROVED: D.T.
SCALE: N.T.S.
DWG. No. 034372-00-201

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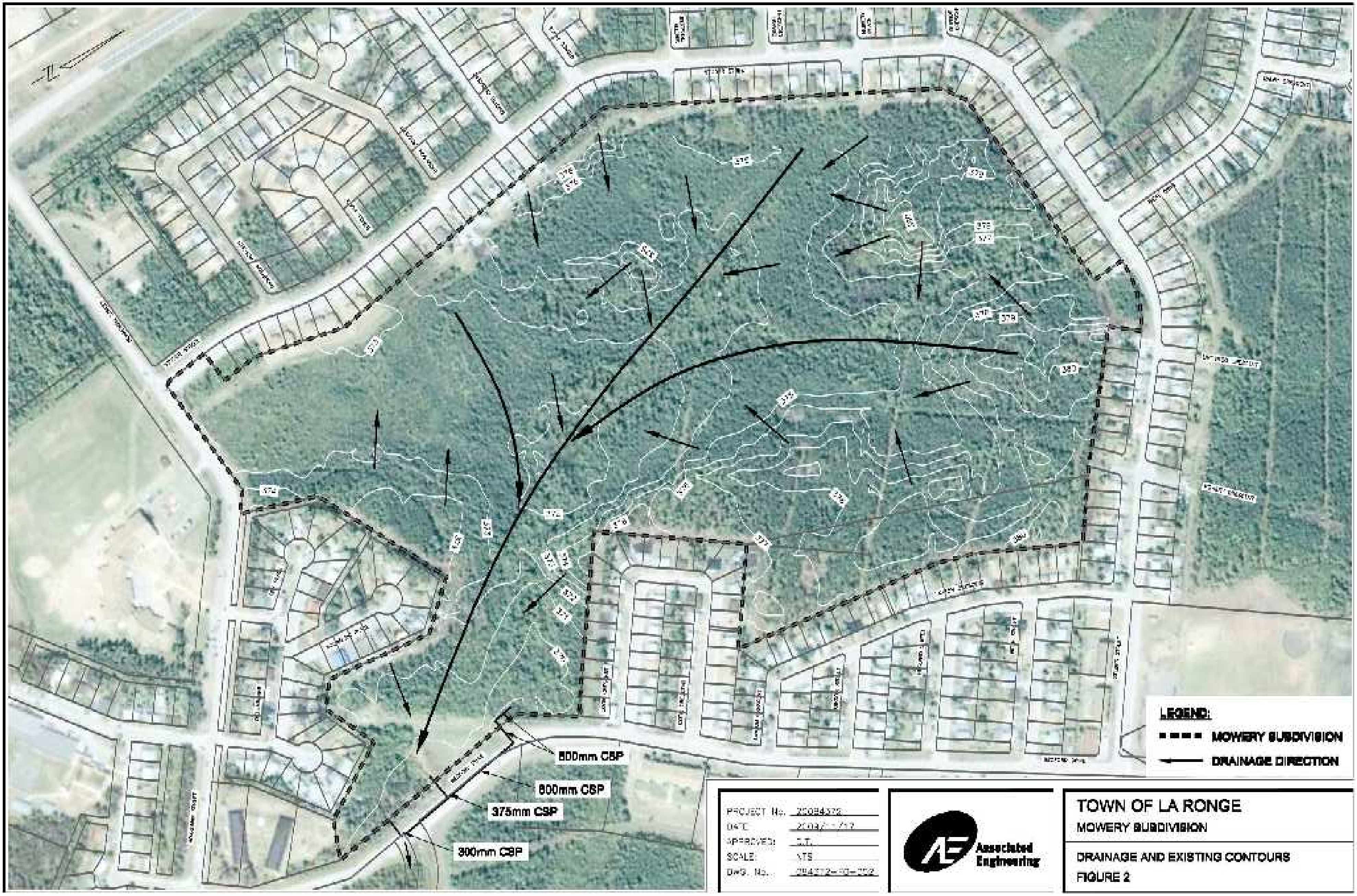
TOWN OF LA RONGE

MOWERY SUBDIVISION

LOCATION PLAN

FIGURE 1



PROJECT No. 20084372
DATE: 2008/11/12
APPROVED: D.T.
SCALE: NTS
DWG. No. 084372-00-202

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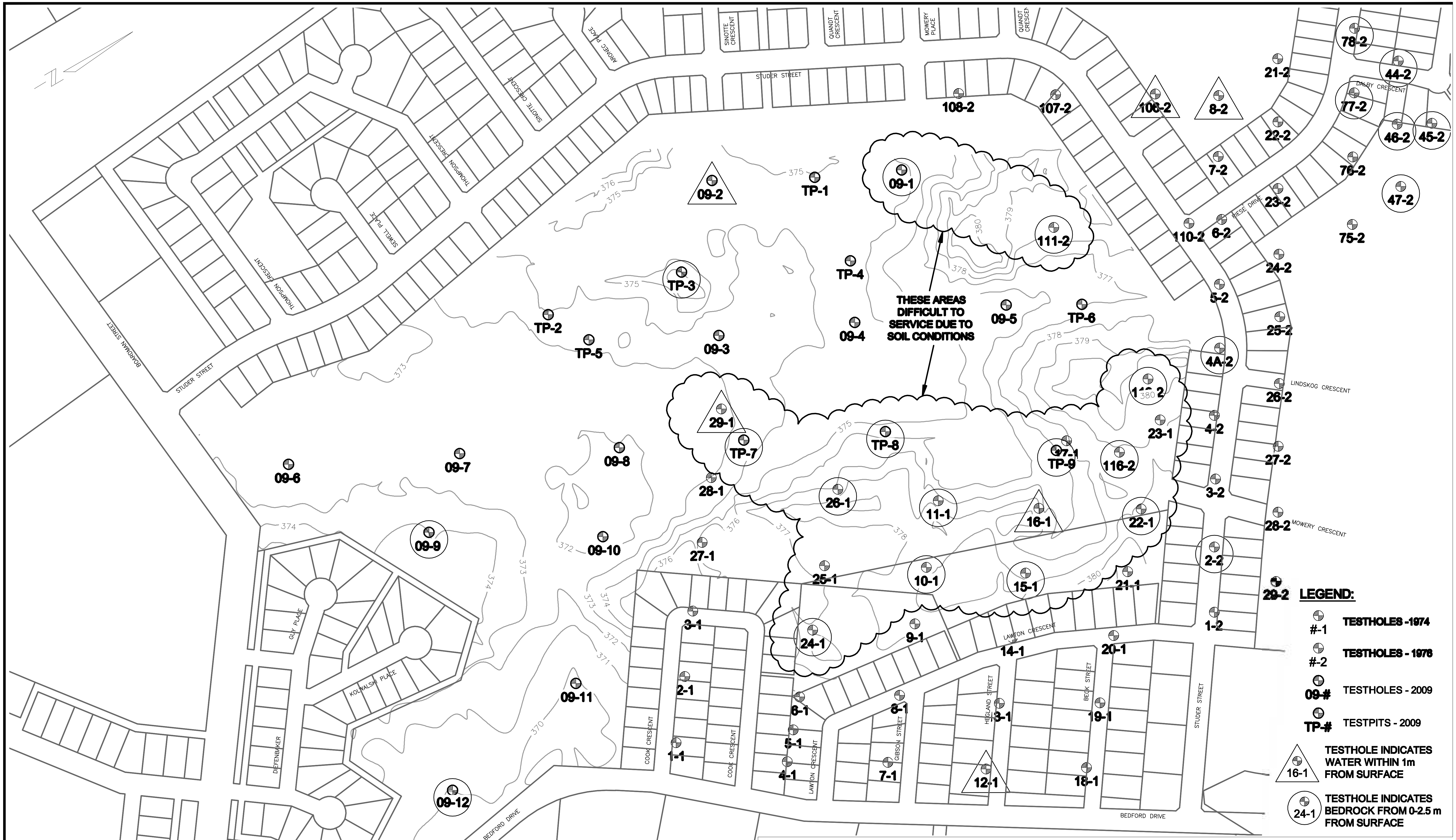
TOWN OF LA RONGE
MOWERY SUBDIVISION

DRAINAGE AND EXISTING CONTOURS
FIGURE 2



NOTE:
- FOR TESTHOLES, 1974 (#-1) DATA, PLEASE REFER TO FIGURE 4, 084372-FG-004.dwg
- FOR TESTHOLES, 1976 (#-2) DATA, PLEASE REFER TO FIGURE 5, 084372-FG-005.dwg
- FOR TESTHOLES AND TESTPITS, 2009 (09-#, TP-#) DATA, PLEASE REFER TO FIGURE 6, 084372-FG-006.dwg

PROJECT No. 20084372
DATE: 2009/11/17
APPROVED: D.T.
SCALE: NTS
DWG. No. 084372-FG-003

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TOWN OF LA RONGE
MOWERY SUBDIVISION

TESTHOLE AND TESTPIT LOCATIONS
FIGURE 3

TESTHOLES (AS BUILT OCTOBER 1974) - UNDERWOOD McLELLAN & ASSOCIATES LIMITED - VILLAGE OF LA RONGE

1-1 **TESTHOLE 1**

0' - 6"	ORGANIC
6" - 3'-6"	SAND (RED,BROWN)
3'-6" - 10'	CLAY
	NO ROCKS, DRY HOLE

2-1 **TESTHOLE 2**

0' - 6"	ORGANIC
6" - 10'	SANDY SILT, GREY
	SMALL STONES, DRY

3-1 **TESTHOLE 3**

0' - 6"	ORGANIC
6" - 10'	FINE SAND, SILT
	LARGE BOULDERS
	SMALL STONES, DRY

4-1 **TESTHOLE 4**

0' - 6"	ORGANIC
6" - 3'-6"	SAND
3'-6" - 12'	GRAVEL, MAX 2"
	DRY HOLE

5-1 **TESTHOLE 5**

0' - 12"	ORGANIC
12" - 4'	SAND
4' - 10'	FINE GREY SILT
	SMALL STONES, DRY

6-1 **TESTHOLE 6**

0' - 6"	ORGANIC
6" - 4'	SANDY SILT
4' - 10'	GRAVEL
	BOULDERS, DRY HOLE

7-1 **TESTHOLE 7**

0' - 12"	ORGANIC
12" - 4'-6"	SAND
4'-6" - 10'	SILT
	BOULDERS, DRY HOLE

8-1 **TESTHOLE 8**

0' - 12"	ORGANIC
12" - 3'-6"	SILT
3'-6" - 10'	GRAVEL, FINE
	DRY HOLE

9-1 **TESTHOLE 9**

0' - 8"	ORGANIC
8" - 2'-6"	COARSE GRAVEL
2'-6" - 10'	SILT
	SMALL STONES, DRY

10-1 **TESTHOLE 10**

0' - 6"	ORGANIC
6" - 2'	SILT
2'	BEDROCK
	SMALL ROCKS, DRY

11-1 **TESTHOLE 11**

0' - 6"	ORGANIC
6" - 1'-2"	SAND
1'-2" - 4'	SILT
4'	BEDROCK, DRY HOLE

12-1 **TESTHOLE 12**

0' - 8"	ORGANIC
8" - 10'	SILT
	WET HOLE

13-1 **TESTHOLE 13**

0' - 6"	ORGANIC
6" - 7'	SILT
7' - 9'	SAND
9' - 10'	GRAVEL, DRY HOLE

14-1 **TESTHOLE 14**

0' - 8"	ORGANIC
8" - 4'-4"	GRAVEL, MAX 2"
4'-4" - 6'-6"	GRAVEL
6'-6" - 10'	SILT
	BOULDERS, DRY

15-1 **TESTHOLE 15**

0' - 8"	ORGANIC
8" - 2'	GRAVEL
2' - 3'-3"	SILT
3'-3"	BEDROCK, DRY HOLE

16-1 **TESTHOLE 16**

0' - 6"	ORGANIC
6" - 10'	SILT, FINE SAND
	STONES, WET HOLE

17-1 **TESTHOLE 17**

0'-9"	ORGANIC
9" - 8'-6"	SAND, SILT
8'-6"	BEDROCK
	ROCKS, DRY HOLE

18-1 **TESTHOLE 18**

0' - 6"	ORGANIC
6" - 3'-8"	FINE SAND
3'-8" - 10'	SILT, DRY HOLE

19-1 **TESTHOLE 19**

0' - 5"	ORGANIC
5" - 2'-4"	FINE SAND
2'-4" - 3'-5"	GRAVEL
3'-5" - 10'	SILT, DRY HOLE

20-1 **TESTHOLE 20**

0' - 5"	ORGANIC
5" - 4'-4"	SILT
4'-4" - 6'-3"	GRAVEL, MAX 2"
6'-3" - 9'-5"	SILT
9'-5" - 10'	GRAVEL, MAX 2"
	BOULDERS, DRY HOLE

21-1 **TESTHOLE 21**

0' - 4"	ORGANIC
4" - 10'	SAND, GRAVEL
	SILT, BOULDERS
10'	BEDROCK, DRY HOLE

22-1 **TESTHOLE 22**

0' - 6"	ORGANIC
6" - 3'-6"	SILT
3'-6" - 7'-6"	GRAVEL, MAX 2"
7'-6"	BEDROCK
	BOULDERS, DRY HOLE

23-1 **TESTHOLE 23**

0' - 5"	ORGANIC
5" - 3'	SILT
3' - 9'-5"	SAND
9'-5"	BEDROCK
	BOULDERS, DRY HOLE

24-1 **TESTHOLE 24**

0' - 3"	ORGANIC
3" - 2'	SILT
2'	BEDROCK
	BOULDERS, DRY HOLE

25-1 **TESTHOLE 25**

0' - 5"	ORGANIC
5" - 4'	SAND
4' - 9'-5"	SILT
9'-5"	BEDROCK
	BOULDERS, DRY HOLE

26-1 **TESTHOLE 26**

0' - 4"	ORGANIC
4" - 2'-6"	SAND, GRAVEL
2'-6"	BEDROCK, DRY HOLE

27-1 **TESTHOLE 27**

0' - 4"	ORGANIC
4" - 7'	SILT
7' - 10'	GRAVEL, MAX 3/4"
	BOULDERS, DRY HOLE

28-1 **TESTHOLE 28**

0' - 3"	ORGANIC
3" - 6'	SILT
6' - 10'	GRAVEL, MAX 3/4"
	BOULDERS, WET HOLE

29-1 **TESTHOLE 29**

0' - 1'-2"	ORGANIC
1'-2" - 10'	SILT
	BOULDERS, WET HOLE

NOTE:
FOR EXISTING TESTHOLES LOCATIONS, PLEASE REFER TO FIGURE 3, 084372-FG-003.dwg

PROJECT No. 20084372
DATE: 2009/11/17
APPROVED: D.T.
SCALE: N/A
DWG. No. 084372-FG-004

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TOWN OF LA RONGE
MOWERY SUBDIVISION

TESTHOLES DATA, 1974 (#-1)
FIGURE 4

TESTHOLES (MAY 1976) - UNDERWOOD McLELLAN & ASSOCIATES LIMITED - VILLAGE OF LA RONGE

1-2 **TESTHOLE 1**

0' - 5"	ORGANIC
5" - 6'	GRAVEL
6' - 11'	SILT & GRAVEL, HARD SOME STONES 1", DRY HOLE

2-2 **TESTHOLE 2**

0' - 3"	ORGANIC
3" - 2'	GRAVEL
2' - 5'	SILT & STONES (1/2-1")
5'	BEDROCK, DRY HOLE

3-2 **TESTHOLE 3**

0' - 4"	ORGANIC
4" - 2'	GRAVEL
2' - 4'	SILT & GRAVEL
4' - 10'	SILT, STONES, BOULDERS DRY HOLE

4-2 **TESTHOLE 4**

0' - 4"	ORGANIC
4" - 1'	GRAVEL
1' - 8'	SILTY GRAVEL, STONES
8'	BEDROCK, DRY HOLE

4A-2 **TESTHOLE 4A**

0' - 4"	ORGANIC
4"	BEDROCK

5-2 **TESTHOLE 5**

0' - 4"	ORGANIC
4" - 9'	SANDY SILT, STONES
9'	BEDROCK, SPONGY SURFACE, WET HOLE

6-2 **TESTHOLE 6**

0' - 4"	ORGANIC
4" - 10'	SANDY SILT, STONES DRY HOLE

7-2 **TESTHOLE 7**

0' - 4"	ORGANIC
4" - 10'	SILTY SAND, STONES BOULDERS, DRY HOLE

8-2 **TESTHOLE 8**

0' - 12"	ORGANIC, WET
12" - 2'-6"	ORGANIC, FROZEN
2'-6" - 6'	SILT, NOT FROZEN
6' - 8'-6"	SILTY SAND, WET HOLE WATER IN FROM 2.5' LEVEL

21-2 **TESTHOLE 21**

0' - 2'	ORGANIC
2' - 8'	HEAVY SILT
8' - 10'	ROCKS, SANDY WET GRAVEL

22-2 **TESTHOLE 22**

0' - 5"	ORGANIC
5" - 4'	SAND & SILT
4' - 10'	SILT WET AT BOTTOM SOME ROCKS

23-2 **TESTHOLE 23**

0' - 4"	ORGANIC
4" - 10'	SANDY SILT, ROCKS DRY HOLE

24-2 **TESTHOLE 24**

0' - 5"	ORGANIC
5" - 2'	ORANGE SILTY TILL SMALL ROCKS
2' - 10'	SAND, GRAVEL, ROCKS DRY HOLE

26-2 **TESTHOLE 26**

0' - 6"	ORGANIC
6" - 10'	SANDY SILT, SOME ROCKS LITTLE WATER AT BOTTOM 2'

27-2 **TESTHOLE 27**

0' - 3"	ORGANIC
3" - 10'	FINE SANDY GRAVEL DRY HOLE

28-2 **TESTHOLE 28**

0' - 4"	ORGANIC
4" - 1'	SAND
1' - 10'	SANDY GRAVEL BOULDERS, DRY HOLE

29-2 **TESTHOLE 29**

0' - 4"	ORGANIC
4" - 1'-6"	GRAVEL
1'-6" - 10'	SANDY GRAVEL BOULDERS, DRY HOLE

44-2 **TESTHOLE 44**

0' - 6"	ORGANIC
6" - 5'	SANDY SILT, STONES
5'	BEDROCK, DRY HOLE

45-2 **TESTHOLE 45**

0' - 4"	ORGANIC
4" - 7'	SANDY SILT, STONES
7'	BEDROCK, DRY HOLE

46-2 **TESTHOLE 46**

0' - 6"	ORGANIC
6" - 1'	GRAVEL
1' - 7'	SILTY GRAVEL, STONES
7'	BEDROCK, DRY HOLE

47-2 **TESTHOLE 47**

0' - 6"	ORGANIC
6" - 1'	SILTY GRAVEL
1'	BEDROCK, DRY HOLE

75-2 **TESTHOLE 75**

0' - 4"	ORGANIC
4" - 8'	SILTY SAND, STONES
8'	BEDROCK, DRY HOLE

76-2 **TESTHOLE 76**

0' - 6"	ORGANIC
6" - 9'	SANDY SILT, STONES
9'	BEDROCK, DRY HOLE

77-2 **TESTHOLE 77**

0' - 5"	ORGANIC
5" - 4'	SANDY SILT, ROCKS
4'	BEDROCK, DRY HOLE

78-2 **TESTHOLE 78**

0' - 4"	ORGANIC
4" - 5'	SANDY SILT, STONES
5'	BEDROCK, DRY HOLE

106-2 **TESTHOLE 106**

0' - 3'	ORGANIC, WATER FORMING FROM SIDES
3' - 10'	WET SILTY SAND WET HOLE

107-2 **TESTHOLE 107**

0' - 8"	ORGANIC
8" - 10'	SILTY SAND, STONES ROCKS, DRY HOLE

108-2 **TESTHOLE 108**

0' - 5"	ORGANIC
5" - 10'	SANDY SILT DRY HOLE

110-2 **TESTHOLE 110**

0' - 8"	ORGANIC
8" - 10'	SANDY SILT, WET HOLE WATER IN AT 8' MARK

111-2 **TESTHOLE 111**

0' - 5"	ORGANIC
5" - 7'	SANDY SILT, ROCKS BOULDERS
7'	BEDROCK, DRY HOLE

113-2 **TESTHOLE 113**

0' - 5"	ORGANIC
5" - 18"	COARSE RED SAND
18"	BEDROCK, DRY HOLE

116-2 **TESTHOLE 116**

0' - 5"	ORGANIC
5" - 3'	SANDY SILT, STONES
3'	BEDROCK, DRY HOLE

NOTE:
FOR EXISTING TESTHOLES LOCATIONS, PLEASE REFER TO FIGURE 3, 084372-FG-003.dwg

PROJECT No. 20084372
DATE: 2009/11/16
APPROVED: D.T.
SCALE: N/A
DWG. No. 084372-FG-005

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TOWN OF LA RONGE
MOWERY SUBDIVISION

TESTHOLES DATA, 1976 (#-2)
FIGURE 5

TESTHOLES (JAN 22, 2009) - P. MACHIBRODA ENGINEERING LTD. - VILLAGE OF LA RONGE

⊕ 09-1

TESTHOLE 1 - ELEV. 375.09

0m - 0.25m	- PEAT, ORGANIC, BLACK, ROOTLETS, FROZEN.
0.25m - 2.3m	- GLACIAL TILL, SAND, SILTY, SOME GRAVEL, TRACE CLAY, COMPACT, WELL GRADED, FINE TO COARSE GRAINED, MOIST, BROWN, COBBLES AND BOULDERS.
	- WET, SEEPAGE, SLOUGHING BELOW 900mm.
	- AUGER REFUSAL ON ASSUMED BEDROCK @ 2.3m.

⊕ 09-2

TESTHOLE 2 - ELEV. 374.56

0m - 0.2m	- PEAT, ORGANIC, BLACK, ROOTLETS, FROZEN.
0.2m - 3.5m	- SILT, SOME SAND, TRACE CLAY, FIRM, NON TO LOW PLASTIC, MOIST, OLIVE BROWN.
	- WET, SEEPAGE, SOUGHING BELOW 1.3m.
	- TRACE SAND, COBBLES/BOULDERS BELOW 1.8m.
	- GLACIAL TILL, SAND, SILTY, SOME GRAVEL, TRACE CLAY, COMPACT, WELL GRADED, FINE TO COARSE GRAINED, WET, BROWN, SEEPAGE, SLOUGHING, COBBLES AND BOULDERS.

⊕ 09-3

TESTHOLE 3 - ELEV. 373.37

0m - 0.4m	- PEAT, ORGANIC, BLACK, ROOTLETS, FROZEN.
0.4m - 4.5m	- GLACIAL TILL, SAND, SILTY, SOME GRAVEL, TRACE CLAY, COMPACT, WELL GRADED, FINE TO COARSE GRAINED, MOIST, BROWN, OXIDE STAINED, COBBLES AND BOULDERS.
	- WET, SEEPAGE, SOUGHING BELOW 1.8m.
	- AUGER REFUSAL ON ASSUMED BEDROCK @ 4.5m.

⊕ 09-4

TESTHOLE 4 - ELEV. 374.76

0m - 4.8m	- GLACIAL TILL, SAND, SILTY, SOME GRAVEL, TRACE CLAY, COMPACT, WELL GRADED, FINE TO COARSE GRAINED, MOIST, BROWN, OXIDE STAINED, COBBLES AND BOULDERS.
	- FROZEN TO 400mm.
	- WET, SEEPAGE, SOUGHING BELOW 3.1m.
	- AUGER REFUSAL ON ASSUMED BEDROCK @ 4.8m.

⊕ 09-5

TESTHOLE 5 - ELEV. 375.46

0m - 0.2m	- PEAT, ORGANIC, BLACK, ROOTLETS, FROZEN.
0.2m - 0.8m	- CLAY, SOME SILT, STIFF, HIGHLY PLASTIC, MOIST, BROWN.
	- FROZEN TO 400mm.
0.8m - 1.8m	- SILT, TRACE CLAY, TRACE SAND, TRACE GRAVEL, FIRM, LOW PLASTIC, MOIST, BROWN.
1.8m - 2.8m	- GLACIAL TILL, SAND, SILTY, SOME GRAVEL, TRACE CLAY, COMPACT, WELL GRADED, FINE TO COARSE GRAINED, MOIST, BROWN, OXIDE STAINED.
	- COBBLES AND BOULDERS BELOW 2.2m.
	- AUGER REFUSAL ON ASSUMED BEDROCK @ 2.8m.

⊕ 09-6

TESTHOLE 6 - ELEV. 372.05

0m - 0.4m	- PEAT, ORGANIC, BLACK, ROOTLETS, FROZEN.
0.4m - 2.4m	- CLAY, SOME SILT, STIFF, HIGHLY PLASTIC, MOIST, OLIVE BROWN.
	- SILTY, FIRM, LOW PLASTIC, OXIDE STAINED BELOW 1.6m.
2.4m - 4.0m	- GLACIAL TILL, SAND, SILTY, SOME GRAVEL, TRACE CLAY, COMPACT, WELL GRADED, FINE TO COARSE GRAINED, WET, BROWN, OXIDE STAINED, SEEPAGE, SLOUGHING.
	- COBBLES AND BOULDERS BELOW 3.2m.
	- AUGER REFUSAL ON ASSUMED BEDROCK @ 4.0m.

⊕ 09-7

TESTHOLE 7 - ELEV. 372.27

0m - 0.2m	- PEAT, ORGANIC, BLACK, ROOTLETS, FROZEN.
0.2m - 1.4m	- CLAY, SOME SILT, STIFF, HIGHLY PLASTIC, MOIST, BROWN.
1.4m - 4.7m	- GLACIAL TILL, SAND, SILTY, SOME GRAVEL, TRACE CLAY, COMPACT, WELL GRADED, FINE TO COARSE GRAINED, WET, BROWN, OXIDE STAINED, SEEPAGE, SLOUGHING, COBBLES AND BOULDERS.
	- AUGER REFUSAL ON ASSUMED BEDROCK @ 4.7m.

