Updated Assessment Report

Sault Ste. Marie Region
Source Protection Area

CHAPTER 2c

TIER 3 WATER BUDGET

With Support Provided by

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Prepared as per Ontario Regulation 287/07, *Clean Water Act, 2006*

ASSESSMENT REPORT
TIER 3 WATER BUDGET

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List of Acronyms

cm	centimetre
d	day
GIS	Geographic Information System
GW	Groundwater
IPZ	Intake Protection Zone
km	kilometre
km2	square kilometre
m	metre
mm	millimetre
m3/s	cubic metres per second
m3/d	cubic metres per day
MECP	Ministry of the Environment, Conservation and Parks (formerly MOE and MOECC)
MNR	Ministry of Natural Resources and Forestry (formerly MNR or OMNR)
MODFLOW	A Three-Dimensional Finite-Difference Ground-Water Flow Model
DEM	Digital Elevation Model
PTTW	Permit To Take Water
SPA	Source Protection Authority
SPC	Source Protection Committee
SSMR SPA	Sault Ste. Marie Region Source Protection Area
SSMRCA	Sault Ste. Marie Region Conservation Authority
SWP	Source Water Protection
WWTP	Wastewater Treatment Plant

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BACKGROUND

The Sault Ste. Marie Region (SSMR) Source Protection Area is situated within the District of Algoma, along the north shore of the St. Marys River and eastern shore of Lake Superior. The SSMR Source Protection Area has a variety of groundwater users; their takings are for domestic, commercial and industrial purposes. However, the most significant use of groundwater in the central and east basin is for municipal drinking water supply. The groundwater system in the SSMR Source Protection Area is divided into West, Central and East Basins. The analyses conducted for the Tier 1/Tier 2 Water Budget study indicate that water takings from the West Basin range from 2% to 3% of the water supply; following the guidance documents, this is interpreted as a low stress on the groundwater system. The water takings from the Central Basin are predicted to range from 23% to 25% of the available supply, and the water takings from the East Basin are predicted to range from 19% to 23% of the water supply. The Central and East Basins are interpreted to have the potential for moderate stress.

As such a Tier 3 Water Budget and Local Area Risk Assessment is mandated to assess the likelihood that the municipality will be able to sustain its allocated (existing, existing plus committed or planned) water supply and to identify threats to the drinking water supply that may influence the municipality's ability to meet its allocated pumping rates. This report provides a summary of the refined conceptual understanding which will serve as a basis for the Tier 3 groundwater and surface water models and builds upon the previous studies.

SURFACE WATER / GROUNDWATER SYSTEM

The SSMR Source Protection Area consists of two distinct landforms. The northern portion is referred to as "Precambrian uplands". South of this region is the relatively flat lying area referred to as the "lowlands". Drainage is via a series of streams flowing southward off the Precambrian uplands, across the lowlands to the St. Marys River.

The runoff generated on the Precambrian uplands area flows downgradient through the streams, overland, or through the shallow soils in a southerly direction and continues into the lowlands. Some of the flow infiltrates into the groundwater system through the thick sand and gravel beach deposits located along the southern edge of the Precambrian uplands.

The conceptual groundwater model shows no mapped aquifers in the Precambrian uplands. Based on the bedrock topography, the groundwater system in the lowlands is divided into three major hydrogeological units, the "West Basin", "Central Basin", and "East Basin". The stratigraphic sequence in all three hydrogeological units in the lowlands area is comprised of a relatively thick clay-rich overburden consisting of glaciolacustrine clays underlain by a layer of coarse-grained glaciolacustrine overburden deposits and the Jacobsville Formation. The deep sand and gravel

aquifer is interconnected with the sandstone aquifer, and forms the regional aquifer formation which supplies the municipal wells and other private wells in the Sault Ste. Marie Area. The sandstone aquifer is confined by the fine-grained glaciolacustrine silt and clay deposits in the lowlands area. The groundwater basins are physically separated by bedrock ridges.

MUNICIPAL WATER SYSTEM

The municipal water supply is managed by the Sault Ste. Marie Public Utilities Commission (PUC). It includes one surface intake at Gros Cap and six municipal wells. The six municipal wells obtain water from the Jacobsville Formation in the Central Basin and overlying sand and gravel unit in the East Basin. There are two (2) wells at the Goulais Well Site and one (1) well at the Steelton Well Site located in the Central Basin. In the East Basin, there are two (2) wells at the Lorna Well Site and one (1) well at the Shannon Well Site.

Lake Superior is a part of the Great Lakes system and is not a part of the watershed. As per the Technical Rules: Assessment Report, water takings from the Great Lakes are not to be considered as part of the water budget at this time.

MUNICIPAL WATER DEMAND

The existing demand is defined as the average pumping during the study period. The PUC maintains pumping records for the water takings. An assessment of pumping rates was conducted on water takings between 2000 and 2012 to represent the existing demand.

The PUC does not have committed or planned demands (as defined in the Technical Rules) for additional groundwater supply as a result of population growth or other new customers, recognized through the City of Sault Ste. Marie Official Plan. However, plans for future development were obtained from the Planning Division of the City of Sault Ste. Marie Engineering and Planning Department for urban expansion plans for the City.

It was assumed that future residential developments would be serviced by the groundwater system alone as a worst case scenario. As such, future water demand for the municipal wells was estimated as the sum of the existing demand and the demand from future residential developments which are approved or whose approvals are pending.

SAFE ADDITIONAL DRAWDOWN

To determine the long term sustainability of the groundwater supply system, each well was assessed to determine the safe water level and potential average safe additional available drawdown which could be achieved. The safe water levels are determined based on the physical and mechanical characteristic of each well. The top of the aquifer was established as the safe water level for each municipal well.

Based on observed water level trends and historical pumping records, the Goulais Wells, Shannon Well and Lorna Wells all demonstrate a comfortable Average Safe Additional Available

Drawdown value which ranges from 40 m to 62 m. The Average Safe Additional Available Drawdown at the Steelton Well is approximately 15 m.

Average pumping results in water levels which are typically above the top of the aquifer. However, there have been instances where water levels have fallen below the safe water level. Future pumping scenarios will need to ensure pumping rates do not draw down the water level past the safe water level.

The assessment report was originally developed under the 2008, 2009 and 2013 versions of the Technical Rules and where updates were made, they were carried out under amendments to the 2017 Rules and 2018 addition of pipelines circumstances to the Table of Drinking Water Threats.

1.0 INTRODUCTION

The *Clean Water Act (2006)* was established to provide a framework for the protection of existing and future municipal drinking water supplies in the Province of Ontario. Source Protection Authorities were established to oversee the preparation of a series of technical studies in accordance with *Ontario Regulation 287/07* and the Technical Rules: Assessment Reports (MOE, 2009). As part of the Assessment Reports, water budget analyses are to be conducted to evaluate the risks to the water quantity within a tiered framework.

The Sault Ste. Marie Region (SSMR) Source Protection Area is situated within the District of Algoma, along the north shore of the St. Marys River and eastern shore of Lake Superior. To meet the requirements of the *