⊕ 09-8

TESTHOLE 8 - ELEV. 371.96

0m - 0.3m	- PEAT, ORGANIC, BLACK, ROOTLETS, FROZEN.
0.3m - 4.5m	- SILT, SOME SAND, TRACE CLAY, FIRM, NON TO LOW PLASTIC, MOIST, OLIVE BROWN.
	- WET, SEEPAGE, SLOUGHING BELOW 1.8m.
	- COBBLES AND BOULDERS BELOW 4.0m.
	- AUGER REFUSAL ON ASSUMED BEDROCK @ 4.5m.

⊕ 09-9

TESTHOLE 9 - ELEV. 373.95

0m - 1.5m	- CLAY, SOME SILT, STIFF, HIGHLY PLASTIC, MOIST, BROWN.
	- FROZEN TO 500mm.
	- COBBLES AND BOULDERS BELOW 1.4m.
	- AUGER REFUSAL ON ASSUMED BEDROCK @ 1.5m.

⊕ 09-10

TESTHOLE 10 - ELEV. 371.82

0m - 0.2m	- PEAT, ORGANIC, BLACK, ROOTLETS, FROZEN.
0.2m - 0.5m	- SAND, SILTY, LOOSE TO COMPACT, POORLY GRADED, FINE GRAINED, WET, OLIVE BROWN, SEEPAGE, SLOUGHING.
0.5m - 1.7m	- CLAY SOME SILT, FIRM TO STIFF, MEDIUM TO HIGHLY PLASTIC, MOIST, BROWN.
	- SILT, SOME CLAY, FIRM, NON TO LOW PLASTIC, WET, OLIVE BROWN, SEEPAGE, SLOUGHING.
1.7m - 4.6m	- GLACIAL TILL, SAND, SILTY, SOME GRAVEL, TRACE CLAY, COMPACT, WELL GRADED, FINE TO COARSE GRAINED, WET, BROWN, SEEPAGE, SLOUGHING, COBBLES AND BOULDERS.
	- AUGER REFUSAL ON ASSUMED BEDROCK @ 4.6m.

⊕ 09-11

TESTHOLE 11 - ELEV. 369.93

0m - 0.1m	- PEAT, ORGANIC, BLACK, ROOTLETS, FROZEN.
0.1m - 1.8m	- CLAY, SOME SILT, STIFF, HIGHLY PLASTIC, MOIST, BROWN.
	- FROZEN TO 500mm.
1.8m +	- GLACIAL TILL, SAND, SILTY, SOME GRAVEL, TRACE CLAY, COMPACT, WELL GRADED, FINE TO COARSE GRAINED, WET, BROWN, OXIDE STAINED, SEEPAGE, SLOUGHING.
	- COBBLES AND BOULDERS BELOW 2.5m.
	- DENSE, MOIST, MOTTLED BROWN AND GREY BELOW 3.4m.

⊕ 09-12

TESTHOLE 12 - ELEV. 370.19

0m - 0.3m	- SAND, SOME GRAVEL, SOME SILT, DENSE, WELL GRADED, FINE TO COARSE GRAINED, MOIST, BROWN, FROZEN.
	- AUGER REFUSAL ON ASSUMED BEDROCK @ 300mm.

TESTPITS (JAN 22, 2009) - P. MACHIBRODA ENGINEERING LTD. - VILLAGE OF LA RONGE - DRAWING A3

TP-1 - GROUND ELEV: 374.47m, TOTAL DEPTH: 3.0m, 2 BIG BOULDERS, 2m TO WATER.
TP-2 - GROUND ELEV: 373.51m, TOTAL DEPTH: 5.0m, WATER DEPTH: 4.5m, COBBLE AND BOULDERS.
TP-3 - GROUND ELEV: 375.60m, TOTAL DEPTH: 2.2m, COBBLE AND BOULDERS, BEDROCK FOUND @ 2.2m.
TP-4 - GROUND ELEV: 374.87m, TOTAL DEPTH: 5.0m, COBBLE AND BOULDERS.
TP-5 - GROUND ELEV: 373.28m, TOTAL DEPTH: 5.0m, WATER DEPTH: 4.5m, COBBLE AND BOULDERS, 3m TO BOULDERS.
TP-7 - GROUND ELEV: 373.79m, 1.3m TO BEDROCK.
TP-8 - GROUND ELEV: 374.98m, TOTAL DEPTH: 0.0m, SURFACE ROCK.
TP-9 - GROUND ELEV: 377.47m, 2.5m TO BEDROCK.
SIDE HILL TEST PIT - 2m TO BEDROCK.

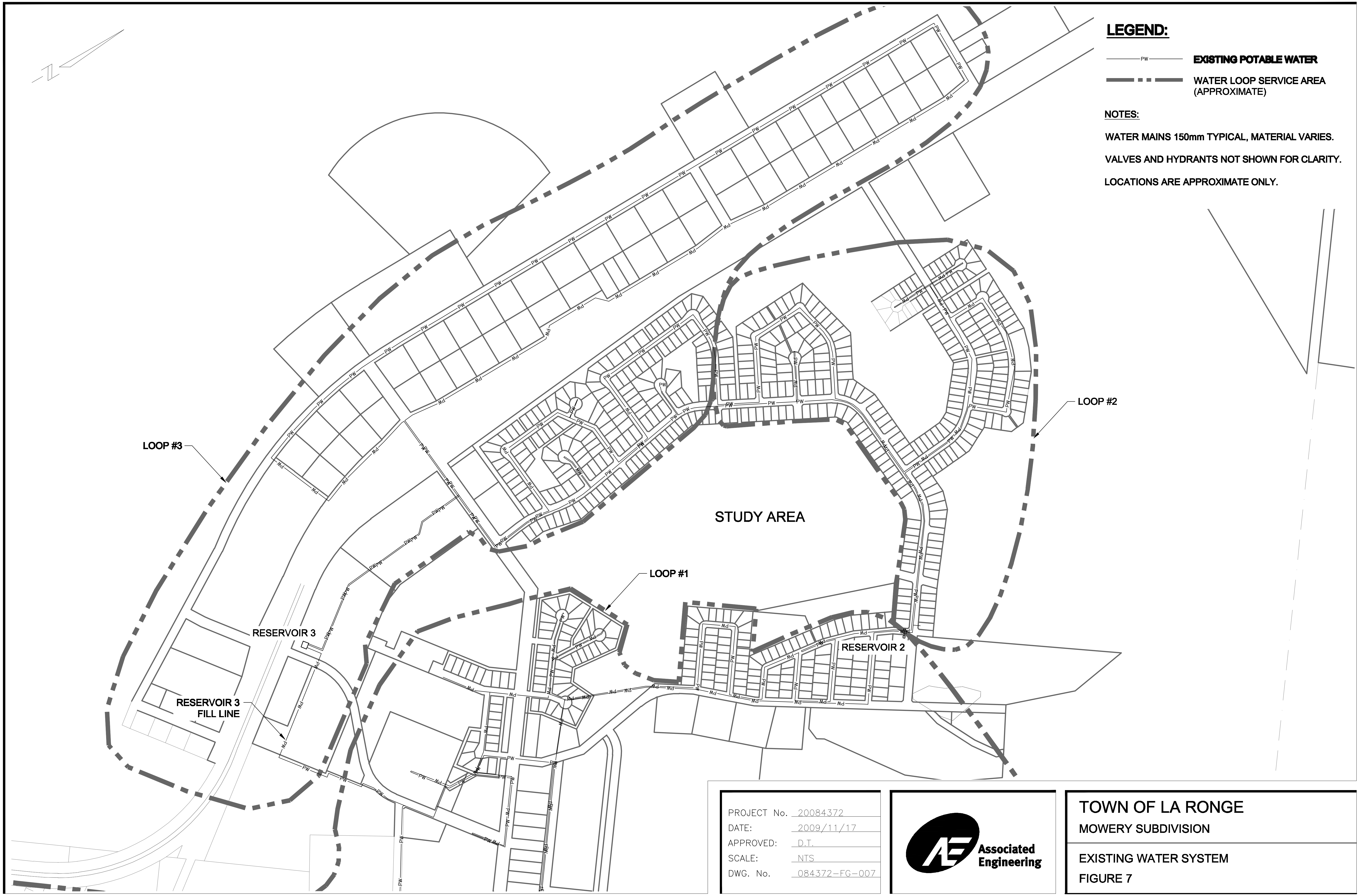
NOTE:
FOR EXISTING TESTHOLES AND TESTPIT LOCATIONS,
PLEASE REFER TO FIGURE 3, 084372-FG-003.dwg

PROJECT No. 20084372
DATE: 2009/11/17
APPROVED: D.T.
SCALE: N/A
DWG. No. 084372–FG–006

Associated Engineering

TOWN OF LA RONGE
MOWERY SUBDIVISION

TESTHOLES/TESTPITS DATA, 2009 (09-#, TP-#)
FIGURE 6



NOTES:

WATER MAINS 150mm TYPICAL, MATERIAL VARIES.

VALVES AND HYDRANTS NOT SHOWN FOR CLARITY.

LOCATIONS ARE APPROXIMATE ONLY.

PROJECT No.	20084372
DATE:	2009/11/17
APPROVED:	D.T.
SCALE:	NTS
DWG. No.	084372-FG-007

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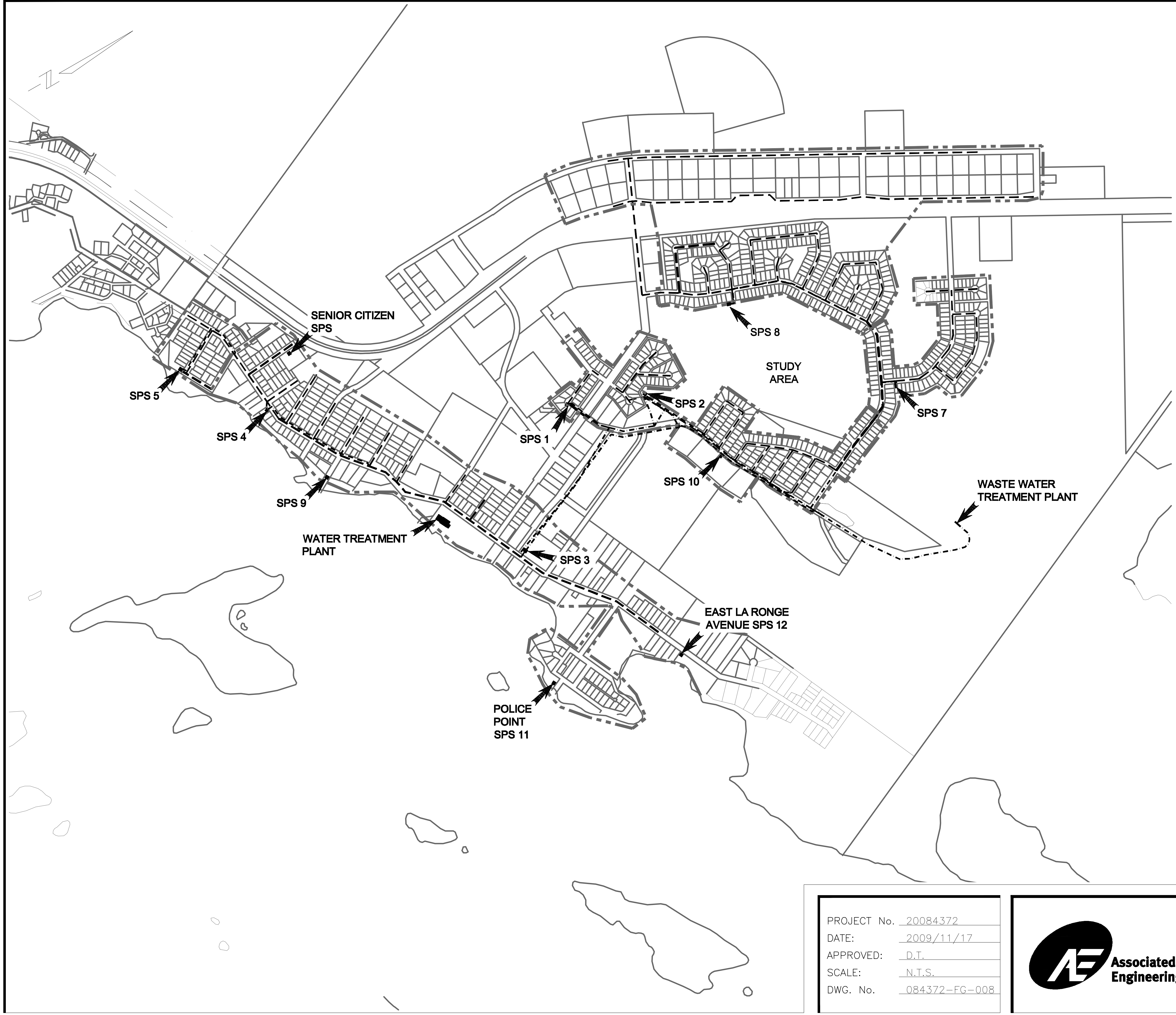
TOWN OF LA RONGE
MOWERY SUBDIVISION

EXISTING WATER SYSTEM
FIGURE 7



LEGEND:

- - - - - - EXISTING SEWER
- · - · - · EXISTING FORCEMAIN
— · — · — CONTRIBUTING AREA BOUNDARY

SEWAGE PUMPING STATION SUMMARY

SPS #1: EST. PUMP CAPACITY = 18.2 L/s @ 9.1 m TDH
PUMP SIZE = 15 hp
EXG. WET WELL SIZE = 850 L

SPS #2: EST. PUMP CAPACITY = 21.6 L/s @ 15.2 m TDH
PUMP SIZE = 30 hp
EXG. WET WELL SIZE = 3000 L

SPS #3: EST. PUMP CAPACITY = 35.0 L/s @ ? m TDH
PUMP SIZE = 30 hp
EXG. WET WELL SIZE = 4600 L

SPS #4: EST. PUMP CAPACITY = 22.0 L/s @ ? m TDH
PUMP SIZE = 7.5 hp
EXG. WET WELL SIZE = 3700 L

SPS #5: EST. PUMP CAPACITY = 5.7 L/s @ ? m TDH
PUMP SIZE = 5 hp
EXG. WET WELL SIZE = 600 L

SPS #7: EST. PUMP CAPACITY = 19.2 L/s @ 7.3 m TDH
PUMP SIZE = 7.5 hp
EXG. WET WELL SIZE = 2700 L

SPS #8: EST. PUMP CAPACITY = 14.2 L/s @ 11.0 m TDH
PUMP SIZE = 7.5 hp
EXG. WET WELL SIZE = 2000 L

SPS #9: EST. PUMP CAPACITY = 5.7 L/s @ 12.2 m TDH
PUMP SIZE = 5 ho
EXG. WET WELL SIZE = 1600 L

SPS #10: EST. PUMP CAPACITY = 25.6 L/s @ 5.2 m TDH
PUMP SIZE = 7.5 hp
EXG. WET WELL SIZE = 700 L

NOTES:

1) ABOVE DATA FROM TOWN OF LA RONGE SEWAGE PUMPING STATION ANALYSIS, UMA 2002 AND 2005 WSA (AECOM).

2) NO DATA AVAILABLE FOR SENIOR CITIZEN SPS, POLICE POINT SPS 11 AND EAST LA RONGE AVENUE SPS 12. 2005 WATERWORKS SYSTEM ASSESSMENT INDICATED PUMPS OPERATE LESS THEN 1 HOUR PER DAY.

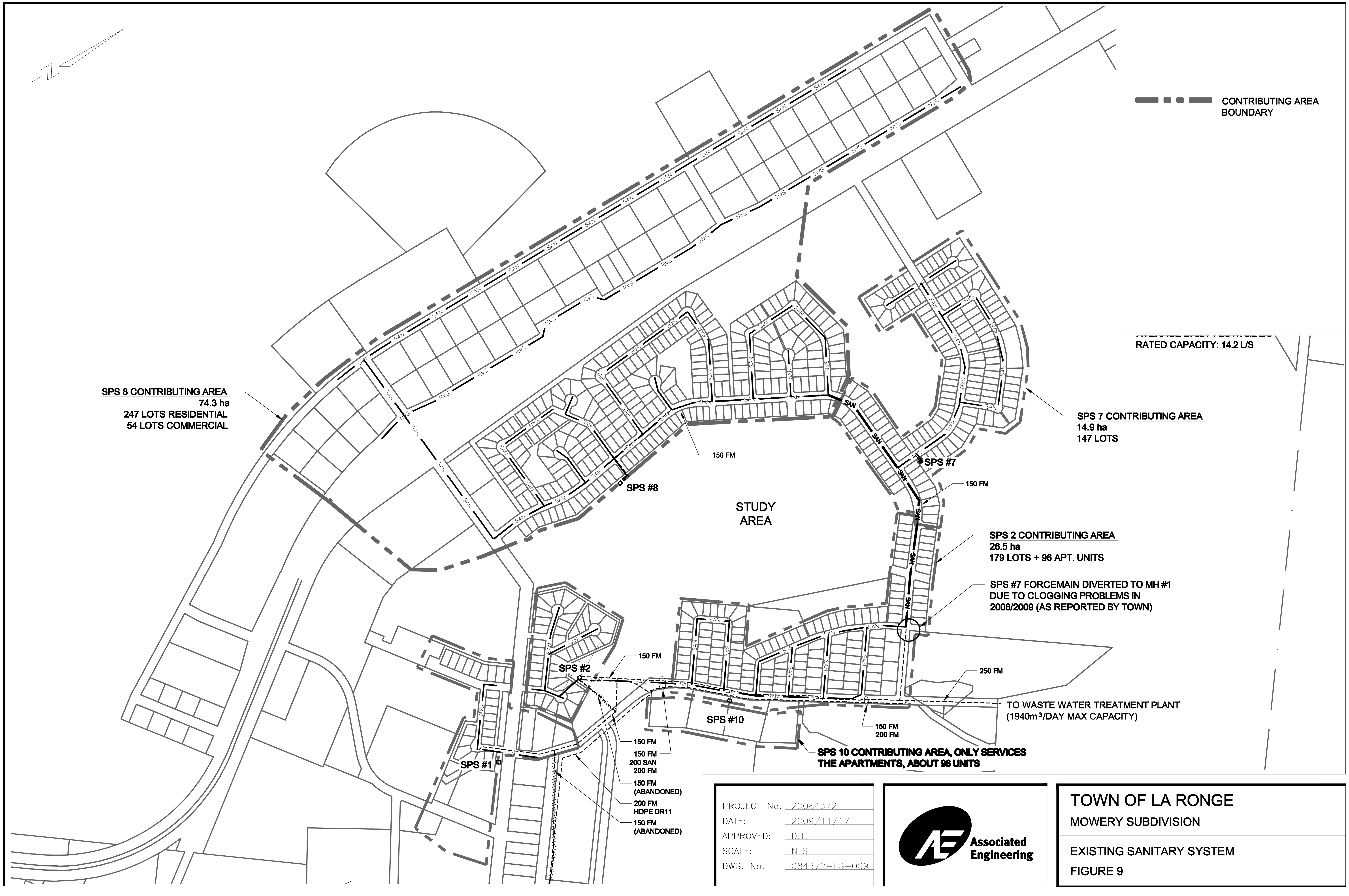
3) PUMP CAPACITIES TAKEN FROM 2005 WSA (AECOM).

PROJECT No. 20084372
DATE: 2009/11/17
APPROVED: D.T.
SCALE: N.T.S.
DWG. No. 084372-FG-008

Associated Engineering

TOWN OF LA RONGE
MOWERY SUBDIVISION

EXISTING TOWN SEWAGE SYSTEM
FIGURE 8



PROJECT No. 20084372
DATE: 2009/11/17
APPROVED: D.T.
SCALE: NTS
DWG. No. 084372-FG-009

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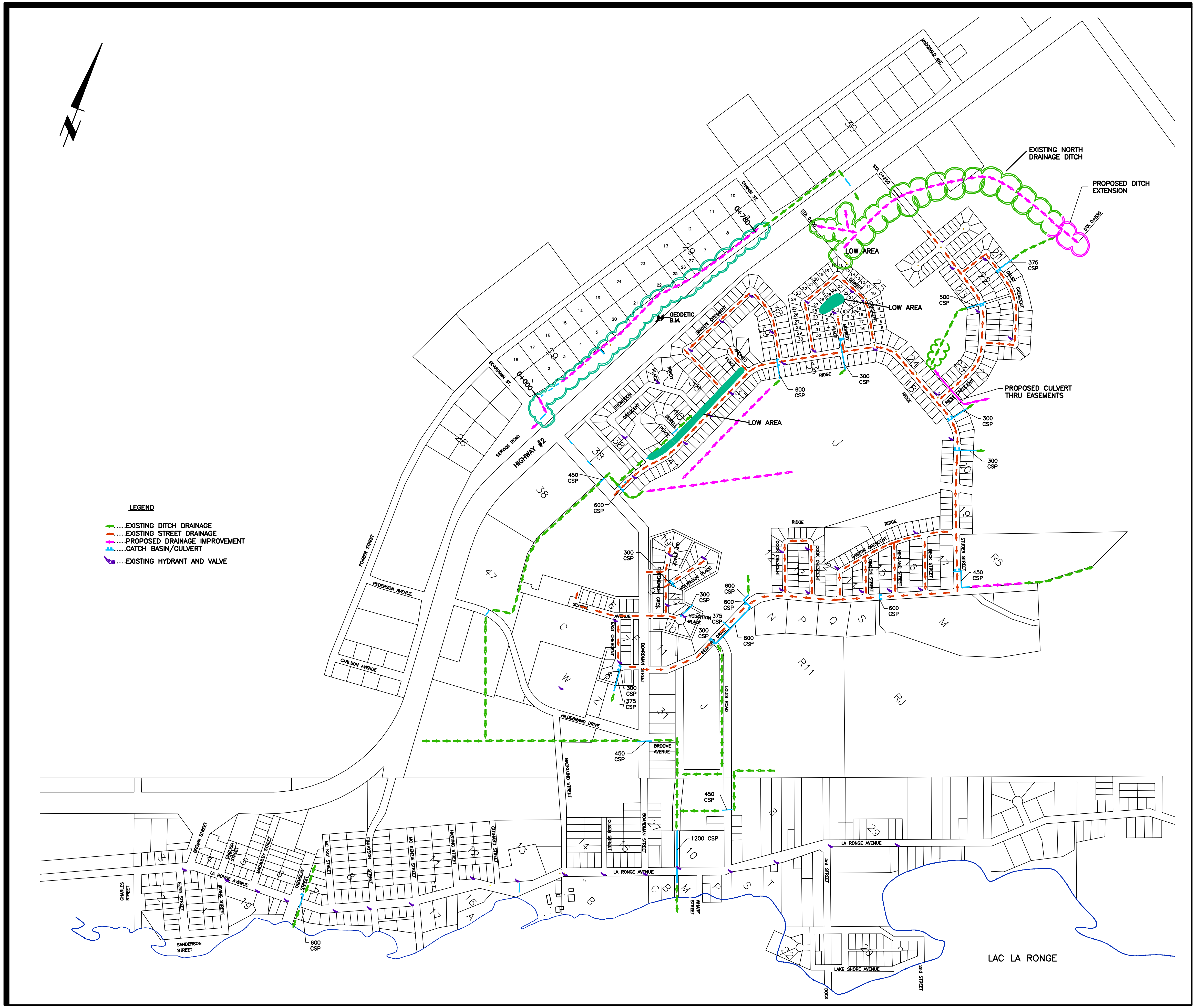
TOWN OF LA RONGE
MOWERY SUBDIVISION

EXISTING SANITARY SYSTEM
FIGURE 9



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0	97/09/15	A.O.M.	B.C.	ISSUED FOR REVIEW

REVISIONS

PROJECT NO.	974412
SCALE	1:5000
DRAWN	S. TAYLOR
DESIGNED	G. EDWARDS
CHECKED	K. CH'NG
APPROVED	A.O. MUNRO
APPROVED	A.O. MUNRO
DATE	SEPTEMBER 1997

ASSOCIATED
ENGINEERING

TOWN OF LARONGE

DRAINAGE STUDY

DRAINAGE PLAN

DRAWING NUMBER	REV. NO.	SHEET
4412-100	0	1/2

LAST REVISED DATE: 27-10-97
ACAD FILE: 44121100.DWG
PLOT SCALE: 1=1
HARD COPY LOCATION: AE STOON
SOFT COPY LOCATION: AE STOON

B

Appendix B - Geotechnical

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GEOTECHNICAL INVESTIGATION
PROPOSED MOWERY RESIDENTIAL SUBDIVISION
LA RONGE, SASKATCHEWAN

PMEL FILE NO. S08-6783
FEBRUARY 9, 2009

PREPARED FOR:

NORTHERN REVENUE SHARING TRUST ACCOUNT
LA RONGE SUBDIVISION
C/O ASSOCIATED ENGINEERING
1 – 2225 NORTHRIDGE DRIVE
SASKATOON, SASKATCHEWAN
S7L 6X6

ATTENTION: MR. DEREK TRISCHUK, P. ENG.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

LIST OF DRAWINGS

LIST OF DRAWINGS (continued)

1.0 INTRODUCTION

The following report has been prepared on the subsurface soil conditions existing at the site of the proposed Mowery Residential Subdivision to be constructed in La Ronge, Saskatchewan. The subject site is bound by Boardman Street to the west, Studer Street to the north and east, and, Bedford Street to the south.

Authorization to proceed with this investigation was provided on December 10, 2008. The terms of reference for this investigation were presented in P. Machibroda Engineering Ltd. (PMEL) Proposal No. 1117-5249, dated November 21, 2008. The test hole drilling and soil sampling were conducted on January 22, 2009.

2.0 FIELD INVESTIGATION

Twelve test holes, located as shown on the Site Plan, Drawing No. S08-6783-1, were dry drilled using our truck-mounted, continuous flight, solid stem auger drill rig. The test holes were 150 mm in diameter and extended to depths of 0.3 to 6.0 metres below the existing ground surface. In addition to the Test Holes, nine Test Pits were excavated as part of the site investigation to supplement the Test Hole information and to verify the presence and depth of bedrock.

Test hole drill logs were compiled during test drilling to record the soil stratification, the groundwater conditions, the position of unstable sloughing soils and the depths at which cobblestones, boulders and bedrock were encountered.

Disturbed samples of auger cuttings, collected during test drilling, were sealed in plastic bags to minimize moisture loss. The soil samples were taken to our laboratory for analysis.

Piezometers (slotted, 50 mm diameter PVC pipe) were installed in Test Hole No. 09-1, 09-2, 09-3, 09-4, 09-5, 09-6, 09-7, 09-8, 09-10 and 09-11 for groundwater monitoring purposes.

3.0 FIELD DRILL LOGS AND TEST PITS

The field drill logs recorded during test drilling have been shown plotted on Drawing Nos. S06-6783-2 to 13, inclusive.

A summary of the soil conditions encountered at the Test Pit locations (as compiled by PMEL and Associated Engineering) has been presented in Table I.

TABLE I. TEST PIT SOIL CONDITIONS

Test Pit No.	Ground Surface Elevation (metres)	Test Pit Depth (Metres)	General Soil Conditions	*Groundwater Elevation After Excavation (metres)	Bedrock Elevation (metres)
**09-1	NA	3.0	Glacial Till, Cobbles/Boulders	2 m below G.L.	NE
09-2	373.5	5.0	Glacial Till, Cobbles/Boulders	369.0	NE
09-3	375.6	2.2	Glacial Till, Cobbles/Boulders	DRY	373.4
09-4	375.0	5.0	Glacial Till, Cobbles/Boulders	DRY	NE
09-5	373.3	5.0	Glacial Till, Cobbles/Boulders	368.8	NE
09-6	377.2	0.3	NA	DRY	376.9
**09-7	NA	1.3	NA	DRY	1.3 m below G.L.
09-8	377.2	1.0	NA	DRY	376.2
**09-9	NA	2.5	NA	DRY	2.5 m below G.L.

NA – Not Available NE - Not Encountered
*Groundwater elevation recorded immediately after excavation – higher water levels are anticipated.
**Information provided by excavation contractor.

The plan location and ground surface elevation at the Test Hole and Test Pit locations was provided by Associated Engineering.

3.1 Soil Profile

The general soil profile consisted of organic peat overlying variable overburden deposits of silt, clay, sand and glacial till, followed by bedrock. Bedrock outcroppings were encountered surficially at many locations within the subject site. The thickness of the overburden deposits is controlled by the underlying bedrock topography, which is anticipated to be highly variable.

3.2 Groundwater Conditions, Sloughing

Groundwater seepage and sloughing conditions were encountered during test drilling. The depths at which groundwater seepage and sloughing conditions were encountered have been shown on Drawing Nos. S08-6783-2 to 13, inclusive.

A summary of the groundwater levels recorded in the piezometers installed during this investigation has been presented in Table II.

TABLE II. RECORDED GROUNDWATER LEVELS

Test Hole No.	Piezometer Rim Elevation (metres)	Ground Surface Elevation (metres)	* Recorded Groundwater Elevation (metres)	
			January 23, 2009	January 30, 2009
09-1	376.2	375.1	373.5	374.1
09-2	375.4	374.6	N/R	373.0
09-3	374.4	373.4	370.4	372.3
09-4	375.9	374.8	371.5	372.2
09-5	376.6	375.5	374.1	374.2
09-6	373.1	372.1	370.7	370.8
09-7	373.2	372.3	370.5	371.2
09-8	373.1	372.0	370.9	371.4
09-10	372.8	371.8	367.9	371.1
09-11	370.9	369.9	368.9	368.9

*Higher and potentially perched static water levels should be expected during or following spring snowmelt and periods of precipitation. The groundwater table is strongly influenced by the underlying bedrock topography. Localized areas of perched (on bedrock) groundwater conditions are anticipated.

3.3 Cobblestones and Boulders, Bedrock

Cobblestones and/or boulders were encountered during test drilling. The depths at which cobblestones and/or boulders were encountered have been shown on Drawing Nos. S08-6783-2 to 13, inclusive. The glacial till consisted of a heterogeneous mixture of gravel, sand, silt and clay-sized particles. The glacial till strata also contained sorted deposits of the above particle sizes. In addition to the sorted deposits, a random distribution of larger particle sizes in the cobblestone range (60 to 200 mm) and boulder-sized range (larger than 200 mm) should be expected at the subject site.

With the exception of Test Hole Nos. 09-2 and 09-11 (soil to a depth of at least 6 metres), auger refusal was encountered on assumed bedrock at depths of 0.2 to 4.8 metres in all Test Holes. Bedrock outcroppings were encountered surficially at many locations within the subject site.

4.0 LABORATORY ANALYSIS

The soil classification and index tests performed during this investigation consisted of a visual classification of the soil, water contents, Atterberg limits and grain size distribution analysis.

The results of the soil classification and index tests conducted on representative samples of soil have been plotted on the drill logs alongside the corresponding depths at which the samples were recovered, as shown on Drawing Nos. S08-6783-2 to 13, inclusive.

The results of the grain size distribution analyses have been plotted on Drawing Nos. S08-6783-14 to 17, inclusive.

5.0 DESIGN RECOMMENDATIONS

Based on the foregoing outline of soil test results, the following foundation considerations and design recommendations have been presented.

5.1 Design Considerations

It is understood that the proposed Subdivision will include residences, new utility installations, roadways, landscape areas and surface drainage features.

The subsurface soil conditions consisted of organic peat overlying variable deposits of silt, clay, sand and glacial till, followed by bedrock. Groundwater seepage and sloughing conditions were encountered during test drilling. The groundwater level at the time of our investigation ranged from about 0.5 to 2.5 metres below existing ground surface, and was situated within 1 to 1.5 metres of ground surface in most test holes. Cobblestones and/or boulders were encountered during test drilling. With the exception of Test Hole Nos. 09-2 and 09-11 (soil to a depth of at least 6 metres), auger refusal was encountered on assumed bedrock at depths of 0.2 to 4.8 metres in all Test Holes. Bedrock outcroppings were encountered surficially at many locations within the subject site. The subgrade soils are frost susceptible and the average depth of frost penetration for the La Ronge area is approximately 2.5 metres.

It is understood that extensive site development will be undertaken (i.e., extensive bush clearing, site stripping and leveling, construction of ditches and roadways, etc.) prior to the construction of any residences or other structures at this site. It is recommended that site development should be designed to lower the groundwater level as much as practical and/or elevate the finished ground surface elevation.

The subgrade soil conditions at the subject site were variable. As such, footing foundations could potentially be based on silt, clay, sand, glacial till, bedrock or a combination of these soils. Footing foundations based on uniform, non-expansive subgrade soils should perform satisfactorily. Footing foundations based on variable soils/bedrock or highly plastic clay soils would be subject to greater potential differential movements. The subgrade soils are frost susceptible and the groundwater table at the site is relatively high at most locations. To minimize the potential for frost heaving and associated building distress, all reasonable measures must be taken to ensure that frost is not allowed to penetrate beneath the footings prior to, during or following construction. It is recommended that footings should be extended to bedrock at all locations where the depth to bedrock is near the design footing elevation and an adequate clearance above the groundwater table can be achieved.

Floor slab (basement) elevations should be set as high as possible above the groundwater table to minimize the potential for water seepage into the basements (a minimum clearance of 500 mm above the groundwater table is recommended). Placement of additional fill may be required to provide adequate frost protection while maintaining adequate clearance above the groundwater table. Perimeter and below slab drainage systems will be required to minimize potential for water infiltration.

A deep foundation system consisting of steel pipe and helix screw piles could perform satisfactorily at locations where adequate thickness of overburden exists. Some construction difficulties are anticipated during the installation of screw piles due to the presence of cobblestones and/or boulders.

As an alternate to a footing or pile foundation system, an at-grade foundation consisting of a reinforced, perimeter edge thickened concrete raft foundation could be utilized as a foundation support.

Recommendations have been prepared for site preparation; excavations and de-watering; standard strip or spread footings; steel pipe and helix screw piles; perimeter edge thickened concrete raft foundation; floor slabs; foundation walls; foundation concrete and subdivision roads and parking structures.

5.2 Site Preparation

All peat, organics, loose fill and other deleterious materials should be removed from the construction areas. The surface of the subgrade should be levelled and compacted to the following minimum density requirements.

Building Areas	-	98 percent of standard Proctor density at optimum moisture content;
Roadway Areas	-	96 percent of standard Proctor density at optimum moisture content;
Landscape Areas	-	90 percent of standard Proctor density at optimum moisture content.

Subgrade fill, if required, should preferably consist of granular material or locally available sand or glacial till soils. The fill should be placed in thin lifts (maximum 150 mm loose) and compacted to 98 percent of standard Proctor density at optimum moisture content. The subgrade fill should be approved by the Geotechnical Consultant prior to placement. The site should be graded to ensure positive site drainage away from all structures.

Perched groundwater conditions and soft/wet soil conditions are anticipated at some locations within the subject site. The use of high-strength, woven geotextile (minimum grab tensile strength of 1,300 N) is recommended where soft/wet soil conditions are encountered to provide soil stabilization and separation. Depending on the design subgrade elevations, over-excavation and replacement with granular fill may be required. It is anticipated that the soft/wet near-surface soils will be easily disturbed. Due to the anticipated poor trafficability, Gradall or track/backhoe equipment will be required at these locations. The geotextile should be placed over the levelled subgrade prior to placing granular fill. It is recommended that the first lift of fill should consist of free-draining granular drainage aggregate placed over the geotextile by end-dump and spread methods in a single lift (i.e., 200 to 300 mm recommended). The initial lift should be lightly compacted with static compaction equipment to minimize disturbance of the underlying soil.

5.3 Excavations and Dewatering

The groundwater table was situated at a depth of about 0.5 to 2.5 meters below existing ground surface on January 30, 2009, and was situated within 1 to 1.5 metres of ground surface in most test holes. Construction difficulties related to groundwater seepage and sloughing are expected below the groundwater table.

It is anticipated that the proposed excavations at this site will be shallow and completed with unbraced, sloped side walls. Blasting techniques will be required where large boulders or shallow bedrock conditions are encountered. The long-term stability of the excavation walls will be affected by wetting and drying of the exposed excavation walls, the depth of excavation, the length of time that the excavation remains open and the consistency and structure (degree of fracturing, slickensiding, etc.) of the subgrade soils. The recommended minimum sideslopes for excavations at this site have been presented in Table III.

TABLE III. RECOMMENDED MINIMUM EXCAVATION SIDESLOPES

Soil Description	*Minimum Safe Sideslope	
	Horizontal	Vertical
Moist Soil	2	1
Saturated Soil	4	1

* Slope flattening will be required where groundwater seepage and sloughing conditions are encountered. Dewatering will be required below the groundwater table.

Groundwater seepage and precipitation runoff should be collected in a drainage system at the base of the excavation (i.e., drainage ditches/interceptors, sump pits). The drainage system should drain positively to a collection sump(s) equipped with a sump pump(s).

5.4 Standard Strip or Spread Footings

The subgrade soil conditions at the subject site were variable. As such, footing foundations could potentially be based on silt, clay, sand, glacial till, bedrock or a combination of these soils. Footing foundations based on uniform, non-expansive subgrade soils should perform satisfactorily. Footing foundations based on variable soils/bedrock or highly plastic clay soils would be subject to greater potential differential movements.

The following minimum recommendations should be incorporated into the design of a footing foundation.

1. For a continually heated dwelling with basement, the footings should be founded on naturally deposited, undisturbed soil at a minimum depth of 1.2 metres below finished ground surface. Footings not protected with an interior heat source and 1.2 metres of soil cover should be based below the average depth of frost penetration (i.e., 2.5 metres) or protected with strategically placed rigid polystyrene insulation. In this case, a continuous layer of rigid polystyrene insulation should be placed over the exterior face of the foundation wall, extending vertically a minimum of 300 mm above grade and laterally a minimum distance of 1.8 metres away from the foundation. The insulation should be a minimum of 75 mm in thickness and should be positively sloped away from the foundation to promote drainage. The insulation should be placed a minimum of 300 mm below finished grade.

2. For footings on bedrock, the bedrock at the footing elevation should be levelled as well as practical. Controlled blasting techniques may be required to provide a level footing surface. Footings on bedrock that are not protected with an interior heat source and a minimum of 1.2 metres of soil cover should be doweled into the bedrock at regular intervals to provide additional frost uplift resistance and/or protected with insulation, as described above.

3. Footings based on naturally deposited, undisturbed soil may be designed to exert an allowable bearing pressure of 75 kPa. The footing excavations should be hand-cleaned to remove all loose, disturbed soil, and, to expose naturally deposited, undisturbed soil. If the subgrade soil at the design footing elevation consists of soft, wet soil, the width of the footing should be increased by fifty (50) percent.

4. Footings on bedrock may be designed to exert an allowable bearing pressure of 500 kPa. The footing excavations should be hand-cleaned to remove all loose, disturbed soil and fractured bedrock.

5. Where silt or sand subgrade soils are encountered at the design footing depth, it is recommended that a mud slab should be placed as soon as practical after cleaning to minimize the potential for disturbance of the silt/sand subgrade soils.

6. A minimum strip footing width of 450 mm is recommended. A minimum dimension of 1,000 mm is recommended for square and rectangular footings.

7. If the subgrade soil/bedrock is disturbed during excavation below the design depth, then the disturbed soil should be removed to an undisturbed, level surface. Fill, required to raise the subgrade elevation to the underside of the footings, should be concrete.

8. Footings should not be constructed on desiccated, frozen or wet subgrade soil. Frost should not be allowed to penetrate beneath the footings prior to, during or after construction. In unheated areas, where potential damage due to frost penetration and upheaval could occur, footings should be based below the depth of frost penetration. Alternately, adequate insulation could be utilized to prevent frost penetration below the footings. In this case, the Geotechnical Consultant should review the proposed insulation details.

9. The finished grade should be landscaped to provide for positive site drainage away from the Residence.

5.5 Steel Pipe and Helix Screw Piles

Steel pipe and helix screw piles could perform satisfactorily at locations where adequate thickness of overburden exists. Some construction difficulties are anticipated during the installation of screw piles due to the presence of cobblestones and/or boulders.

Steel pipe screw piles are installed by rotating a steel pipe, equipped with one or more helix flightings, into the ground. Steel pipe screw piles may be designed on the basis of skin friction and end bearing capacity. The skin friction capacity of the subgrade soils is summarized below.

TABLE IV. SKIN FRICTION BEARING PRESSURES (SCREW PILES)

Zone (metres)	Allowable Skin Friction Capacity (kPa)
0 to 2	0
Below 2	15

The screw piles should be extended to a minimum depth of 4 metres. For determination of skin friction capacity, the effective shaft length may be taken as the depth of embedment of the pile shaft (to the top of the helix) minus the diameter of the helix. For piles with multiple helixes, the skin friction capacity between the helixes can be calculated on the basis of the projected surface area of the soil column between the helixes and the skin friction values presented in Table IV.

The allowable end bearing pressure for screw piles designed with end bearing contribution has been presented below.

TABLE V. END BEARING PRESSURE (SCREW PILES)

Depth (metres)	Allowable End Bearing Pressure (kPa)
Below 4	250

To minimize the potential for soil disturbance adjacent to the pile shaft (which could cause a reduction or loss of skin friction capacity), screw piles must be stopped immediately upon contact with the bedrock surface. A representative of the Geotechnical Consultant must inspect and document the installation of each steel pipe screw pile on a continuous basis. If pile installation documentation is not conducted, then the screw piles should be designed on the basis of end bearing capacity only.

The end bearing capacity of steel screw piles may be calculated utilizing the cross-sectional area of the bottom-most helix (including central shaft). The helical plate shall be normal to the central shaft (within 3 degrees) over its entire length. Multiple helixes (if required) should be spaced at 3 helix diameters apart. Continuous monitoring of the installation torque should be undertaken during installation to determine whether the screw pile has been damaged during installation and to monitor the consistency of the subsurface soils. Screw piles should be designed on the basis of appropriate Geotechnical Engineering principals pertaining to helical pile foundations.

5.6 Perimeter Edge Thickened Concrete Raft Foundation

The proposed Residences may be supported at ground surface on a perimeter edge thickened concrete raft foundation. The following minimum recommendations should be incorporated into the design of a continually heated, reinforced, perimeter edge thickened concrete raft foundation. A typical perimeter edge thickened concrete raft foundation is shown on Figure No. 1.

1. Remove all peat, loose fill, organics and deleterious material from the proposed Building footprint. Level and compact the uppermost 150 mm of the subgrade surface to 98 percent of standard Proctor density at optimum moisture content.

2. Place a woven geotextile with a minimum grab tensile strength of 1,300 Newtons over the surface of the prepared subgrade (generally as shown on Figure No. 1). The geotextile should be installed in accordance with the manufacturer's specifications.

3. Place a minimum of 300 mm of compacted, crushed granular base course material over the placed geotextile (600 mm where highly plastic clay subgrade soils are encountered). All granular fill should be placed and compacted in thin lifts (150 mm loose, maximum) to a minimum of 100 percent of standard Proctor density at optimum moisture content. The granular fill should extend laterally away from the edge of the raft a distance at least equal to the fill thickness.

4. A raft bearing on the structural granular fill may be designed to exert an allowable bearing pressure of 75 kPa.

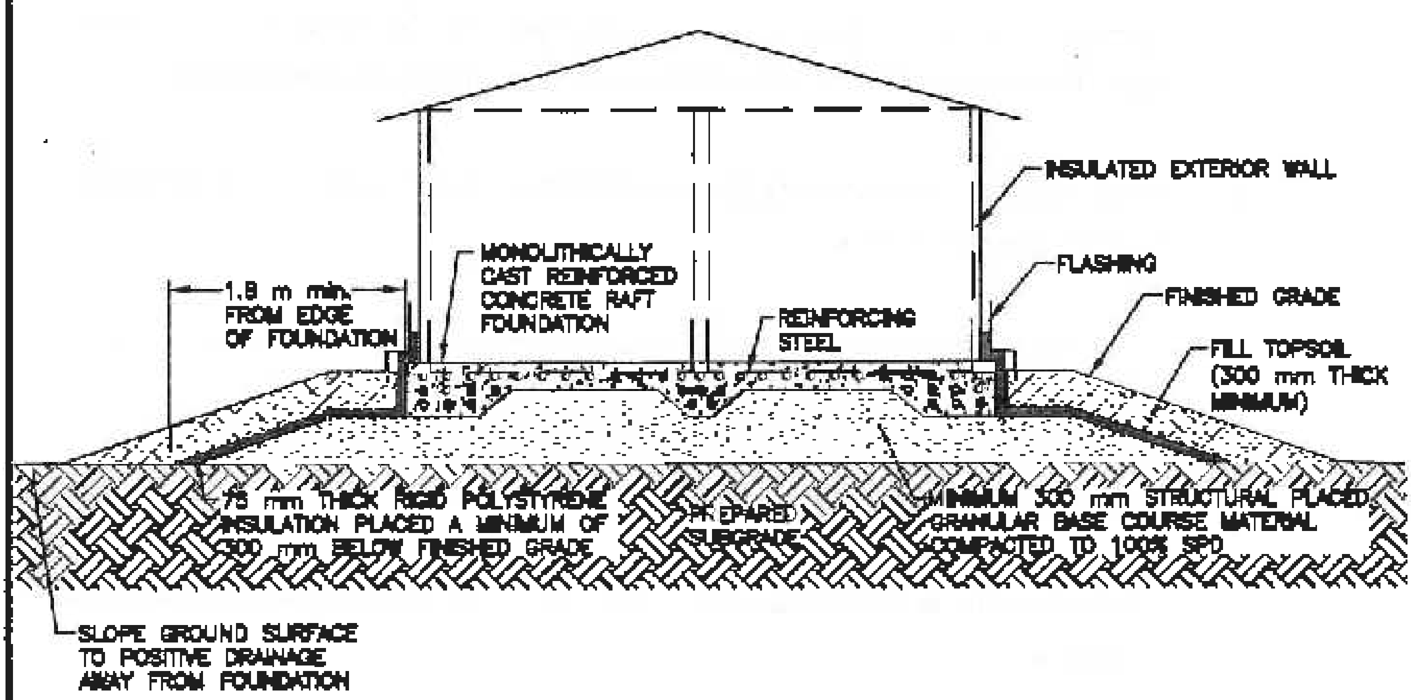
5. Reinforce the concrete slab and articulate the slab at regular intervals to provide for controlled cracking.

6. Provide positive site drainage away from the Residence.

7. Concrete slabs should not be constructed on wet or frozen granular material or subgrade.

8. Frost should not be allowed to penetrate beneath the slab just prior to, during or after construction.

9. To minimize the amount of differential movement associated with potential frost heaving, it is recommended that rigid polystyrene insulation be placed around the perimeter of the Residence. The insulation should be at least 75 mm in thickness and should extend out a minimum of 1.8 metres from the edge of the raft foundation. The insulation should be placed at least 300 mm below finished grade and should be sloped away from the building. The insulation should also be placed alongside/over the exterior face of the thickened edge portion of the slab to provide a continuous layer of insulation extending to the insulated exterior wall of the Residence. The insulation should have a compressive strength capable of supporting the design loading of the overlying structures.



NOTE:
1. THIS DRAWING IS FOR CONCEPTUAL PURPOSES ONLY. ACTUAL LOCATIONS MAY VARY AND NOT ALL STRUCTURES ARE SHOWN.

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GEOTECHNICAL
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DRAWING TITLE:
THICKENED PERIMETER MONOLITHICALLY CAST RAFT FOUNDATION

PROJECT:
PROPOSED MOWERY RESIDENTIAL SUBDIVISION
LA RONGE, SK

APPROVED BY: CZ

DRAWN BY: JH

DRAWING NUMBER: S08-6783-FIGURE No.1

DATE: FEBRUARY, 2009

SCALE: NOT TO SCALE

5.7 Floor Slabs

Floor slabs should be maintained as high as possible above the groundwater table (a minimum clearance of 500 mm is recommended). The following minimum provisions should be incorporated into the design of a heated grade-supported, cast-in-place, reinforced concrete slab subject to light floor loading.

1. Prepare the site in accordance with Section 5.2, Site Preparation. For residences with basements, over-excavate the subgrade soil to allow for the placement of a minimum of 200 mm of clean, drainage aggregate below the floor slab. Shape the subgrade surface to allow for free drainage to a sump pit(s). The drainage aggregate should meet the following gradation requirements.

Sieve Designation	Percent Passing
25.0 mm	100
9.5 mm	60 - 100
4.75 mm	44 - 90
2.00 mm	20 - 60
0.850 mm	0 - 53
0.425 mm	0 - 32
0.150 mm	0 - 10
0.071 mm	0 - 3

For Residences without basements, provide a minimum of 200 mm of compacted, crushed, granular base course material between the subgrade soils and the underside of the slab.

2. Excavate soft subgrade areas and replace with suitable, non-expansive fill, placed and compacted to 98 percent of standard Proctor density.

3. Subgrade fill, if required, should preferably consist of granular soil or locally available sand or glacial till soils, placed in thin lifts (maximum 150 mm loose) and compacted to 98 percent of standard Proctor density at optimum moisture content.

4. All granular fill placed above the subgrade elevation should be compacted to a minimum of 98 percent of standard Proctor density at optimum moisture content.

5. A sump pit(s) is recommended below basement floor slabs to collect any free water which may accumulate beneath the floor, and, to collect water from the perimeter drainage system. The surface of the subgrade should be positively graded towards the sump pit(s). The sump pit(s) should be perforated to allow water to drain in from the sub-slab drainage layer. All water collected within the sump pit should be discharged in a controlled manner well away from the Residence.

6. Separate the slab from the fill by means of a polyethylene vapour barrier.

7. Provide positive site drainage away from the Residence.

8. Floor slabs should not be constructed on desiccated, wet, or frozen subgrade soil, fill or base.

9. Frost should not be allowed to penetrate beneath the floor slab just prior to, during or after construction.

10. If insulation is to be utilized below the floor slab, a minimum of 1 metre of un-insulated space should be provided around the perimeter of the foundation walls to allow heat loss to the underside of the perimeter strip footing.

The above recommended floor system should perform satisfactorily if some floor movements resulting in cracking is deemed tolerable. Partition walls, staircases and any other structural elements resting on the floor slab should be designed to accommodate differential movements without imparting stresses on the upper levels of the Residence.

In unheated structures (i.e., garage), frost heaving is a common cause of differential slab movement and cracking. If some slab movements and cracking is not deemed tolerable, increasing the depth of granular fill, thickness of concrete slab and amount of reinforcing steel could be utilized to minimize floor slab distress. Heating the area to about +5 ° C with adequate air circulation would minimize the depth of frost penetration below the slab. Alternately, strategically placed rigid polystyrene insulation could be utilized to limit frost penetration below floor slabs.

5.8 Foundation Walls

Subsurface foundation walls should be designed to resist lateral earth pressure exerted by the backfill as well as the horizontal pressure induced by any surcharge loading. The lateral earth pressure may be calculated on the basis of an equivalent fluid pressure distribution of 9 kN/m^3 for drained conditions (i.e., perforated drainage pipe drainage system and clean, free-draining backfill as discussed below). The surcharge loading should be calculated on the basis of actual loads.

The lateral earth pressure loading of 9 kN/m^3 assumes that the backfill will be free-draining, uniformly placed around the structure and lightly compacted, and, a perforated drainage pipe will be installed alongside the foundation walls with the invert elevation at or below the base of the foundation. The perforated drainage pipe should be at least 100 mm in diameter and installed on non-woven geotextile capable of transmitting a flow of not less than 50 litres per second per square metre (ASTM D-4491).

The geotextile should be placed on naturally deposited, undisturbed soil or free-draining sand as may be required for levelling. The geotextile should be used to encapsulate at least 300 mm of clean, granular drainage aggregate above the invert of the drainage pipe. The clean drainage aggregate should meet the gradation requirements presented in Section 5.7 – Floor Slabs.

In the zone 300 mm above the invert of the drainage pipe and extending to within 600 mm of ground surface, clean, free-draining granular material with less than 5 percent material finer than the 0.071 mm sieve size should be used. The uppermost 600 mm should consist of clay or other low permeability material.

5.9 Foundation Concrete

Water soluble sulphate salts (gypsum crystals) exist in the geologic deposits in this region. Sulphate resistant (CSA Designation HS) cement is recommended for all foundation concrete in contact with the soil. All concrete at this site should be manufactured in accordance with current CSA standards.

It should be recognized that water soluble sulphate salts combined with moist soil conditions or low pH soils, could render the soil highly corrosive to some types of metal water lines, elbows, connectors, etc., in contact with the soil.

5.10 Subdivision Roads and Parking Structures

Suitable borrow soils (i.e., sand, clay or glacial till) exist at the subject site for construction of subdivision roads and parking areas.

It is anticipated that the subdivision roads and parking areas will be subject to predominantly passenger car and light truck traffic and infrequent heavy truck traffic. As a subgrade support, the California Bearing Ratio (CBR) rating of the compacted subgrade soil should be in the order of 3 to 5.

Based on the CBR rating, the following pavement and granular surfacing structures have been presented.

TABLE VI. THICKNESS DESIGN FOR ACCESS ROADS

Pavement/Granular Structure	Heavy Truck Traffic Wheel Loading (5,400 kg) (mm)		Light Truck/Passenger Vehicle Traffic Wheel Loading (1,830 kg) (mm)	
Surfacing Gravel	-	50	-	50
Asphalt Concrete	100	-	65	-
Granular Base (Min CBR = 65)	150	150	125	150
Granular Sub-Base (Min. CBR = 20)	250	400	175	225
Prepared Subgrade	(150)	(150)	(150)	(150)
Geotextile	*	*	*	*
Total Thickness	**500**	**600**	**365**	**425**

*Geotextile will be required where soft subgrade soils are encountered. High-strength (1,300 Newtons minimum), permeable, woven geotextile is recommended.

All granular fill placed above the subgrade elevation should be placed in thin lifts (150 mm loose, maximum) and compacted to at least 98 percent of standard Proctor density. The granular base, sub-base course and surfacing material should meet the following aggregate gradation requirements.

TABLE VII. AGGREGATE GRADATION REQUIREMENTS

Grain Size (mm)	Percent Passing		
	Surfacing Gravel	Base Course	Sub-Base Course
50.0	–	–	100
25.0	100	100	85 – 100
18.0	–	87 – 100	80 – 100
12.5	–	72 – 93	70 – 100
5.0	45 – 80	45 – 77	50 – 85
2.0	25 – 60	26 – 56	35 – 75
0.900	–	18 – 39	25 – 50
0.400	0 – 30	13 – 28	15 – 35
0.160	–	7 – 18	8 – 22
0.071	–	6 – 11	0 – 13
Plasticity Index (%)	0 – 6	0 – 6	0 – 6
CBR (min.)	–	65	20
% Fracture (min.)	40	50	–

The following minimum general recommendations should be incorporated into the design of the proposed subdivision roads and parking structures.

1. Prepare the site in accordance with Section 6.2, Site Preparation.

2. Excavate soft subgrade areas and replace with suitable soil compacted to 98 percent of standard Proctor density at optimum moisture content. Geotextile may be required to reinforce and stabilize the subgrade soils.

3. All borrow material for the subject roadways and parking areas should be placed in thin lifts (maximum 150 mm loose) and compacted to 98 percent of standard Proctor density at optimum moisture content.

4. In cut areas, the subgrade should be scarified (to 150 mm in light traffic areas and 300 mm in heavy traffic areas) and re-compacted to 98 percent of standard Proctor density.

5. All common borrow used for embankment construction should consist of imported granular material or locally available sand, clay or glacial till soils.

6. All granular fill should be placed in thin lifts (maximum 150 mm loose) and compacted to at least 98 percent of standard Proctor density.

7. Positive surface drainage is recommended to minimize the potential for moisture infiltration into the subgrade soil. Ditches and culverts should be provided where necessary to provide adequate site drainage. Surface water should be prevented from seeping back under the outer edges of the road structure. The subject roadways should be constructed with a shoulder height of at least 1.2 metres above ditchbottom elevation.

8. For sand, clay or glacial till borrow materials, roadway embankment slopes should be no steeper than 3.0 Horizontal to 1.0 Vertical (3H:1V). Similarly, ditch sideslopes should be no steeper than 3H:1V.

9. Erosion protection is recommended for all embankment sideslopes. The slopes should be covered with topsoil and seeded to encourage vegetation growth. Alternately, erosion control blankets (North American Green S150 or equivalent) or hydromulch could be installed.

10. Periodic maintenance of the granular/pavement surface will be required (i.e., grading of the gravel surface or crack sealing of the pavement surface). The final road grade should be elevated a minimum of 600 mm above the average terrain to minimize snow accumulation on the road.

6.0 LIMITATIONS

The presentation of the summary of the field drill logs and foundation design recommendations has been completed as authorized. Twelve, 150 mm diameter test holes were dry drilled using our continuous flight, solid stem auger drill rig. Field drill logs were compiled for the Test Holes during test drilling which, we believe, were representative of the subsurface conditions at the Test Hole locations at the time of test drilling. Variations in the subsurface conditions from that shown on the drill logs at locations other than the exact Test Hole locations should be anticipated. If conditions should differ from those reported here, then we should be notified immediately in order that we may examine the conditions in the field and reassess our recommendations in the light of any new findings.

The Terms of Reference for this geotechnical investigation did not include any environmental assessment of the site. No detectable evidence of environmentally sensitive materials such as hydrocarbon odour was detected during the actual time of the field test drilling program. If, on the basis of any knowledge, other than that formally communicated to us, there is reason to suspect that environmentally sensitive materials may exist, then additional test holes should be drilled and samples recovered for chemical analysis.

The subsurface investigation necessitated the drilling of deep test holes and excavation of test pits. The test holes and test pits were backfilled at the completion of test drilling. Please be advised that some settlement of the backfill materials will occur which may leave a depression or an open hole. It is the responsibility of the client to inspect the site and backfill, as required, to ensure that the ground surface at each Test location is maintained level with the existing grade.

This report has been prepared for the exclusive use of the Northern Revenue Sharing Trust Account, Associated Engineering and their agents for specific application to the proposed Mowery Residential Subdivision to be constructed in La Ronge, Saskatchewan. It has been prepared in accordance with generally accepted geotechnical engineering practices and no other warranty, express or implied, is made.

Any use which a Third Party makes of this report, or any reliance on decisions to be made based on it, are the responsibility of such Third Parties. 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.

The acceptance of responsibility for the design/construction recommendations presented in this report are contingent on adequate and/or full time inspection (as required, based on site conditions at the time of construction) by a representative of the Geotechnical Consultant. PMEL will not accept any responsibility on this project for any unsatisfactory performance if adequate and/or full time inspection is not performed by a representative of PMEL.

If this report has been transmitted electronically, it has been digitally signed and 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.

C.Z.
09 02 10

Cory Zubkowski, P. Eng.

K. Pardoski

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CZ/KP/clb

Association of Professional Engineers &
Geoscientists of Saskatchewan
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P. MACHIBRODA ENGINEERING LTD.
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Geotechnical ______ 12138
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APPENDIX A

EXPLANATION OF TERMS ON TEST HOLE LOGS

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CLASSIFICATION OF SOILS

Coarse-Grained Soils: Soils containing particles that are visible to the naked eye. They include gravels and sands and are generally referred to as cohesionless or non-cohesive soils. Coarse-grained soils are soils having more than 50 percent of the dry weight larger than particle size 0.080 mm.

Fine-Grained Soils: Soils containing particles that are not visible to the naked eye. They include silts and clays. Fine-grained soils are soils having more than 50 percent of the dry weight smaller than particle size 0.080 mm.

Organic Soils: Soils containing a high natural organic content.

Soil Classification By Particle Size

Clay – particles of size	< 0.002 mm
Silt – particles of size	0.002 – 0.080 mm
Sand – particles of size	0.08 – 2.0 mm
Gravel – particles of size	2.0 – 80 mm
Cobbles – particles of size	80 – 200 mm
Boulders – particles of size	>200 mm

TERMS DESCRIBING CONSISTENCY OR CONDITION

Coarse-grained soils: Described in terms of compactness condition and are often interpreted from the results of a Standard Penetration Test (SPT). The standard penetration test is described as the number of blows, N, required to drive a 51 mm outside diameter (O.D.) split barrel sampler into the soil a distance of 0.3 m (from 0.15 m to 0.45 m) with a 63.5 kg weight having a free fall of 0.76 m.

Compactness Condition	SPT N-Index (blows per 0.3 m)
Very loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	Over 50

Fine-Grained Soils: Classified in relation to undrained shear strength.

Consistency	Undrained Shear Strength (kPa)	N Value (Approximate)	Field Identification
Very Soft	<12	0-2	Easily penetrated several centimetres by the fist.
Soft	12-25	2-4	Easily penetrated several centimetres by the thumb.
Firm	25-50	4-8	Can be penetrated several centimetres by the thumb with moderate effort.
Stiff	50-100	8-15	Readily indented by the thumb, but penetrated only with great effort.
Very Stiff	100-200	15-30	Readily indented by the thumb nail.
Hard	>200	>30	Indented with difficulty by the thumbnail.

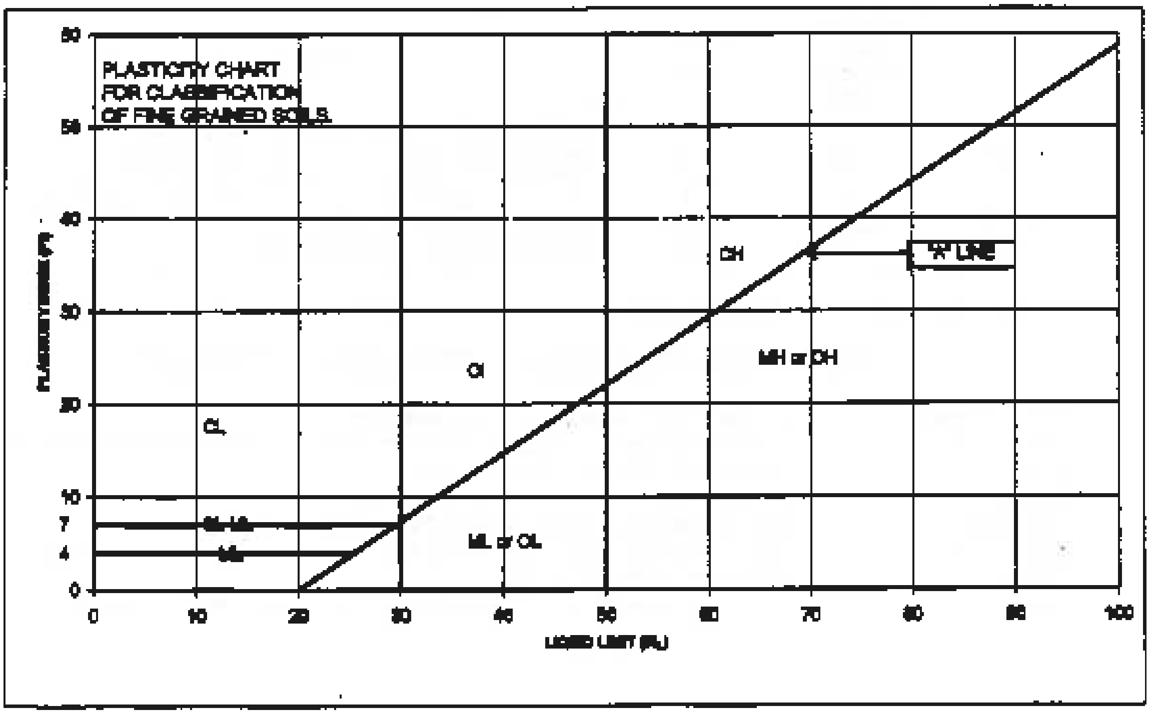
Organic Soils: Readily identified by colour, odour, spongy feel and frequently by fibrous texture.

DESCRIPTIVE TERMS COMMONLY USED TO CHARACTERIZE SOILS

Poorly Graded	- predominance of particles of one grain size.
Well Graded	- having no excess of particles in any size range with no intermediate sizes lacking.
Mottled	- marked with different coloured spots.
Nuggety	- structure consisting of small prismatic cubes.
Laminated	- structure consisting of thin layers of varying colour and texture.
Slickensided	- having inclined planes of weakness that are slick and glossy in appearance.
Fissured	- containing shrinkage cracks.
Fractured	- broken by randomly oriented interconnecting cracks in all 3 dimensions.

SOIL CLASSIFICATION SYSTEM (MODIFIED U.S.C.)

MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR AND OFTEN FIBROUS TEXTURE
COARSE-GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN NO. 200 SIEVE SIZE)	GRAVELS More than half coarse fraction larger than No. 4 sieve size	CLEAN GRAVELS	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES <5% FINES	$C_u = \frac{D_{60}}{D_{10}} > 4$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3
			GP	POORLY-GRADED GRAVELS AND GRAVEL-SAND MIXTURES <5% FINES	NOT MEETING ALL ABOVE REQUIREMENTS FOR GW
		DIRTY GRAVELS	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES >12% FINES	ATTERBERG LIMITS BELOW "A" LINE OR PI < 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES >12% FINES	ATTERBERG LIMITS ABOVE "A" LINE WITH PI > 7
	SANDS More than half coarse fraction smaller than No. 4 sieve size	CLEAN SANDS	SW	WELL-GRADED SANDS, GRAVELLY SANDS MIXTURES <5% FINES	$C_u = \frac{D_{60}}{D_{10}} > 6$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3
			SP	POORLY-GRADED SANDS OR GRAVELLY SANDS <5% FINES	NOT MEETING ALL GRADATION REQUIREMENTS FOR SW
		DIRTY SANDS	SM	SILTY SANDS, SAND-SILT MIXTURES >12% FINES	ATTERBERG LIMITS BELOW "A" LINE OR PI < 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES >12% FINES	ATTERBERG LIMITS ABOVE "A" LINE WITH PI >7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT PASSING NO. 200 SIEVE SIZE)	SILTS Below "A" line on plasticity chart; negligible organic content		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	$W_L < 50$
			MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS	$W_L > 50$
	CLAYS Above "A" line on plasticity chart; negligible organic content		CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS	$W_L < 30$
			CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS	$W_L > 30 < 50$
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	$W_L > 50$
	ORGANIC SILTS & ORGANIC CLAYS Below "A" line on plasticity chart		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	$W_L < 50$
			OH	ORGANIC CLAYS OF HIGH PLASTICITY	$W_L > 50$



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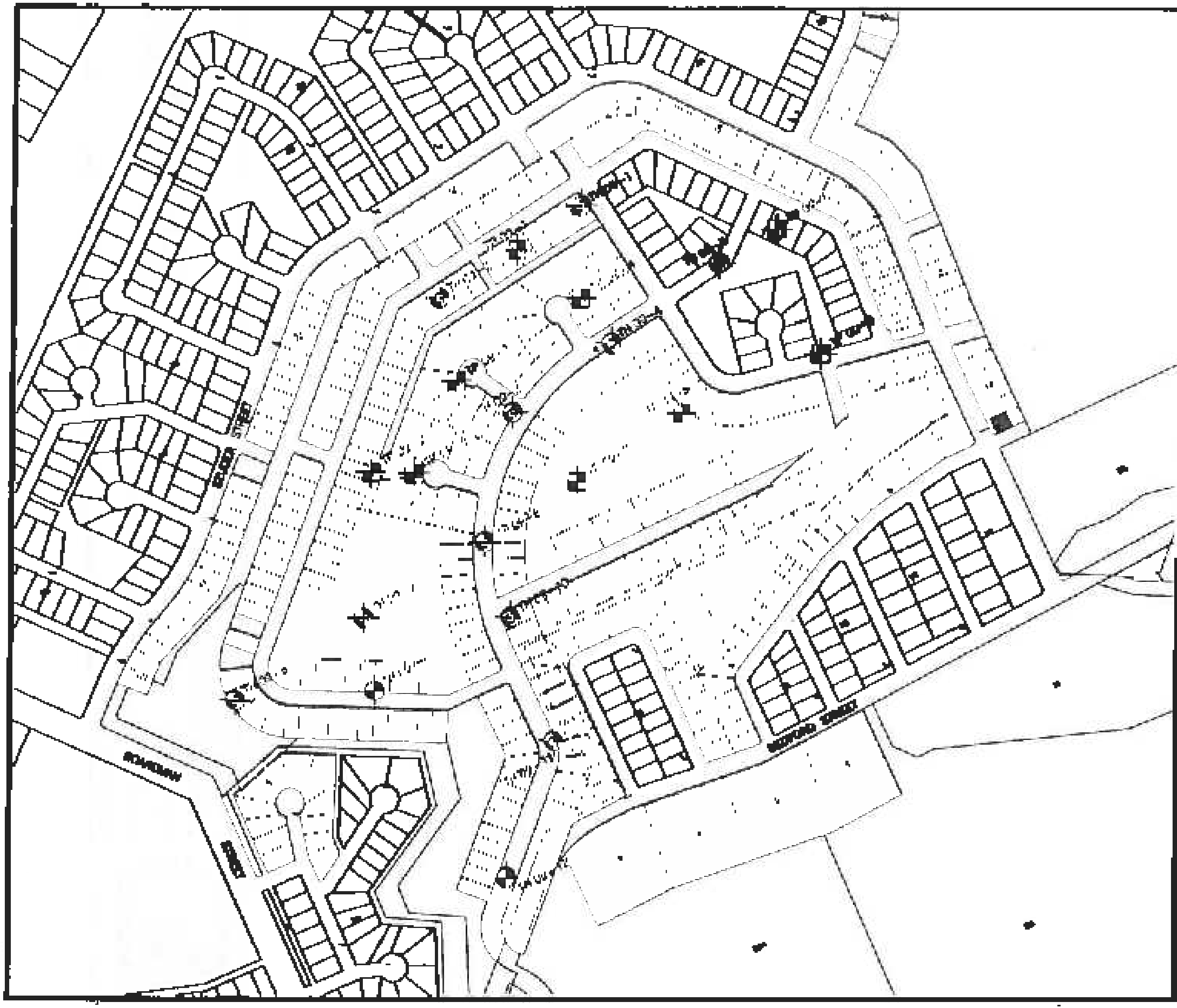
DRAWINGS



NOTE:
1. THIS DRAWING IS FOR CONCEPTUAL PURPOSES ONLY. ACTUAL LOCATIONS MAY VARY AND NOT ALL STRUCTURES ARE SHOWN.
2. THIS DRAWING WAS COMPILED FROM ASSOCIATED ENGINEERING PROJECT: TOWN OF LA RONGE, MOWERY SUBDIVISION. PROJECT No. [illegible]. DRAWING: EXISTING AND PROPOSED WETLANDS, FIGURE 7. DRAWING No. [illegible].
3. TEST HOLE ELEVATIONS TAKEN FROM SURVEY DATA PROVIDED BY ASSOCIATED ENGINEERING.

LEGEND

⌖ — PMEL TEST HOLE	⌖ — PMEL TEST PIT
● — PMEL TEST HOLE (PIEZOMETER INSTALLED)	

P. MACHIBRODA ENGINEERING LTD.

CONSULTING
GEOENVIRONMENTAL
GEOTECHNICAL
ENGINEERING

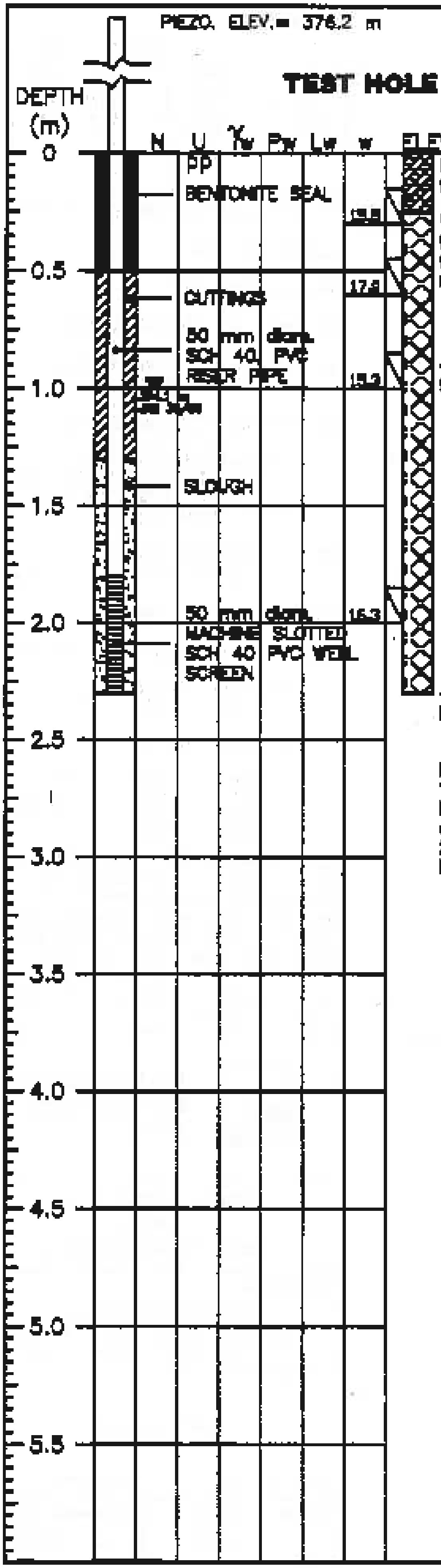
2023 B FAITHFULL AVENUE
SASKATOON, SK
S7K [illegible]

DRAWING TITLE:
SITE PLAN - TEST HOLE
AND TEST PIT LOCATIONS

PROJECT:
PROPOSED MOWERY RESIDENTIAL SUBDIVISION
LA RONGE, SK

APPROVED BY: CZ	DRAWN BY: JH
DATE: JANUARY, 2009 SCALE: 1:4000	DRAWING NUMBER: S08-6763-1

TEST HOLE 09-1



PEAT, organic, black, rootlets, frozen.

GLACIAL TILL, sand, silty, some gravel, trace clay, compact, well graded, fine to coarse grained, moist, brown, cobbles/boulders.

—wet, seepage, sloughing below 900 mm.

—auger refusal on assumed bedrock at 2.3 m.

NOTE:
1. Auger refusal on assumed bedrock at 2.3 m during first drilling attempt.
2. Test Hole sloughed to 1.3 m I.A.D.

LEGEND:

▨ PEAT ▤ FILL ▩ GRAVEL ▦ SAND ▥ SILT ▧ CLAY ☒ GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N.....STANDARD PENETRATION TEST (ROPE—CATHEAD & DONUT HAMMER) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4 ....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

■ SHELBY TUBE ☒ SPLIT SPOON ◪ CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT: PROPOSED MOWERY RESIDENTIAL SUBDIVISION

LOCATION: LA RONGE, SK

NORTHING:

EASTING:

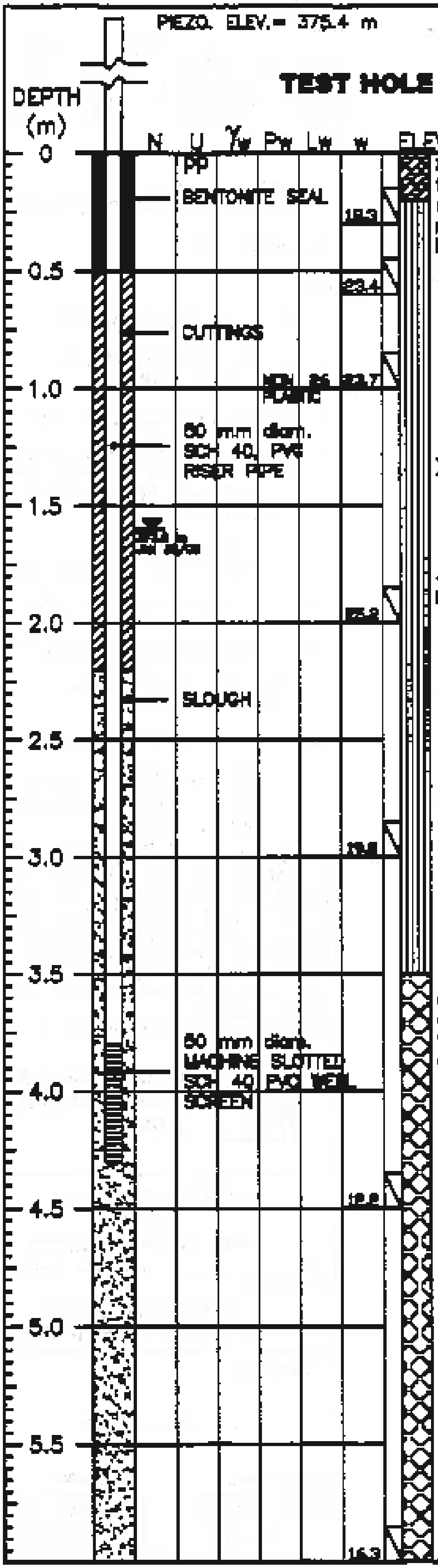
DATE DRILLED: JAN 22/09

DRAWING NUMBER: S08-6783-2

PIEZO. ELEV.= 375.4 m

TEST HOLE 09-2

ELEV: 374.6 m



PEAT, organic, black, rootlets, frozen.

SILT, some sand, trace clay, firm, non to low plastic, moist, olive brown.

—wet, seepage, sloughing below 1.3 m.

—trace sand, cobbles/boulders below 1.8 m.

GLACIAL TILL, sand, silty, some gravel, trace clay, compact, well graded, fine to coarse grained, wet, brown, seepage, sloughing, cobbles/boulders.

NOTE:
1. Test Hole sloughed to 2.2 m I.A.D.

LEGEND:

▨ PEAT ☰ FILL ● GRAVEL ▦ SAND ▥ SILT ▧ CLAY ⊠ GLACIAL TILL

w....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γw...WET UNIT WEIGHT (kN/m³)

U....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N.....STANDARD PENETRATION TEST (ROPE-CATHEAD & DONUT HAMMER) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO₄.....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

■ SHELBY TUBE ⊠ SPLIT SPOON ◸ CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:
PROPOSED MOWERY RESIDENTIAL SUBDIVISION

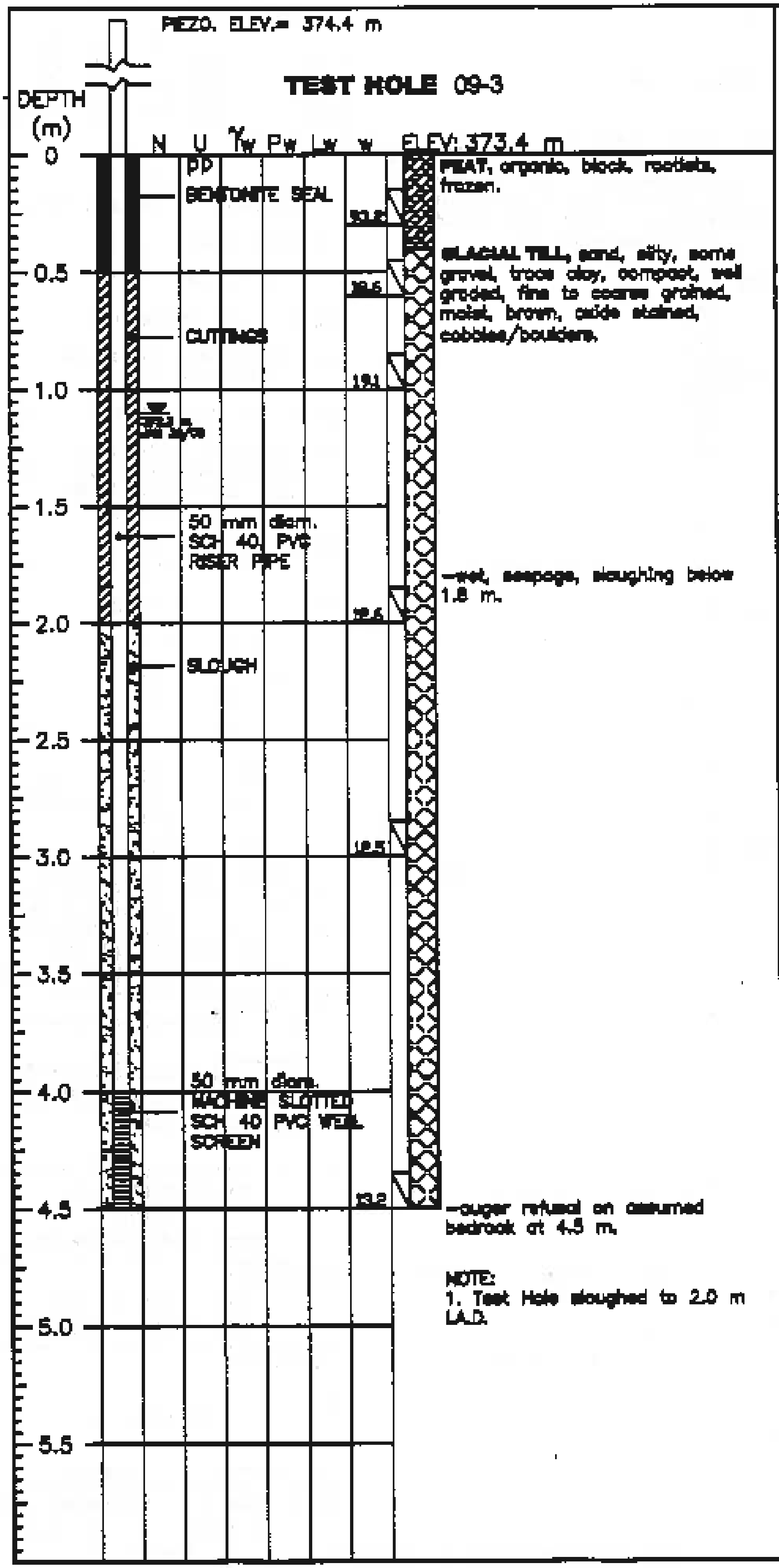
LOCATION:
LA RONGE, SK

NORTHING:

EASTING:

DATE DRILLED:	**DRAWING NUMBER:**
JAN 22/09	S08-6783-3

TEST HOLE 09-3



PEAT, organic, black, rootlets, frozen.

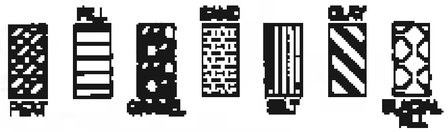
GLACIAL TILL, sand, silty, some gravel, trace clay, compact, well graded, fine to coarse grained, moist, brown, oxide stained, cobbles/boulders.

—wet, seepage, sloughing below 1.8 m.

—auger refusal on assumed bedrock at 4.5 m.

NOTE:
1. Test Hole sloughed to 2.0 m I.A.D.

LEGEND:



w....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N....STANDARD PENETRATION TEST (ROPE-CATHEAD & DONUT HAMMER) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4 ....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

■ SHELBY TUBE ☒ SPLIT SPOON ◩ CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT: PROPOSED MOWERY RESIDENTIAL SUBDIVISION

LOCATION: LA RONGE, SK

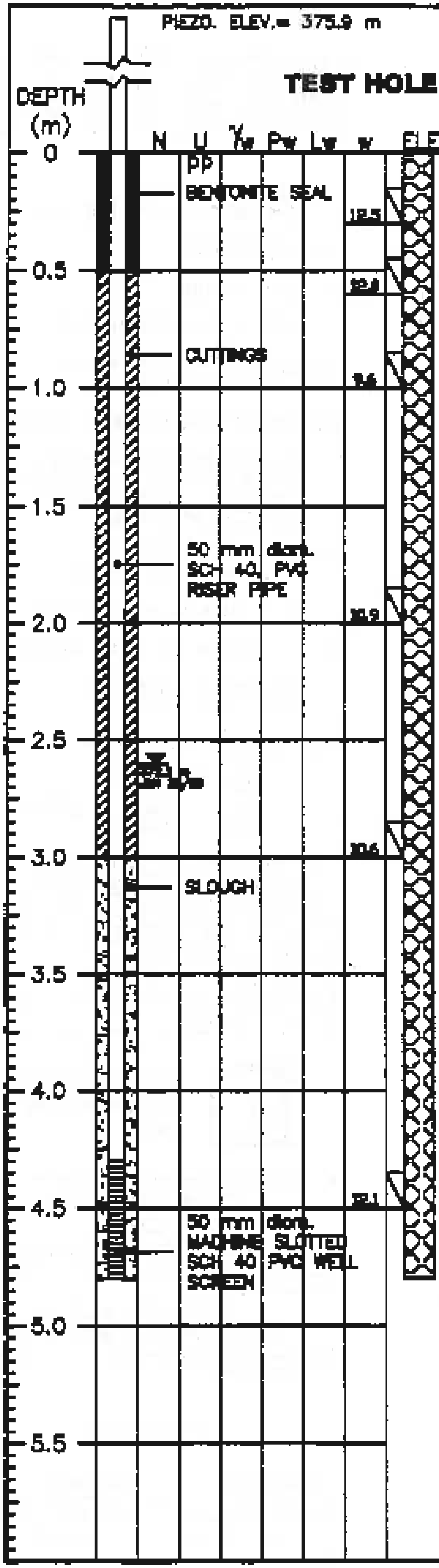
NORTHING:

EASTING:

DATE DRILLED: JAN 22/09

DRAWING NUMBER: S08-6783-4

TEST HOLE 09-4



GLACIAL TILL, sand, silty, some gravel, trace clay, compact, well graded, fine to coarse grained, moist, brown, oxide stained, cobbles/boulders.
–frozen to 400 mm.

–wet, seepage, sloughing below 3.1 m.

–auger refusal on assumed bedrock at 4.8 m.

NOTE:
1. Test Hole sloughed to 3.0 m I.A.D.

LEGEND:

▨	▤	●	⠿	▥	▧	☒
PEAT	FILL	GRAVEL	SAND	SILT	CLAY	GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N.....STANDARD PENETRATION TEST (ROPE–CATHEAD & DONUT HAMMER) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4.....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

■	☒	▧
SHELBY TUBE	SPLIT SPOON	CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:
PROPOSED MOWERY RESIDENTIAL SUBDIVISION

LOCATION:
LA RONGE, SK

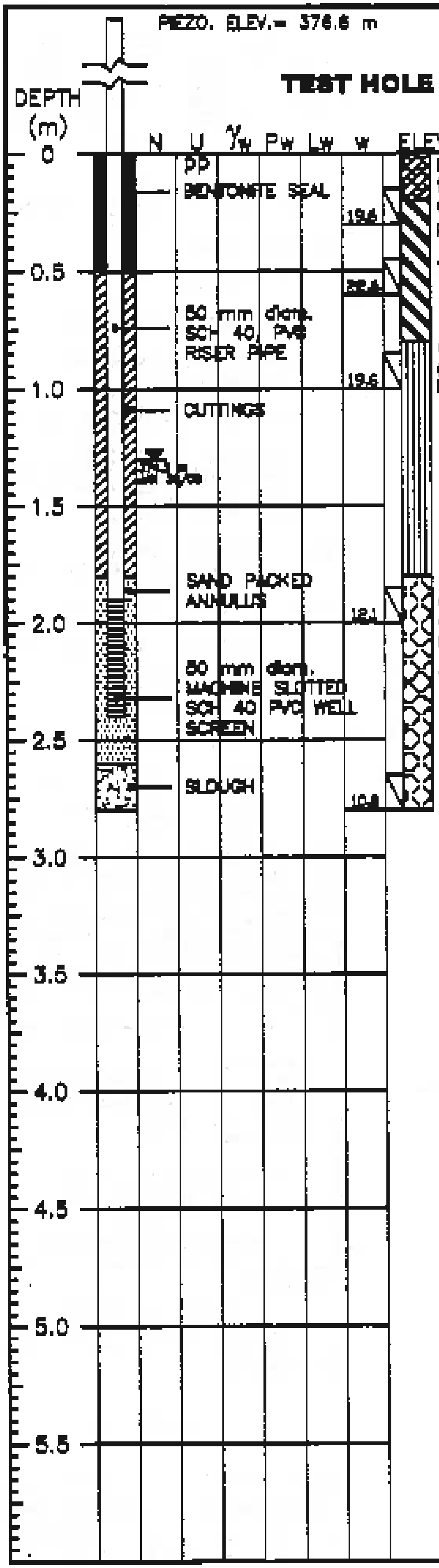
NORTHING:

EASTING:

DATE DRILLED:
JAN 22/09

DRAWING NUMBER:
S08-6783-5

TEST HOLE 09-5



PEAT, organic, black, rootlets, frozen.

CLAY, some silt, stiff, highly plastic, moist, brown.

–frozen to 400 mm.

SILT, trace clay, trace sand, trace gravel, firm, low plastic, moist, brown.

GLACIAL TILL, sand, silty, some gravel, trace clay, compact, well graded, fine to coarse grained, moist, brown, oxide stained.

—cobbles/boulders below 2.2 m.

—auger refusal on assumed bedrock at 2.8 m.

NOTE:
1. Auger refusal on assumed bedrock at 2.8 m during first drilling attempt.
2. Test Hole sloughed to 2.6 m I.A.D.

LEGEND:

PEAT, FILL, GRAVEL, SAND, SILT, CLAY, GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N.....STANDARD PENETRATION TEST (ROPE–CATHEAD & DONUT HAMMER) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4.....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

■ SHELBY TUBE ☒ SPLIT SPOON ◺ CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT: PROPOSED MOWERY RESIDENTIAL SUBDIVISION

LOCATION: LA RONGE, SK

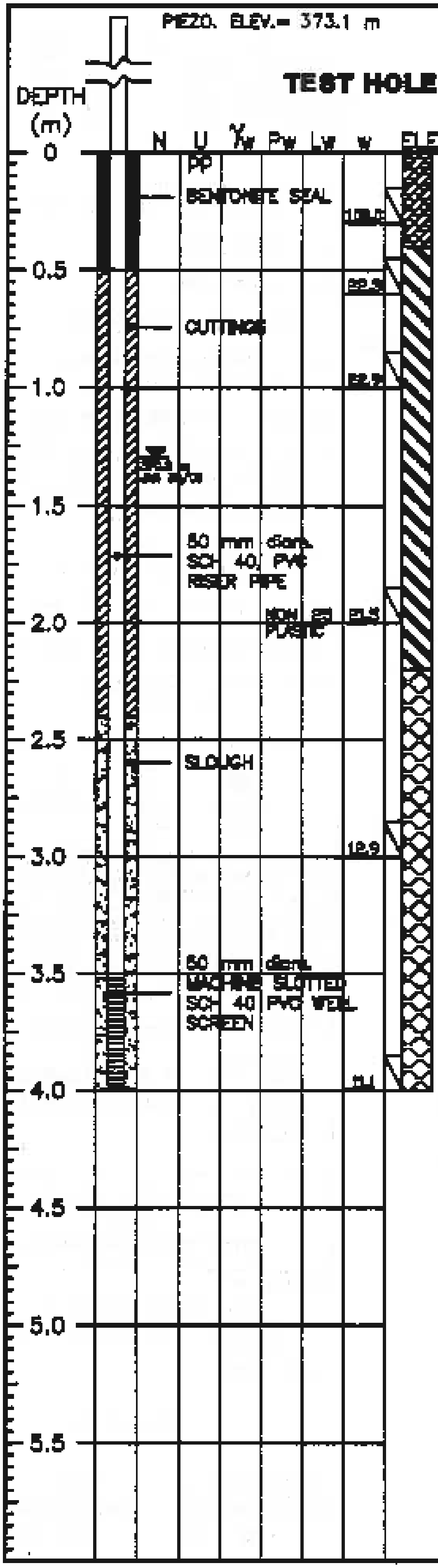
NORTHING:

EASTING:

DATE DRILLED: JAN 22/09

DRAWING NUMBER: S08-6783-5

TEST HOLE 09-6



PEAT, organic, black, rootlets, frozen.

CLAY, some silt, stiff, highly plastic, moist, olive brown.

—silty, firm, low plastic, oxide stained below 1.5 m.

GLACIAL TILL, sand, silty, some gravel, trace clay, compact, well graded, fine to coarse grained, wet, brown, oxide stained, seepage, sloughing.

—cobbles/boulders below 3.2 m.

—auger refusal on assumed bedrock at 4.0 m.

NOTE:
1. Test Hole sloughed to 2.4 m I.A.D.

LEGEND:

- ▨ PEAT
- ☰ FILL
- ● GRAVEL
- ░ SAND
- ▥ SILT
- ▧ CLAY
- ☒ GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m^3)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm^2)

N.....STANDARD PENETRATION TEST (ROPE-CATHEAD & DONUT HAMMER) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

■ SHELBY TUBE ☒ SPLIT SPOON ⧅ CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:
PROPOSED MOWERY RESIDENTIAL SUBDIVISION

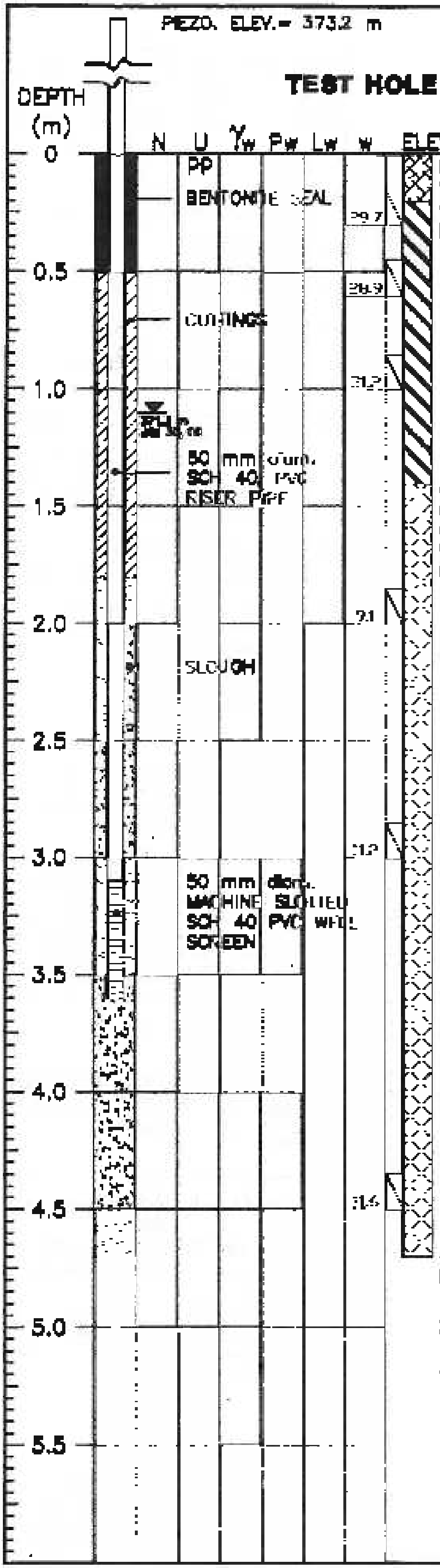
LOCATION:
LA RONGE, SK

NORTHING:
EASTING:

DATE DRILLED: JAN 22/09

DRAWING NUMBER: S08-6783-7

TEST HOLE 09-7



PEAT, organic, black, rootlets, frozen.

CLAY, some silt, stiff, highly plastic, moist, brown.

GLACIAL TILL, sand, silty, some gravel, trace clay, compact, well graded, fine to coarse grained, wet, brown, oxide stained, seepage, sloughing, cobbles/boulders.

—auger refusal on assumed bedrock at 4.7 m.

NOTE:
1. Test Hole sloughed to 1.8 m I.A.D.

LEGEND:

- PEAT ▨
- FILL ☰
- GRAVEL
- SAND
- SILT
- CLAY ▧
- GLACIAL TILL ☒

w....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N....STANDARD PENETRATION TEST (ROPE—CATHEAD & DONUT HAMMER) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

■ SHELBY TUBE ☒ SPLIT SPOON ◸ CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

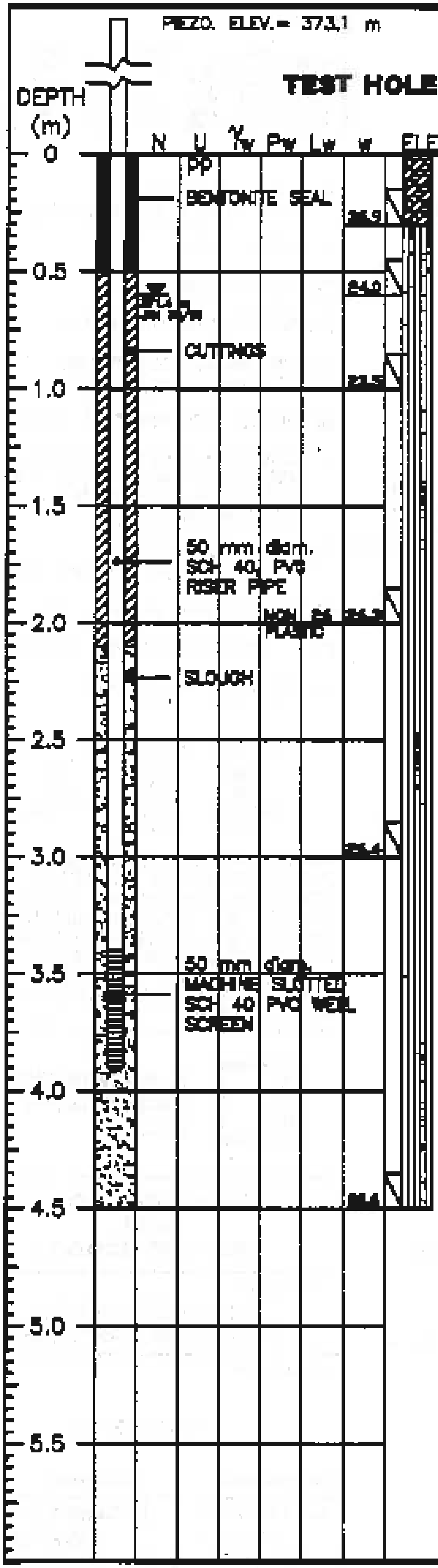
PROJECT:
PROPOSED MOWERY RESIDENTIAL SUBDIVISION

LOCATION:
LA RONGE, SK

NORTHING:
EASTING:

DATE DRILLED:	**DRAWING NUMBER:**
JAN 22/09	S08 6763-6

TEST HOLE 09-8



PEAT, organic, black, rootlets, frozen.

SILT, some sand, trace clay, firm, non to low plastic, moist, olive brown.

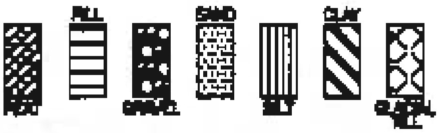
–wet, seepage, sloughing below 1.8 m.

–cobbles/boulders below 4.0 m.

–auger refusal on assumed bedrock at 4.5 m.

NOTE:
1. Test Hole sloughed to 2.1 m I.A.D.

LEGEND:



w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m^3)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm^2)

N.....STANDARD PENETRATION TEST (ROPE-CATHEAD & DONUT HAMMER) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO$_4$.....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

■ SHELBY TUBE ☒ SPLIT SPOON ◩ CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:
PROPOSED MOWERY RESIDENTIAL SUBDIVISION

LOCATION:
LA RONGE, SK

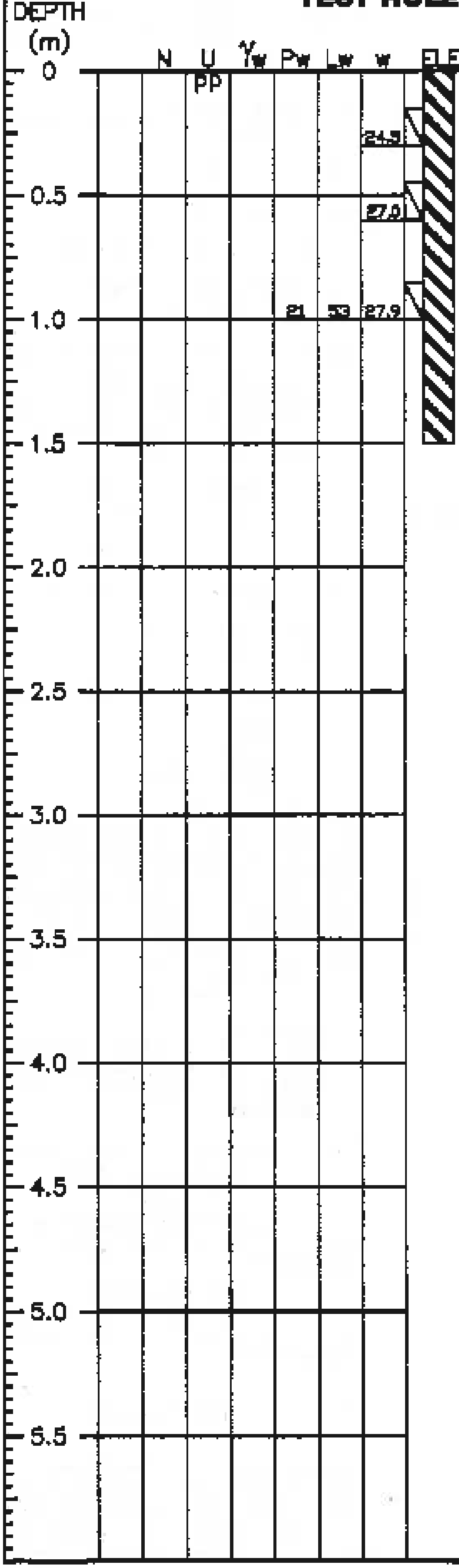
NORTHING:

EASTING:

DATE DRILLED: JAN 22/09

DRAWING NUMBER: S08-6763-9

TEST HOLE 09-9



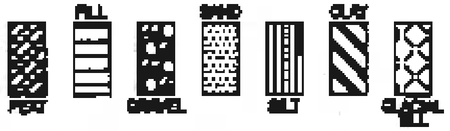
CLAY, some silt, stiff, highly plastic, moist, brown.

-frozen to 500 mm.

-cobbles/boulders below 1.4 m.
-auger refusal on assumed bedrock at 1.5 m.

NOTE:
1. Auger refusal on assumed bedrock at 1.5 m during first three drilling attempts.
2. Test Hole open to 1.5 m and dry I.A.D.

LEGEND:



w....WATER CONTENT
(PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m^3)

U....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm^2)

N....STANDARD PENETRATION TEST
(ROPE–CATHEAD & DONUT HAMMER)
(50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4 ....SULPHATE CONTENT
(PERCENT OF DRY SOIL WEIGHT)

P200....% PASSING No. 200 SIEVE

I.A.D....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL
(TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:
PROPOSED MOWERY RESIDENTIAL SUBDIVISION

LOCATION:
LA RONGE, SK

NORTHING:
EASTING:

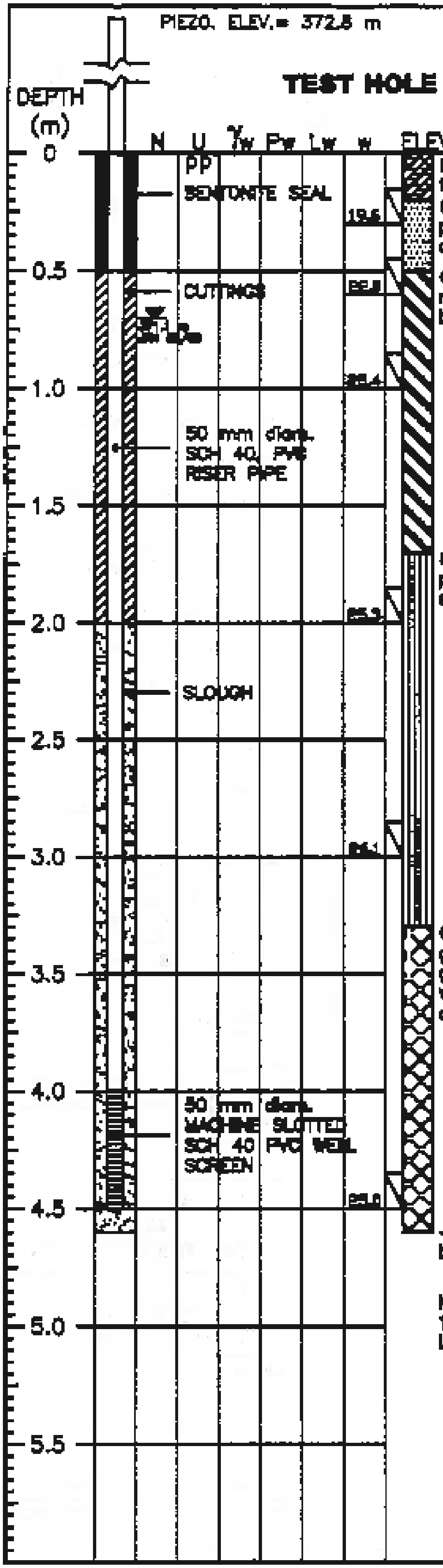
DATE DRILLED:
JAN 22/09

DRAWING NUMBER:
S08-5783-10

TEST HOLE 09-10

PIEZO. ELEV.= 372.8 m

ELEV: 371.8 m



PEAT, organic, black, rootlets, frozen.

SAND, silty, loose to compact, poorly graded, fine grained, wet, olive brown, seepage, sloughing.

CLAY, some silt, firm to stiff, medium to highly plastic, moist, brown.

SILT, some clay, firm, non to low plastic, wet, olive brown, seepage, sloughing.

GLACIAL TILL, sand, silty, some gravel, trace clay, compact, well graded, fine to coarse grained, wet, brown, seepage, sloughing, cobbles/boulders.

—auger refusal on assumed bedrock at 4.6 m.

NOTE:
1. Test Hole sloughed to 2.0 m I.A.D.

LEGEND:

▨ PEAT ▤ FILL ● GRAVEL ░ SAND ▥ SILT ▧ CLAY ☒ GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N.....STANDARD PENETRATION TEST (ROPE-CATHEAD & DONUT HAMMER) (30/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4.....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

■ SHELBY TUBE ☒ SPLIT SPOON ◸ CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:
PROPOSED MOWERY RESIDENTIAL SUBDIVISION

LOCATION:
LA RONGE, SK

NORTHING:

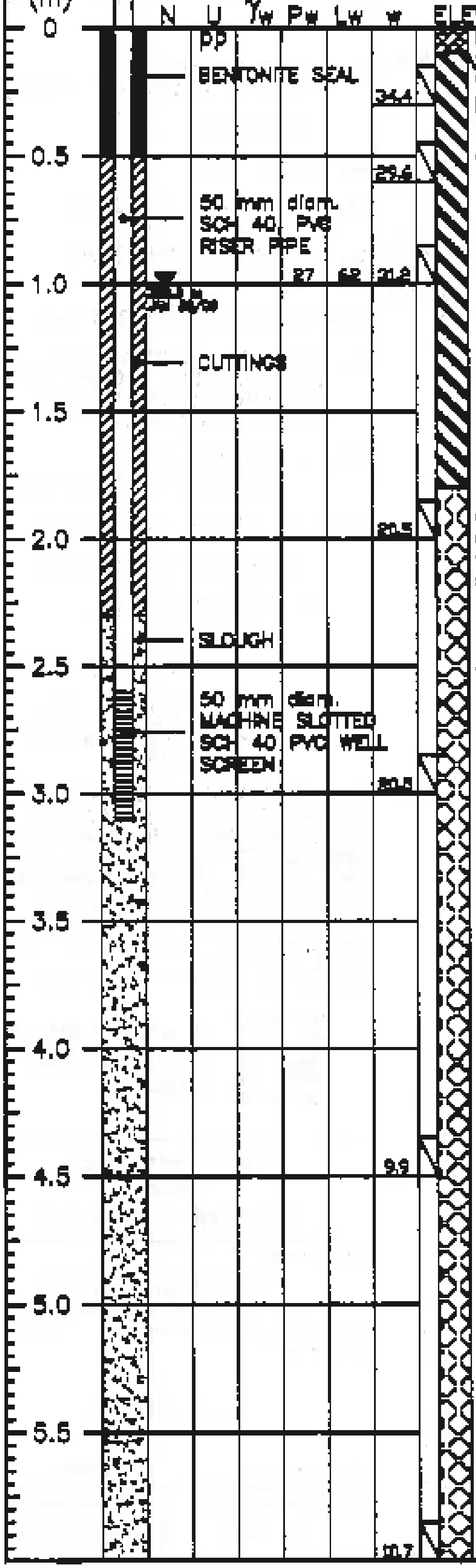
EASTING:

DATE DRILLED:	**DRAWING NUMBER:**
JAN 22/09	S08-6783-11

PIEZO. ELEV.= 370.9 m

TEST HOLE 09-11

ELEV: 369.9 m



PEAT, organic, black, rootlets, frozen.

CLAY, some silt, stiff, highly plastic, moist, brown.

–frozen to 500 mm.

GLACIAL TILL, sand, silty, some gravel, trace clay, compact, well graded, fine to coarse grained, wet, brown, oxide stained, seepage, sloughing.

–cobbles/boulders below 2.6 m.

–dense, moist, mottled brown/grey below 3.4 m.

NOTE:
1. Test Hole sloughed to 2.3 m I.A.D.

LEGEND:

▨	☰	●	▒	▥	▧	☒
PEAT	FILL	GRAVEL	SAND	SILT	CLAY	GLACIAL TILL

w....WATER CONTENT
(PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N....STANDARD PENETRATION TEST (ROPE–CATHEAD & DONUT HAMMER) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4 ....SULPHATE CONTENT
(PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

■	☒	◸
SHELBY TUBE	SPLIT SPOON	CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA
ENGINEERING
LTD.

FIELD DRILL LOG
AND
SOIL TEST RESULTS

PROJECT:
PROPOSED MOWERY
RESIDENTIAL SUBDIVISION

LOCATION:
LA RONGE, SK

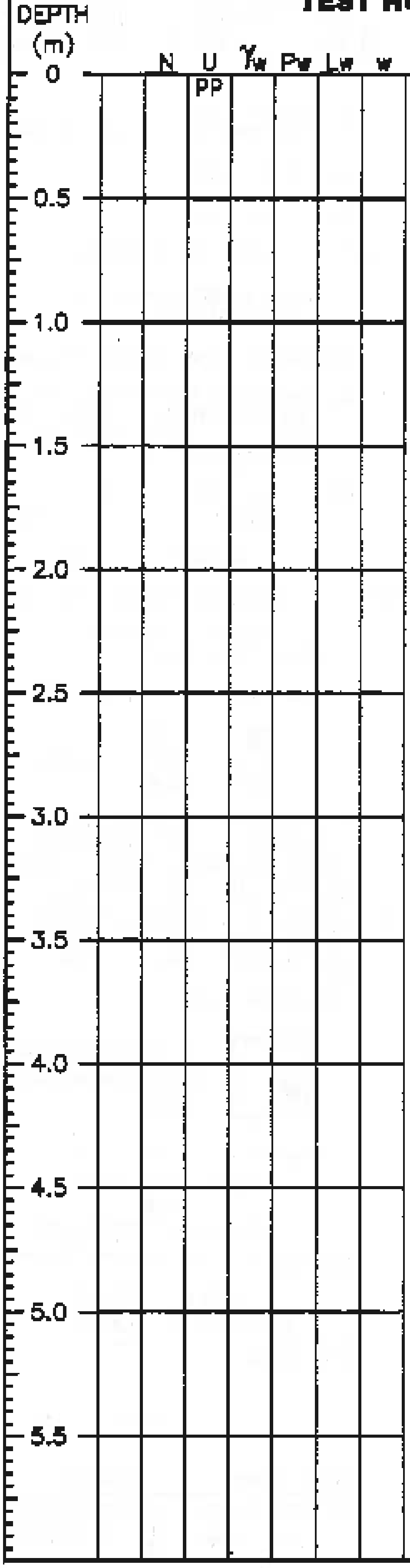
NORTHING:

EASTING:

DATE DRILLED: JAN 22/09

DRAWING NUMBER: S08-6783-12

TEST HOLE 09-12



ELEV: 370.2 m

▨ **SAND**, some gravel, some silt, dense, well graded, fine to coarse grained, moist, brown, frozen.

—auger refusal on assumed bedrock at 300 mm.

NOTE:
1. Auger refusal on assumed bedrock at 200 to 300 mm during first two drilling attempts.
2. Test Hole open to 300 mm and dry I.A.D.

LEGEND:

- ▨ PEAT
- ☰ FILL
- ● GRAVEL
- ▒ SAND
- ▥ SILT
- ▧ CLAY
- ☒ GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N.....STANDARD PENETRATION TEST (ROPE—CATHEAD & DONUT HAMMER) (50/125 = BLOWS/SAMPLER PENETRATION (mm))

SO_4.....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

■ SHELBY TUBE ☒ SPLIT SPOON ◸ CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:
PROPOSED MOWERY RESIDENTIAL SUBDIVISION

LOCATION:
LA RONGE, SK

NORTHING:

EASTING:

DATE DRILLED: JAN 22/09

DRAWING NUMBER: S08-6783-13

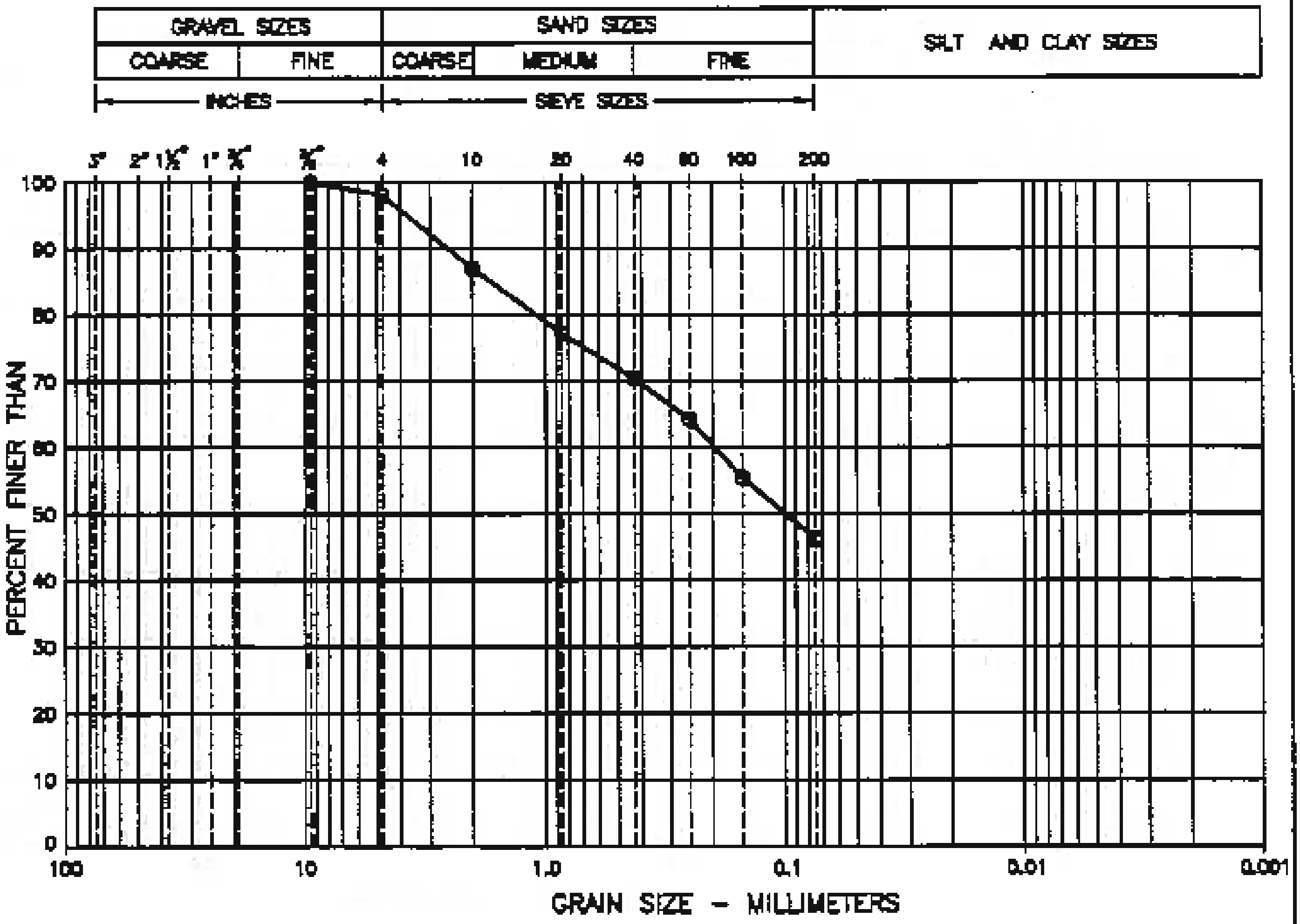
GRAIN SIZE DISTRIBUTION TEST REPORT

Project:	PROPOSED MOWERY RESIDENTIAL SUBDIVISION LA RONGE, SK
Project No.:	S08-6783
Date Tested:	JANUARY 28, 2009
Test Hole No.:	09-1
Sample No.:	10
Depth (m):	1.0
Remarks:	

SIEVE SIZE	PERCENT PASSING
3/8 inch	100.0
No. 4	98.1
No. 10	87.0
No. 20	77.1
No. 40	70.3
No. 80	64.2
No. 100	55.4
No. 200	46.2

Material Description

% Gravel Size	% Sand Size	% Silt and Clay Size
2	52	46



P. MACHIBRODA ENGINEERING LTD.

DRAWING NO.

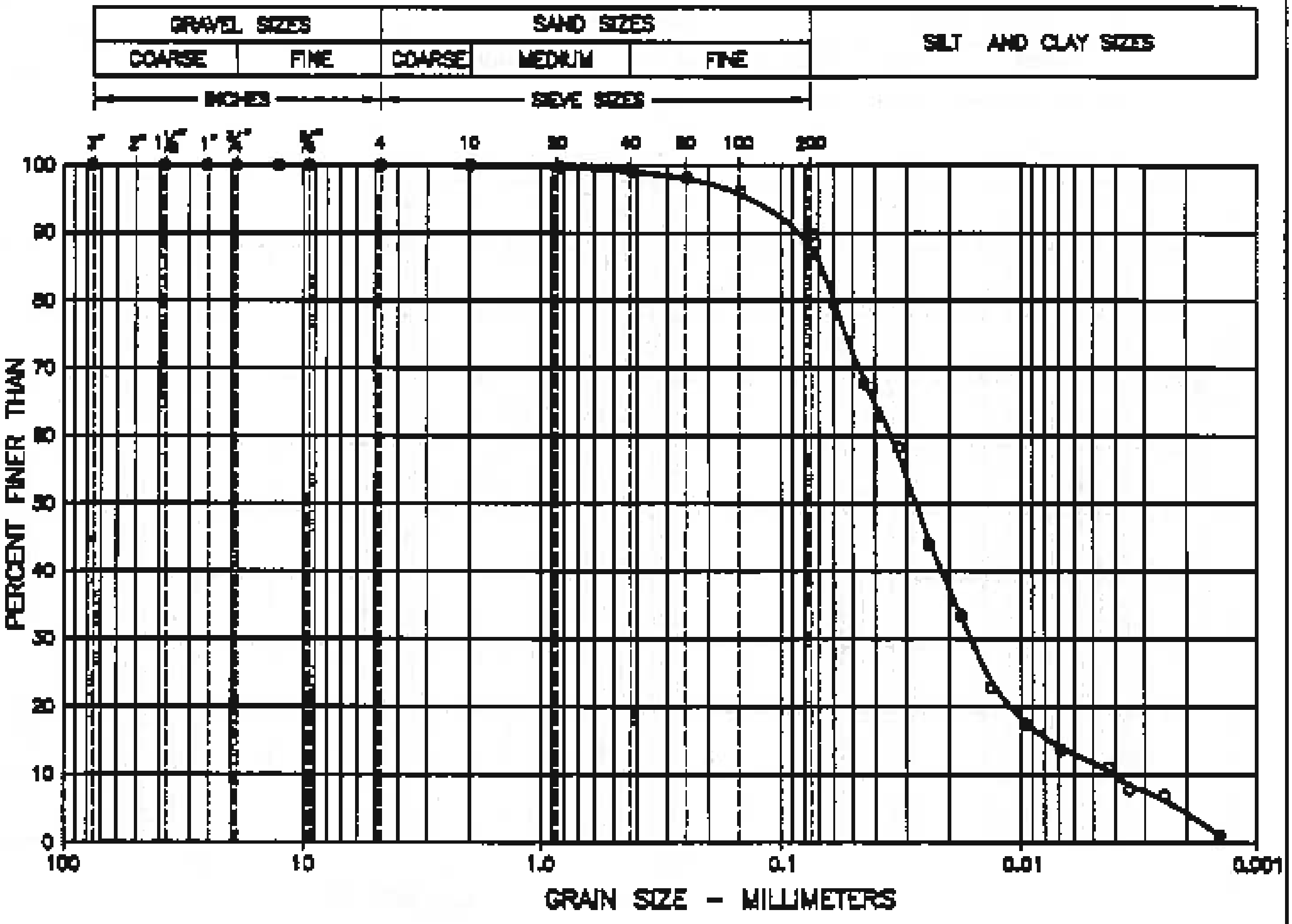
S08-6783-14

GRAIN SIZE DISTRIBUTION TEST REPORT

Project: PROPOSED MOWERY RESIDENTIAL SUBDIVISION
LA RONGE, SK
Project No.: S08-6783
Date Tested: JANUARY 30, 2009
Test Hole No.: 09-2
Sample No.: 3
Depth (m): 1.0
Remarks:

Material Description

% Gravel Size	% Sand Size	% Silt Size	% Clay Size
0	10	86	4



P. MACHIBRODA ENGINEERING LTD.

DRAWING NO.
S08-6783-15

GRAIN SIZE DISTRIBUTION TEST REPORT

Project: PROPOSED MOWERY RESIDENTIAL SUBDIVISION
LA RONGE, SK

Project No.: S08-6783

Date Tested: JANUARY 28, 2009

Test Hole No.: 09-4

Sample No.: 15

Depth (m): 2.0

Remarks:

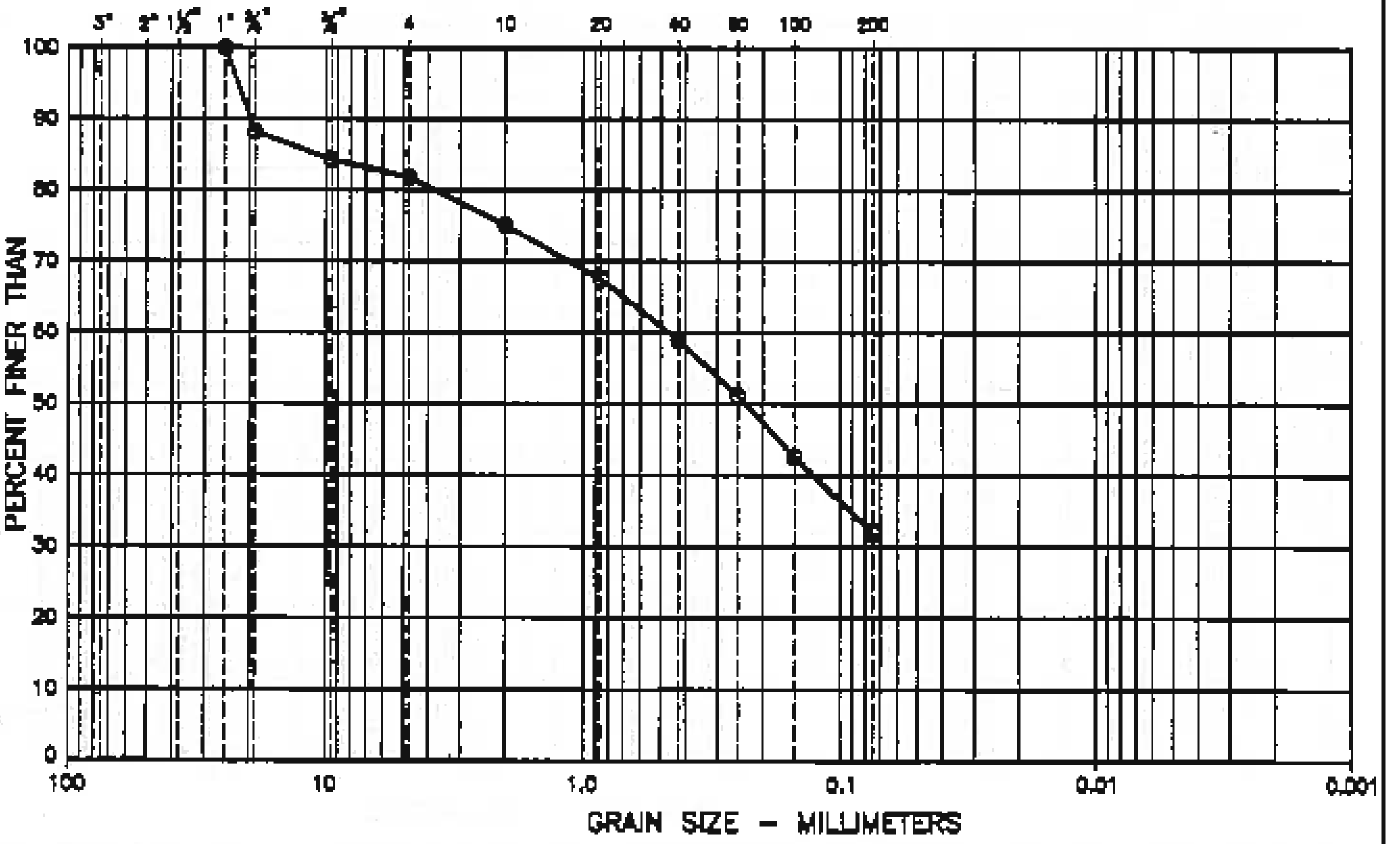
SIEVE SIZE	PERCENT PASSING
1 Inch	100.0
3/4 Inch	88.2
3/8 Inch	84.3
No. 4	81.8
No. 10	75.1
No. 20	67.5
No. 40	58.0
No. 60	51.2
No. 100	42.6
No. 200	32.0

Material Description

% Gravel Sizes	% Sand Sizes	% Silt and Clay Sizes
18	50	32

GRAVEL SIZES		SAND SIZES			SILT AND CLAY SIZES
COARSE	FINE	COARSE	MEDIUM	FINE	

INCHES — SIEVE SIZES



P. MACHIBRODA ENGINEERING LTD.

DRAWING NO.

S08-6783-16

GRAIN SIZE DISTRIBUTION TEST REPORT

Project: PROPOSED MOWERY RESIDENTIAL SUBDIVISION
LA RONGE, SK

Project No.: S08-6783

Date Tested: JANUARY 30, 2009

Test Hole No.: 09-8

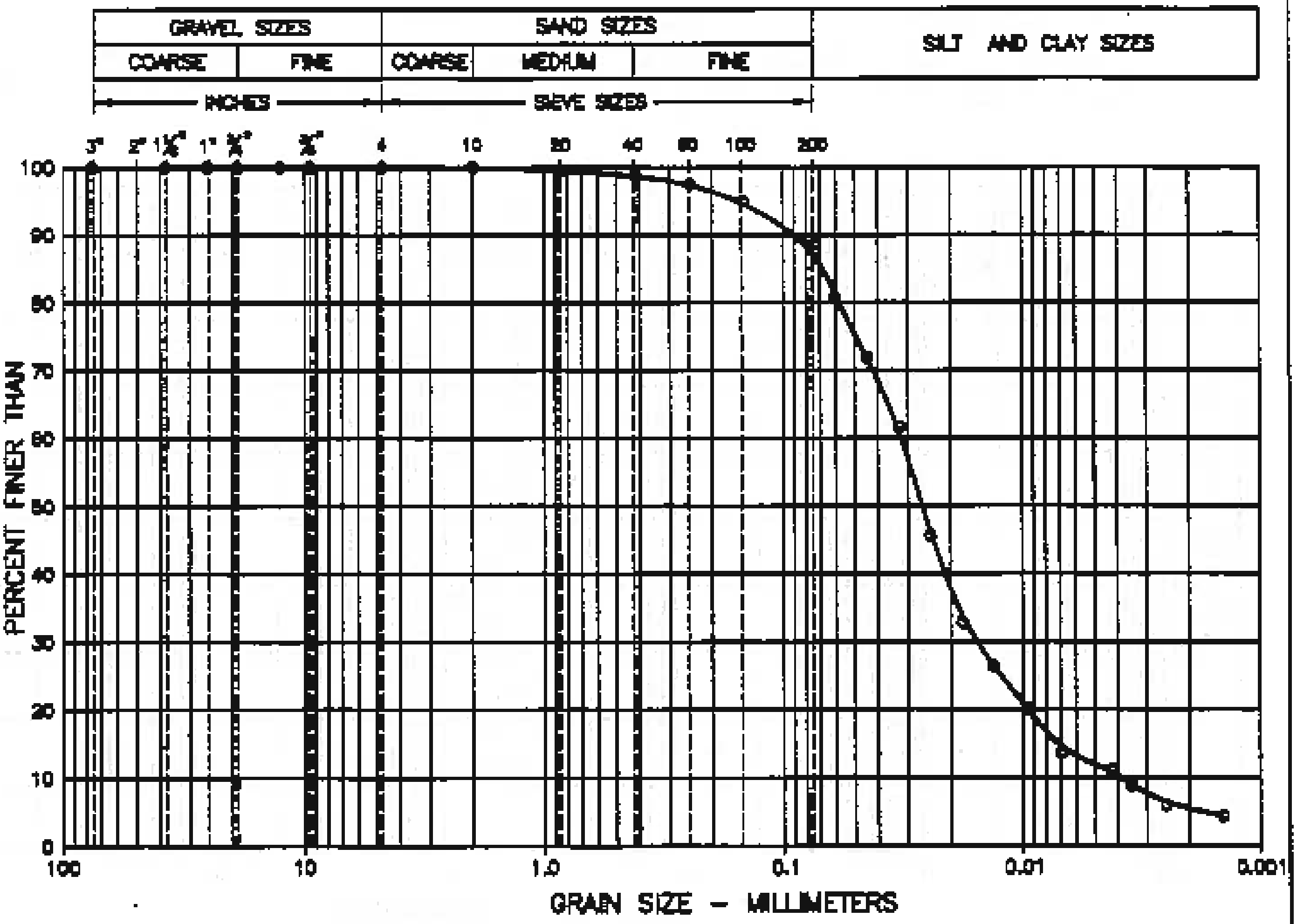
Sample No.: 40

Depth (m): 2.0

Remarks:

Material Description

% Gravel Size	% Sand Size	% Silt Size	% Clay Size
0	11	84	5



P. MACHIBRODA ENGINEERING LTD.

DRAWING NO.

S08-6783-17

Project No. 20084372
File: 20084372.E02
Client: Town of La Ronge
Subject: Field Notes – Mowery Subdivision Test Holes
By: Ryan McDowell
Date: January 29, 2009
Sheet: 1 of 2
Chk'd:

DESIGN NOTES

Test Hole Data:

Test Pit 2 - Ground Elevation: 373.51m
Total Depth: 5.0m
Water Depth: 4.5m
Boulder and Cobbles
See Photos: 150, 151

Test Pit 3 - Ground Elevation: 375.60m
Total Depth: 2.2m
Boulder and Cobbles
Bedrock found at 2.2m
See Photos: 152, 153

Test Pit 4 - Ground Elevation: 374.97m
Total Depth: 5.0m
Boulder and Cobbles

Test Pit 5 - Ground Elevation: 373.29m
Total Depth: 5.0m
Water Depth: 4.5m
Boulder and Cobbles
See Photos: 132, 133

Test Pit 6 - Ground Elevation: 377.24m
Total Depth: 0.0m
Surface Rock

Test Pit 8 - Ground Elevation: 377.24m
Total Depth: 0.0m
Surface Rock

Photos:
IMG_0124: At TH 1 looking SW to TH2
IMG_0131: At TH 4
IMG_0132: TP 5
IMG_0133: TP 5

IMG_0136: At TH 10 Looking N to TH 8
IMG_0137: At TH3 Looking SW to TP 5
IMG_0138: Looking SE to TH 9
IMG_0139: TH 2
IMG_0140: At TH 2 looking NE to TH 1
IMG_0141: Looking SE to TP 4
IMG_0142: In Middle of clearing looking towards intersection of Mowery Cres. and Studer St.
IMG_0143: Looking Opposite direction as IMG_0142
IMG_0146: Looking W to TH 12
IMG_0147: Opposite of IMG_0146 looking towards Studer St.
IMG_0148: Looking at bedrock outcrop S of TH 12
IMG_0149: Looking W to TH 12
IMG_0150: TP 2
IMG_0151: TP 2
IMG_0152: TP 3
IMG_0153: TP 3

C

Appendix C - Sewage Pump Station Upgrade Pre-Design Report, UMA 2002


IMG_0124 – at TH 1 looking SW to TH 2


IMG_0131 – at TH 4


IMG_0132 – TP 5


IMG_0133 – TP 5


IMG_0136 – at TH 10 looking N to TH 8


IMG_0137 – at TH 3 looking SW to TP 5

IMG_0138 – looking SE to TH 9

IMG_0139 – TH 2


IMG_0140 – at TH 2 looking NE to TH 1


IMG_0141 – Looking SE to TP 4

IMG_0142 – In middle of clearing looking toward intersection of Mowery Cres and Studer St.

IMG_0143 – Looking opposite direction as IMG_0142

IMG_0146 – Looking W to TH 12

IMG_0147 – Opposite of IMG_0146 looking towards Studer St.

IMG_0148 – Looking at bedrock outcrop S of TH 12

IMG_0149 – Looking W to TH 12

IMG_0150 – TP 2

IMG_0151 – TP 2


IMG_0152 – TP 3


IMG_0153 – TP 3

file
0602-072-00
(4.6)

TOWN OF LA RONGE
SEWAGE PUMPING STATION ANALYSIS
DRAFT

Prepared for:
Town of La Ronge

Prepared by:
UMA Engineering Ltd.
200-2100 8th Street East
Saskatoon, Saskatchewan
S7H 0V1

UMA Project No. 31-02-0602-065

May 2002

TABLE OF CONTENTS

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1.0 EXECUTIVE SUMMARY

The Sewage Pumping Station Study has been completed, as was recommended in the Saskatchewan Municipal Affairs and Housing (SMACH) Northern Infrastructure Study (NIS). The Study was recommended to provide a comprehensive study of the sewage pumping stations. Statements given in the NIS regarding deficiencies in the sewage pumping stations have been reviewed, including an on site review and a detailed analysis of the specific operating conditions for each of the sewage pumping stations.

Recommendations presented in NIS and this Study include; measures to mitigate groundwater infiltration in the dry pit, improvements to provide adequate ventilation in the wet well and dry pit, inclusion of swab launch facility for force main cleaning, building exterior improvements, replacement of electrical controls and equipment that do not meet current codes, increased capacity of wet wells to improve pumping efficiencies and installation of new pumps for increased capacity.

The only recommendation given in the NIS not being recommended at this time is the installation of standby generators in sewage pumping stations #2, #3, #7 and #8, due to the high cost ($50,000 to $100,000 each). Currently, if there is a power outage water distribution is halted leaving a minimal amount of wastewater flow to the sewage pumping stations. The Town has the capability to distribute water during power outages (standby generator at the water treatment plant) for fire fighting purposes. If the water is turned on during a power outage, there is a potential problem for sewage flooding in basements. Similarly, if the power is out for a very long period of time, there is some potential for ground water infiltration into the sewage system could also cause basement flooding.

SPSs #1, #4, #5, #8 and #9 had varying amounts of groundwater infiltration into the dry pit, from a slight amount to a very significant amount and need attention. Epoxy injection into joints, cracks and holes to inhibit groundwater infiltration should prove to be most effective. A benefit to this method is the positive effects of the grouting are usually known immediately.

There are three sewage pumping stations that do not need any upgrading. These are the senior citizens sewage pumping station, the Police Point sewage pumping station and the sewage pumping station on the east end of La Ronge Avenue. These are all relatively new stations, have had no reported problems and are not expected to experience problems in the near future.

If the timing for the development of a residential subdivision feeding SPS #2 is longer than 5 years, sewage pumping station #2's pumps and electrical controls should not be upgraded until development proceeds. Sewage pumping station #2 has more than adequate capacity for current conditions and is in relatively good shape. Therefore, only minor improvements are recommended.

The recommendations identified above are estimated to cost $772,000. Of this amount, the electrical controls and new pumps for sewage pumping stations #1, #2, #3, #4, #5 and #9 are estimated to cost $455,000, which includes a 10% contingency.

2.0 INTRODUCTION

2.1 Sewage Pumping Stations Review

The 1999 Saskatchewan Municipal Affairs Culture and Housing (SMACH) Northern Infrastructure Study (NIS) for the Northern Town of La Ronge identified that there were several deficiencies with many of the sewage pumping stations (SPSs). The general overall comment given in the NIS was that the SPSs were in relatively good running condition except there are deficiencies that should be continually upgraded over time. It was also recommended that before initiating any major capital expenditures a comprehensive sewer system study be done.

The deficiencies identified are as follows:

- add standby generators in major sewage pumping stations (2,3,7,8) to prevent the dry pits from being flooded by ground water infiltration and resulting in damage occurring to the sewage pumps during power outages
- increase the wet well sewage storage capacity to reduce the number of pumps starts and resulting pump wear
- upgrade wet well and dry pit air ventilation to current standards for safer maintenance conditions
- upgrade pumps so capacity of each pump can handle the peak hour flow to that station
- upgrade electrical components to current regulations including external warning lights and bells
- install swab launch facilities for ease of cleaning the sewage force main
- replace pumps with limited life expectancy with new pumps
- improve building exteriors, roofing and siding

3.0 EXISTING SEWAGE PUMPING STATIONS

3.1 Status of Sewage Pumping Stations in 2002

Each of the sewage pumping stations was reviewed on site on March 13, 2002. This was done to verify the deficiencies as described in the NIS and to get a first hand view of the SPSs to determine the degree of difficulty in carrying out upgrades. Out of the 12 SPSs in La Ronge (there is no SPS #6), there are three that do not need any improvements at this time. These are the senior citizen's SPS, the SPS serving Police Point and the most easterly SPS on La Ronge Avenue. The Police Point and La Ronge Avenue SPSs are only a couple years old and were designed to serve for the next 20 years. The senior citizen's pumping station is also relatively new, operates only approximately 15 hours per year and should serve for many more years. This station has also been relatively trouble free. See Sewage Pumping Station Locations Drawing at the back of the study.

The sewage pumping stations were generally found in good running conditions as reported in the 1999 NIS. There was one significant change in conditions from the 1999 NIS was the run time of sewage pumps in SPS #3. The run time of the pumps had gone up significantly to 14 hours (total time for both pumps) per day. The NIS had reported that the run times for the pumps in SPS #3 in 1999 were 3 hours in the summer and 5.5 hours in the winter. The difference is due to the new water treatment plant (2000) now being on line. The extended hours for the pumps are to predominately handle the backwash water from the new water filters in the water treatment plant. Previously, backwash water was returned directly to the lake.

3.2 Standby Generators

The discussion from the NIS indicated that standby generators should be installed in the major sewage pumping stations (#2, #3, #7 & #8), however this is a costly procedure ($50,000 to $100,000 each). It is recognized that there is a potential flooding and environmental contamination risk. Standby generators should therefore be considered in the next funding program.

Currently, if there is a power outage, water distribution is halted leaving a minimal amount of wastewater flow to the sewage pumping stations. If there is a fire and a power outage, the Town has the capability to distribute water using a standby generator at the water treatment plant. This could create a problem of sewage flooding in basements, if residences started to use water without the sewage pumping stations having power. Similarly, if the power is out for a very long period of time, ground water infiltration into the sewage system could also cause basement flooding.

Another problems is of groundwater infiltration into the dry pit of SPSs #1, #4, #5, #8 and #9. Using epoxy injection into joints, cracks and holes could inhibit the infiltration. A benefit to this method is the positive effects of the grouting are known immediately.

3.3 Overall Deficiencies

An overall list of deficiencies for each of the sewage pumping stations was developed and is summarized in Table 3-1:

SPS Deficiencies
Table 3-1

SPS #	Ground Water Infiltration	Inadequate Ventilation	Require Swab Launch	Rough Building Exterior	Inadequate Electrical Controls	Minimal Wet Well Size	Inefficient or Aged Pumps
1	x	x	x		x	x	x
2		x	x	x	x	x	x
3			x		x		x
4	x	x	x	x	x	x	x
5	x	x	x	x	x		x
7		x	x				
8	x	x	x				
9	x	x	x		x		x
10		x	x				

3.4 Wastewater Flow for Sewage Pumping Stations

Wastewater flow for each the SPSs was calculated from the information given in the NIS and information obtained during the on site review. The results were then checked against water consumption records kept for the water treatment plant, including wastewater from backwashing and daily records from the Sewage Treatment Plant.

The estimated quantity of wastewater going through sewage pumping stations was approximately 10% higher than water consumption records (including backwash). It is known that there is groundwater seepage into the sewer system, but it is assumed that the amount of groundwater entering the sewage system in the summer is equal to or less than treated water that is consumed and does not enter into the sewage system, i.e. watering gardens and lawns in the summer. Also, water seepage into the sewage system in the winter months is limited with the zone of frozen soil near the mains. Therefore, the assumption used in the analysis of the SPSs is wastewater flowing in the sewage mains equals the amount of treated water plus backwash water. Using this analogy, the pumps are only pumping 90% of the rated capacity. This seems reasonable, given the age of most of the pumps.

4.0 EVALUATION AND UPGRADE CRITERIA

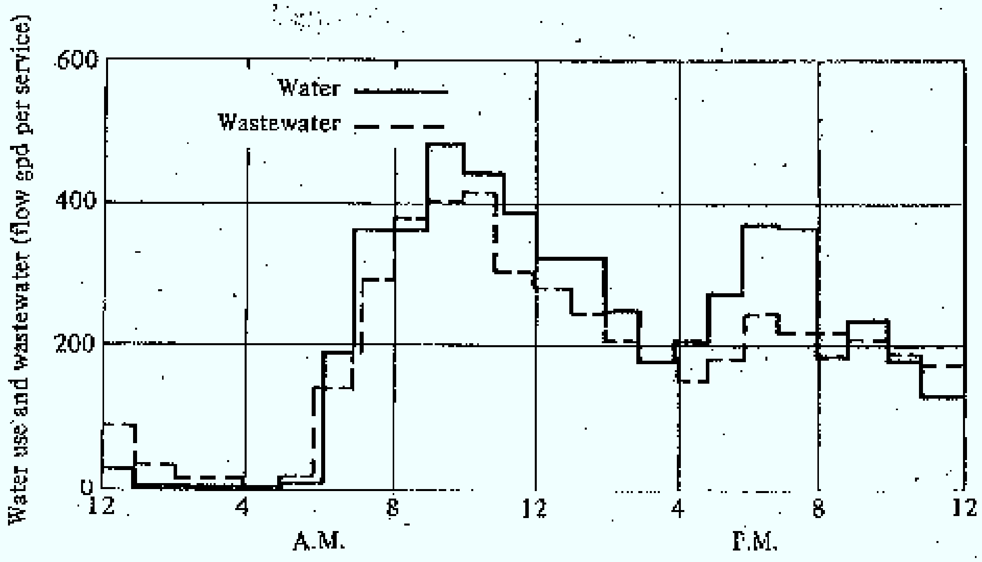
4.1 Sewage Pumping Stations Operating Conditions

The evaluation and upgrade criteria used for the sewage pumping stations is based on the need for the sewage pumping station to be operated in a safe and efficient manner and meet current guidelines. Each of the sewage pumps at a station should be able to handle the peak hour flow, the sewage pumping station should have current and up to date equipment and the operator should have a safe environment to work in.

4.2 Sewage Flow Model

A model for sewage flow is required to estimate the operating conditions of the sewage pumps at each of the sewage pumping stations as compared to the operating condition point of the existing pumps. Understanding that the Northern Town of La Ronge's wastewater is mainly of a domestic nature, it is reasonable to assume that the sewage wastewater flows resemble that of a residential setting. Based on this, a residential model was adopted from the "3rd Edition of Water Supply and Pollution Control" textbook by John W. Clark/Warren Viessman, Jr. Mark J. Hammer (see Figure 4-1, from page 125). It is significant to note that the majority of the wastewater flow takes place in the A.M. and tapers off into the mid afternoon, followed a small peak around 6 to 7 P.M.

Residential Flow
Figure 4-1



Based on this model, the peak hour flow is 2 times the average daily flow, calculated on 18-hour day of water consumption. Using this information, specific peak and average flow rates for each of the sewage pumping station was then calculated (see Table 2.0). This information in turn has been used to complete the analysis of each of the sewage pumping stations pump capacities and need for upgrading.

4.3 Sewage Pump Analysis

4.3.1 Sequence of Wastewater Flow

The sequence of wastewater flow through the Town is as follows:

- SPS #2, #3 & #7 pump directly to the wastewater treatment plant
- SPS #5 pumps to SPS #4
- SPS #4, #9, senior citizen's SPS, Police Point SPS & east La Ronge Avenue SPS all pump to SPS #3, including backwash filter water from the water treatment plant
- SPS #1 & #10 pump to SPS #2
- SPS #8 pump to SPS #7.

4.3.2 Future Wastewater Flow from Residential Subdivisions

All the future (20 years) wastewater flow is expected to come mainly from the proposed residential development area bounded by Studer Street, Bedford Drive and Boardman Street and is anticipated to flow to SPS #2. Beyond this area, the longer term (>20 years) residential development is anticipated to take place to the north of Studer Street and Riese Drive. Wastewater from this area will likely flow to SPS #7. Currently, SPS #2 and #7 both have excess pumping capacity.

4.3.3 Specific Sewage Pumping Station Considerations

SPS #3 was upgrade in 1995 with new larger capacity pumps. The current volume and peak hour flow of sewage at SPS #3 with the operation of the new water treatment plant (2000) is greater than the pump's capacity. There has also been a problem with these pumps. In the past when the pumps ran in parallel one of the pumps shaft would break. This has not happened lately, but nothing in particular has been done to rectify or verify what the problem was either. The pump supplier has suggested using a different impeller (5 vane vs. 2 vane) to remedy the situation, but this has not been tested to date. The 5 vane impeller does not handle as large of solids as the 2 vane. It is likely that if the 5 vane impeller is used, pump plugging will become the next problem to over come. This usually results with the pumps plugging up. In any event, the pumps are undersized.

SPS #5 collects sewage from several buildings from the La Ronge Band First Nation Reserve. SPS #5 subsequently pumps to SPS #4. During the summer there are periods when SPS #5 has been operating at capacity, resulting in SPS #4 not being able to keep up.

SPS #9 was built using many used parts from other sewage pumping stations. The basic building is in fair condition and its pumping capacity is reportedly sufficient. However, the electrical controls and pumps are from other sewage pump stations that were upgraded are considered near the end of their usable cycle.

4.3.4 Sewage Pumping Station Analysis Results

Table 2.0 shows the results of the analysis sewage pumps based on the model adopted:

Sewage Pumping Station Analysis
Table 4-1

Sewage Pumping Station Number	Existing Pump, Hp and Manufac.	'NIS' Reported Pumping Capacity (L/s@m TDH)	Revised Pumping Capacity 90% eff (L/s@m TDH)	Exist Wet Well Size (L)	Recom. Wet Well Size for pump (L)	Average Model Inflow (L/s)	Peak Model Inflow (L/s)	Pump Run/Cycle Times Average Inflow (seconds)	Pump Run/Cycle Times Peak Inflow (seconds)
1	15/Morris	**20.2**/9.1	18.2	850	**3030**	2.5	4.9	"1" **0.9**/6.6 **3.2/23.4**	"2" **1.1**/4.0 **3.8/14.1**
2	30/Morris "3"	**24.0**/15.2 **24.0/25.2**	21.6	3000	4000 **4000**	4.9 **9.8**	9.8 **18.6**	**3.0**/13.2 - / - **4.7/11.5**	**4.2**/9.3 - / - **13.1/16.7**
3	30/Gorman "3"	**31.7**/32.0 **36.4/35.0**	28.5	4600	4755 **5460**	18.2 **18.2**	34.5 **34.5**	**7.4**/11.6 - / - **5.0/10.0**	continuously on for 4 hrs **47.9/50.5**
4	7.5/Morris "3"	**22.0**/7.6 **26.0/8.6**	19.8	1600	3300 **3900**	9.9 **9.9**	19.8 **19.8**	**2.7**/5.4 **5.6**/12.2 **4.0/10.6**	continuously on for 4 hrs **10.5/13.8**
5	5/Morris "3"	6.0/8.0 **10,0/9.0**	5.4	1600	1600 **1600**	2.5 **2.5**	5.0 **5.5**	**9.2**/19.9 - / - **3.6/14.3**	**66.7**/77.4 **5.9/10.8**
7	7.5/Morris	**22.1**/7.3	19.2	2300	3300	9.0	18.0	**3.5**/7.8 **6.0**/11.0	**20.2**/26.0 **29.0**/35.0
8	7.5/Morris	**15.8**/11	14.2	2000	2550	5.2	10.4	**2.2**/8.6 - / -	**8.7**/12.0 - / -
9	5/Chicago Wenco-P.	**6.3**/12.2 (estimated)	5.7	1600	n/a	2.0 (estimated	4.0)	**7.2**/20.5 - / -	**15.7**/22.3 - / -
10	7.5/Morris	**28.4**/5.2	25.6	700	n/a	n/a	n/a		

"1" First set of numbers represent the pump **run/cycle time** for Average Flow into existing SPS wet well
Second set of numbers represent the pump **run/cycle time** for Average Flow into SPS
with an expanded wet well

"2" First set of numbers represent the pump **run/cycle time** for Peak Flow into existing SPS wet well
Second set of number represent the pump **run/cycle time** for Peak Flow into SPS
with an expanded wet well

"3" Represents the recommended larger capacity replacement pump and corresponding data

5.0 RECOMMENDATIONS

The recommended upgrading for existing sewage pumping stations in La Ronge are described below.

5.1 Groundwater Infiltration

Groundwater infiltration is primary a concern at SPS #1, #4, #5 and #9 and epoxy injection is recommended to reduce this infiltration. While the epoxy injection method of sealing manholes is generally successful, factors such as ground movement and condition of manhole exterior surfaces can affect the epoxy's ability to maintain its seal.

5.2 Ventilation Improvements

Ventilation improvements are required to provide adequate air flows to meet current guidelines.

5.3 Swab Launch Facilities

Swab launch facilities are recommended for SPS #1, #2, #3, #4, #5, #7, #8, #9 and #10. Swab launch facilities greatly aid in cleaning of the force main. A clean force main improves the performance of the pumps and subsequently reduces pump maintenance. Energy savings will also be realized with regular force main cleaning with reduced pump run times.

5.4 Building Exteriors

It is recommended that various components of the exteriors of SPS #2, #4 and #5 be fixed so the interiors are kept secure from the outside weather elements.

5.5 Electrical Controls and Equipment

New electrical controls and equipment are recommended for all the sewage pumping stations that are recommended to receive new pumps (SPS #1, #2, #3, #4, #5 and #9). This is recommended not only to upgrade obsolete equipment, but also to ensure there are no warranty conflicts with the operation of the pumps. Parts will also be more readily available for the new electrical equipment.

5.6 Increased Wet Well Capacities

It is recommended that SPSs #1, #2 and #4 have increased wet well capacity. The recommended increased wet well storage volume will result in longer pumping cycles and ultimately reduce pump wear.

5.7 New Pumps for Sewage Pumping Stations

Based on the sewage pump analysis, it is recommended that SPS #1, #3, #4, #5, #9 be fitted with new pumps. This will result in improved pump capacity.

6.0 COST ESTIMATES

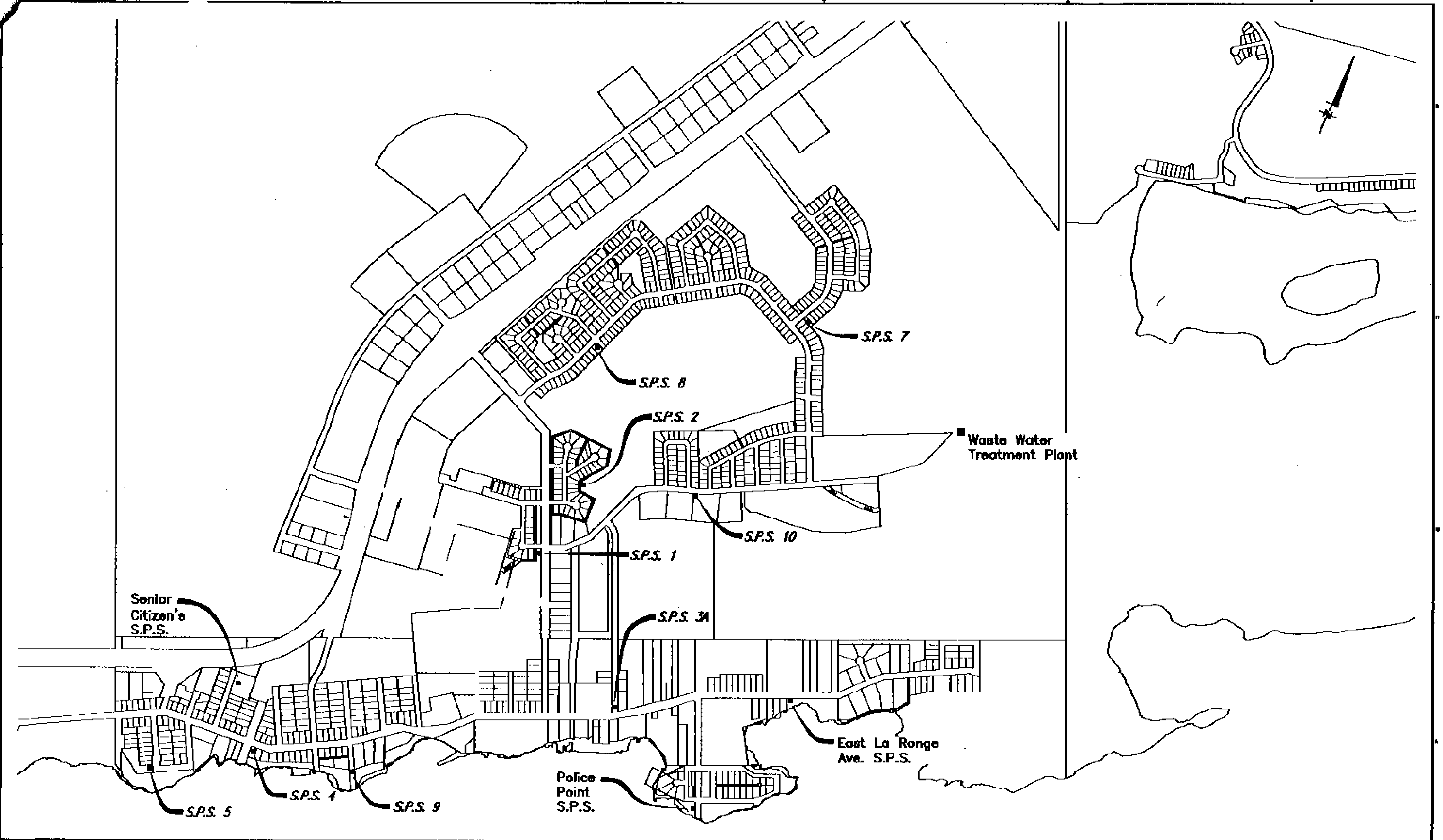
The cost to upgrade the various components of each SPS is given in Table 6-1.

Table 6-1
Cost Estimate for Recommended Upgrades

SPS #	Grouting Ground Water Infiltration	Improved Ventilation	Install Swab Launch Facility	Improve Building Exterior	Provide New Electrical/ Controls	Increase Wet Well Size	New Larger Capacity Pumps	**Total Cost $**
1	8,000	14,000	8,000		28,000	17,000	42,000	**117,000**
2		14,000	8,000	6,000	28,000	17,000	43,000	**116,000**
3			8,000		28,000		48,000	**84,000**
4	8,000	14,000	8,000	1,000	28,000	17,000	41,000	**117,000**
5	8,000	14,000	8,000	6,000	28,000		36,000	**100,000**
7		14,000	8,000					**22,000**
8	8,000	14,000	8,000					**30,000**
9	8,000	14,000	8,000		28,000		36,000	**94,000**
10		14,000	8,000					**22,000**
	40,000	**112,000**	**72,000**	**13,000**	**168,000**	**51,000**	**246,000**	**702,000**

The overall cost to improve the entire system is estimate to cost $702,000, plus a 10% contingency of $70,000 for a total budget of $772,000. Engineering fees and GST are not included in these figures.

DRAWINGS



UMA Engineering Ltd.
Engineers, Planners & Surveyors

Saskatchewan

A.P.E.G.S. Certificate of Authorization No. 210

DRAWN	MDO
CHECKED	KLN
DESIGN	KLN
SCALE	[illegible]
PROJECT ENG.	
APPROVED	

TOWN OF LA RONGE

SEWAGE PUMPING STATION
LOCATION PLAN

DATE	MAY 2002
JOB	[illegible]
REVISION	
SHEET	OF
DWG. NO.	P–1

D

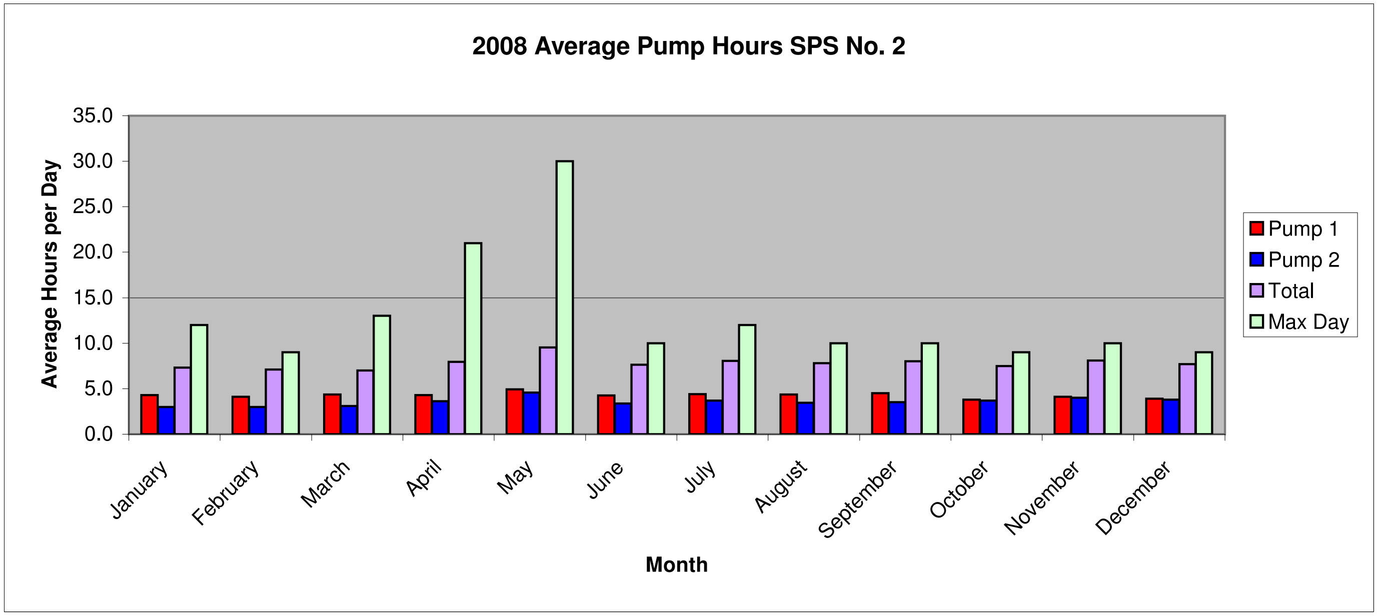
Appendix D - SPS Data Analysis

2008 Pump Hours Analysis for SPS No. 2

Date	Pump 1	Pump 2	Total	Volume (L/day)	Flow (L/s)	Peak Hour		Flow (L/s)
January	**4.3**	**3.0**	**7.3**	**630720.0**	**7.3**	**14.6**	**Max Day**	**12.0**
February	**4.1**	**3.0**	**7.1**	**613737.9**	**7.1**	**14.2**	**Max Day**	**9.0**
March	**4.4**	**3.1**	**7.0**	**604800.0**	**7.0**	**14.0**	**Max day**	**13.0**
April	**4.3**	**3.6**	**7.9**	**685440.0**	**7.9**	**15.9**	**Max day**	**21.0**
May	**4.9**	**4.6**	**9.5**	**822193.5**	**9.5**	**19.0**	**Max day**	**30.0**
June	**4.3**	**3.4**	**7.6**	**659520.0**	**7.6**	**15.3**	**Max day**	**10.0**
July	**4.4**	**3.7**	**8.1**	**696774.2**	**8.1**	**16.1**	**Max day**	**12.0**
August	**4.4**	**3.5**	**7.8**	**674477.4**	**7.8**	**15.6**	**Max day**	**10.0**
September	**4.5**	**3.5**	**8.0**	**694080.0**	**8.0**	**16.1**	**Max day**	**10.0**
October	**3.8**	**3.7**	**7.5**	**646606.5**	**7.5**	**15.0**	**Max day**	**9.0**
November	**4.1**	**4.0**	**8.1**	**699840.0**	**8.1**	**16.2**	**Max day**	**10.0**
December	**3.9**	**3.8**	**7.7**	**666116.1**	**7.7**	**15.4**	**Max day**	**9.0**
Total				**674879**	**7.8**	**15.6**		

Capacity = 24.0 L/s

85th	9.0	
Max Daily	30.0	26-May-08

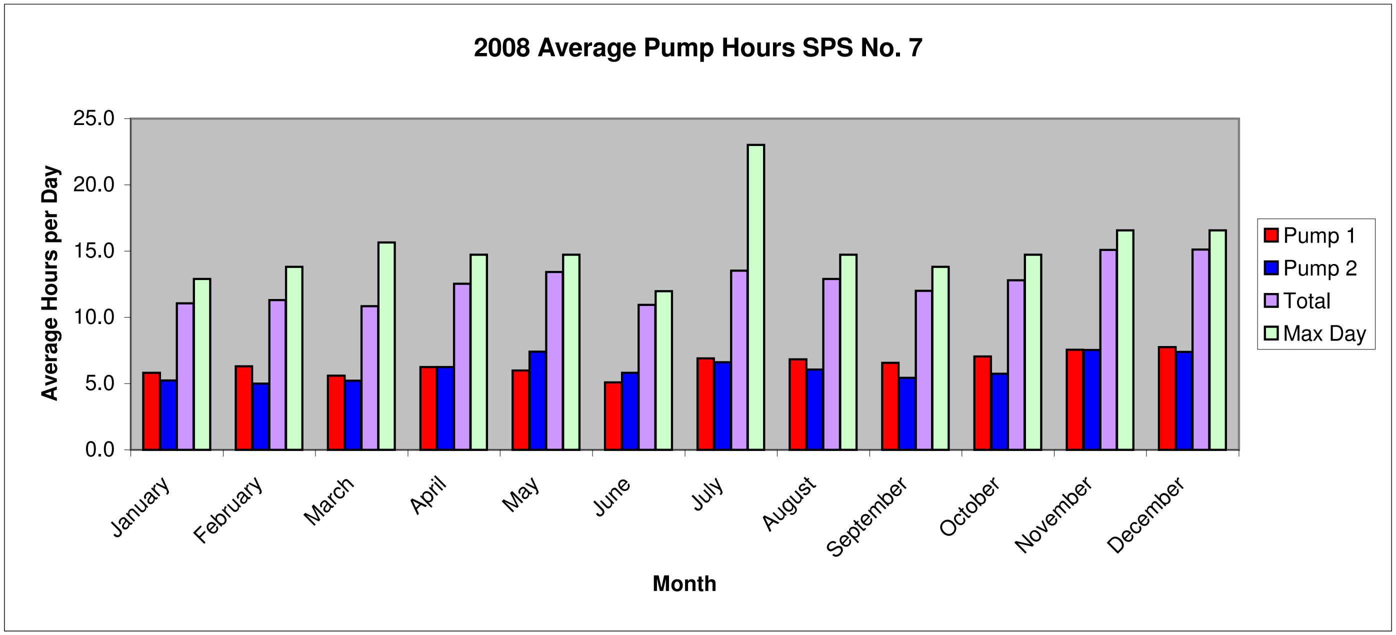


2008 Pump Hours Analysis for SPS No. 7

Date	Pump 1	Pump 2	Total	Volume (L/day)	Flow (L/s)	Peak Hour		Flow (L/s)
January	**5.8**	**5.2**	**11.1**	**880464.0**	**10.2**	**20.4**	**Max Day**	**12.9**
February	**6.3**	**5.0**	**11.3**	**899851.0**	**10.4**	**20.8**	**Max Day**	**13.8**
March	**5.6**	**5.2**	**10.8**	**862327.7**	**10.0**	**20.0**	**Max day**	**15.7**
April	**6.3**	**6.3**	**12.5**	**997152.0**	**11.5**	**23.1**	**Max day**	**14.7**
May	**6.0**	**7.4**	**13.4**	**1067643.9**	**12.4**	**24.7**	**Max day**	**14.7**
June	**5.1**	**5.8**	**10.9**	**869856.0**	**10.1**	**20.1**	**Max day**	**12.0**
July	**6.9**	**6.6**	**13.5**	**1075343.2**	**12.4**	**24.9**	**Max day**	**23.0**
August	**6.8**	**6.1**	**12.9**	**1026580.6**	**11.9**	**23.8**	**Max day**	**14.7**
September	**6.6**	**5.4**	**12.0**	**954720.0**	**11.1**	**22.1**	**Max day**	**13.8**
October	**7.1**	**5.7**	**12.8**	**1018881.3**	**11.8**	**23.6**	**Max day**	**14.7**
November	**7.6**	**7.5**	**15.1**	**1201356.0**	**13.9**	**27.8**	**Max day**	**16.6**
December	**7.7**	**7.4**	**15.1**	**1203665.8**	**13.9**	**27.9**	**Max day**	**16.6**
Total				**1005170**	**11.6**	**23.3**		

Capacity = 22.1 L/s

85th Percentile	13.8	
Max Daily	23.0	5-Jul-08

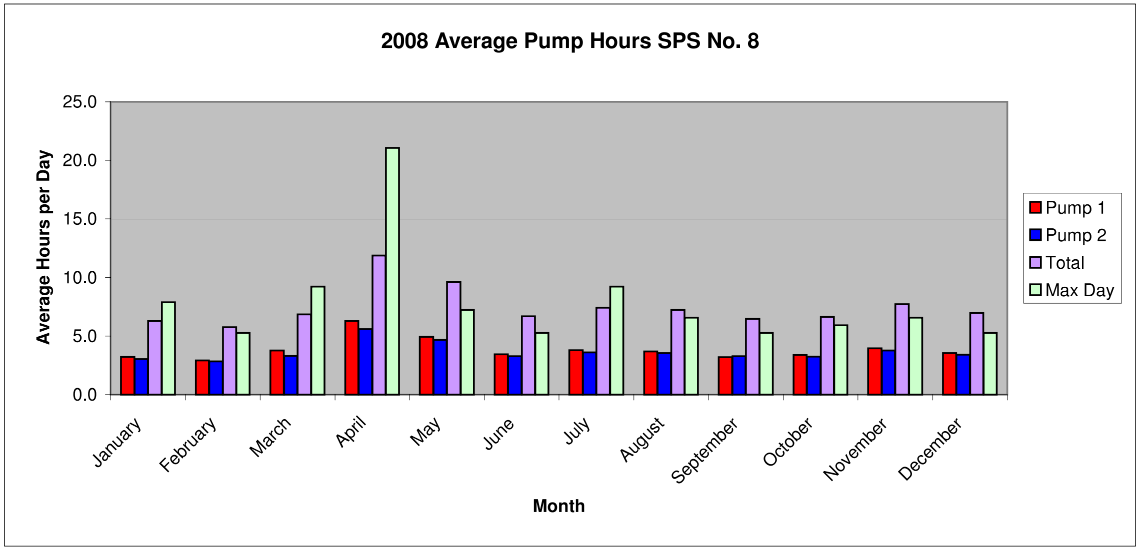


2008 Pump Hours Analysis for SPS No. 8

Date	Pump 1	Pump 2	Total	Volume (L/day)	Flow (L/s)	Peak Hour		Flow (L/s)
January	**3.2**	**3.0**	**6.3**	**356448.0**	**4.1**	**8.3**	**Max Day**	**7.9**
February	**2.9**	**2.8**	**5.8**	**327550.3**	**3.8**	**7.6**	**Max Day**	**5.3**
March	**3.8**	**3.3**	**6.8**	**388985.8**	**4.7**	**9.3**	**Max day**	**9.2**
April	**6.3**	**5.6**	**11.9**	**674976.0**	**7.8**	**15.6**	**Max day**	**21.1**
May	**4.9**	**4.7**	**9.6**	**546781.9**	**6.3**	**12.7**	**Max day**	**7.2**
June	**3.4**	**3.3**	**6.7**	**381096.0**	**4.4**	**8.8**	**Max day**	**5.3**
July	**3.8**	**3.6**	**7.4**	**422012.9**	**4.9**	**9.8**	**Max day**	**9.2**
August	**3.7**	**3.5**	**7.2**	**411003.9**	**4.8**	**9.5**	**Max day**	**6.6**
September	**3.2**	**3.3**	**6.5**	**367824.0**	**4.3**	**8.5**	**Max day**	**5.3**
October	**3.4**	**3.3**	**6.6**	**377976.8**	**4.4**	**8.7**	**Max day**	**5.9**
November	**4.0**	**3.8**	**7.7**	**439872.0**	**5.1**	**10.2**	**Max day**	**6.6**
December	**3.5**	**3.4**	**7.0**	**396325.2**	**4.6**	**9.2**	**Max day**	**5.3**
Total				**424563**	**4.9**	**9.8**		

Capacity = 15.8 L/s

85th Percentile	5.9	
Max daily	21.1	16-Apr-08



E

Appendix E - Cost Estimate

Client: NRSTA
Project: Town of La Ronge - Mowery Subdivision
Project# 20084372
Date 5-May-09
Subject: Phase 1 - Option 1 - 25 lots

Created by: M. Wilson
Checked by: D. Thomson

Item	Description	Unit	Price	Quantity	Extension
1	**Roadwork and Site Preparation**				
	Clearing and Grubbing	ha	$10,000.00	3.5	$35,000
	Common Fill (1.0m on lots)	m^3	$10.00	15,000	$150,000
	Topsoil Remove and Dispose (top 0.4m)	m^3	$12.00	5,600	$67,000
	Asphalt (100 mm)	m^2	$25.00	3,800	$95,000
	Subgrade prep	m^2	$4.00	3,800	$15,000
	Subbase (250mm)	m^2	$30.00	3,800	$114,000
	Base (150mm)	m^2	$45.00	3,800	$171,000
	Curb and Gutter (includes sidewalk)	lin.m	$175.00	650	$114,000
	Bedrock (blasting and removal)	m^3	$340.00	200	$68,000
	Drainage	lin.m	$175.00	30	$5,000
Total 1	**TOTAL ROADWORK AND SITE PREPARATION**				**$834,000**
2	**Sanitary Sewer**				
	Excavation, Main Trench	lin.m	$220.00	325	$72,000
	200mm Dia PVC SDR35 Sewer Main Pipe	lin.m	$60.00	250	$15,000
	200mm Dia. PVC SDR35 Insulated Sewer Pipe	lin.m	$170.00	75	$13,000
	Manhole Base, Frame and Cover	ea	$2,500.00	4	$10,000
	Sanitary tie-in	ea	$5,000.00	1	$5,000
	Video Inspection	lin.m	$15.00	325	$5,000
Total 2	**TOTAL SANITARY SEWER**				**$120,000**
3	**Water Distribution**				
	Excavaton, Double Water Main in Single Trench	lin.m	$175.00	325	$57,000
	150 mm PVC C900 Water Main	lin.m	$70.00	650	$46,000
	150 mm Gate Valve and Box	ea	$2,000.00	8	$16,000
	150 mm Dia. Hydrant	ea	$5,000.00	4	$20,000
	Tie-in to existing	ea	$500.00	2	$1,000
Total 3	**TOTAL WATER DISTRIBUTION**				**$140,000**
4	**Building Services**				
	25 mm Dia. Water Service Pipe	lin.m	$30.00	250	$8,000
	25 mm Corporation Stop, direct tap	ea	$150.00	25	$4,000
	25 mm Curb Stop, Box and S.S. Stem	ea	$300.00	25	$8,000
	100mm SDR 28 Sewer Service Pipe	lin.m	$30.00	250	$8,000
	200mm x 100mm Service Saddle	ea	$50.00	25	$1,000
	Building Services in Common Trench	m	$200.00	250	$50,000
Total 4	**BUILDING SERVICES**				**$79,000**
5	**Construction Sub Total**				**$1,173,000**
6	**Contingency (10%)**				$235,000
7	**Engineering Allowance**	LS			$190,000
8	**Legal Survey Allowance**	ea	$700.00	25	$17,500
9	**Shallow Utility Allowance (SaskTel, SaskPower, SaskEnergy) @ $2,000/lot**	ea	$2,000.00	25	$50,000
10	**Total Estimate**				**$1,665,500**

No Lots	25
Cost per lot	$67,000

Notes

1. Actual construction costs may vary.

Client: NRSTA
Project: Town of La Ronge - Mowery Subdivision
Project# 20084372
Date 5-May-09
Subject: Phase 1 - Option 2 - 48 lots (minimum)

Created by: M. Wilson
Checked by: D. Thomson

Item	Description	Unit	Price	Quantity	Extension
1	**Roadwork and Site Preparation**				
	Clearing and Grubbing	ha	$10,000.00	3.53	$35,000
	Common Fill (0.4m on lots)	m^3	$10.00	11,000	$110,000
	Topsoil Remove and Dispose (top 0.2m)	m^3	$12.00	5,600	$67,000
	Asphalt (100 mm)	m^2	$25.00	5,400	$135,000
	Subgrade prep	m^2	$4.00	5,400	$22,000
	Subbase (250mm)	m^2	$30.00	5,400	$162,000
	Base (150mm)	m^2	$45.00	5,400	$243,000
	Curb and Gutter (includes sidewalk)	lin.m	$175.00	1,100	$193,000
	Bedrock (blasting and removal)	m^3	$340.00	100	$34,000
	Drainage	lin.m	$175.00	60	$11,000
Total 1	**TOTAL ROADWORK AND SITE PREPARATION**				**$1,012,000**
2	**Sanitary Sewer**				
	Excavation, Main Trench	lin.m	$220.00	425	$94,000
	200mm Dia PVC SDR35 Sewer Main Pipe	lin.m	$60.00	350	$21,000
	200mm Dia. PVC SDR35 Insulated Sewer Pipe	lin.m	$170.00	75	$13,000
	Manhole Base, Frame and Cover	ea	$2,500.00	6	$15,000
	Sanitary tie-in	ea	$5,000.00	1	$5,000
	Video Inspection	lin.m	$15.00	425	$6,000
Total 2	**TOTAL SANITARY SEWER**				**$154,000**
3	**Water Distribution**				
	Excavaton, Double Water Main in Single Trench	lin.m	$175.00	575	$101,000
	150 mm PVC C900 Water Main	lin.m	$70.00	700	$49,000
	150 mm Gate Valve and Box	ea	$2,000.00	4	$8,000
	150 mm Dia. Hydrant	ea	$5,000.00	4	$20,000
	Tie-in to existing	ea	$500.00	2	$1,000
	Modification to existing Reservoir No. 3 supply and return line @ Boardman Dr.	ea	$100,000.00	1	$100,000
Total 3	**TOTAL WATER DISTRIBUTION**				**$279,000**
4	**Building Services**				
	25 mm Dia. Water Service Pipe	lin.m	$30.00	480	$14,000
	25 mm Corporation Stop, direct tap	ea	$150.00	48	$7,000
	25 mm Curb Stop, Box and S.S. Stem	ea	$300.00	48	$14,000
	100mm SDR 28 Sewer Service Pipe	lin.m	$30.00	480	$14,000
	200mm x 100mm Service Saddle	ea	$50.00	48	$2,000
	Building Services in Common Trench	m	$200.00	480	$96,000
Total 4	**BUILDING SERVICES**				**$147,000**
5	**Construction Sub Total**				**$1,592,000**
6	**Contingency (20%)**				$318,000
7	**Engineering Allowance**	LS			$258,000
8	**Legal Survey Allowance**	ea	$700.00	48	$33,600
9	**Shallow Utility Allowance (SaskTel, SaskPower, SaskEnergy) @ $2,000/lot**	ea	$2,000.00	48	$96,000
10	**Total Estimate**				**$2,297,600**

No Lots	48
Cost per lot	$48,000

Notes

1. Actual construction costs may vary.

Client: NRSTA
Project: Town of La Ronge - Mowery Subdivision
Project# 20084372
Date 5-May-09
Subject: Phase 1 - Option 2 - 32 lots (minimum)

Created by: M. Wilson
Checked by: D. Thomson

Item	Description	Unit	Price	Quantity	Extension
1	**Roadwork and Site Preparation**				
	Clearing and Grubbing	ha	$10,000.00	2.29	$23,000
	Common Fill (0.4m on lots)	m^3	$10.00	7,000	$70,000
	Topsoil Remove and Dispose (top 0.2m)	m^3	$12.00	3,500	$42,000
	Asphalt (100 mm)	m^2	$25.00	4,200	$105,000
	Subgrade prep	m^2	$4.00	4,200	$17,000
	Subbase (250mm)	m^2	$30.00	4,200	$126,000
	Base (150mm)	m^2	$45.00	4,200	$189,000
	Curb and Gutter (includes sidewalk)	lin.m	$175.00	600	$105,000
	Bedrock (blasting and removal)	m^3	$340.00	100	$34,000
	Drainage	lin.m	$175.00	40	$7,000
Total 1	**TOTAL ROADWORK AND SITE PREPARATION**				**$718,000**
2	**Sanitary Sewer**				
	Excavation, Main Trench	lin.m	$220.00	300	$66,000
	200mm Dia PVC SDR35 Sewer Main Pipe	lin.m	$60.00	225	$14,000
	200mm Dia. PVC SDR35 Insulated Sewer Pipe	lin.m	$170.00	75	$13,000
	Manhole Base, Frame and Cover	ea	$2,500.00	4	$10,000
	Sanitary tie-in	ea	$5,000.00	1	$5,000
	Video Inspection	lin.m	$15.00	300	$5,000
Total 2	**TOTAL SANITARY SEWER**				**$113,000**
3	**Water Distribution**				
	Excavaton, Double Water Main in Single Trench	lin.m	$175.00	450	$79,000
	150 mm PVC C900 Water Main	lin.m	$70.00	450	$32,000
	150 mm Gate Valve and Box	ea	$2,000.00	4	$8,000
	150 mm Dia. Hydrant	ea	$5,000.00	4	$20,000
	Tie-in to existing	ea	$500.00	2	$1,000
	Modification to existing Reservoir No. 3 supply and return line @ Boardman Dr.	ea	$100,000.00	1	$100,000
Total 3	**TOTAL WATER DISTRIBUTION**				**$240,000**
4	**Building Services**				
	25 mm Dia. Water Service Pipe	lin.m	$30.00	320	$10,000
	25 mm Corporation Stop, direct tap	ea	$150.00	32	$5,000
	25 mm Curb Stop, Box and S.S. Stem	ea	$300.00	32	$10,000
	100mm SDR 28 Sewer Service Pipe	lin.m	$30.00	320	$10,000
	200mm x 100mm Service Saddle	ea	$50.00	32	$2,000
	Building Services in Common Trench	m	$200.00	320	$64,000
Total 4	**BUILDING SERVICES**				**$101,000**
5	**Construction Sub Total**				**$1,172,000**
6	**Contingency (20%)**				$234,000
7	**Engineering Allowance**	LS			$190,000
8	**Legal Survey Allowance**	ea	$700.00	32	$22,400
9	**Shallow Utility Allowance (SaskTel, SaskPower, SaskEnergy) @ $2,000/lot**	ea	$2,000.00	32	$64,000
10	**Total Estimate**				**$1,682,400**

No Lots	32
Cost per lot	$53,000

Notes

1. Actual construction costs may vary.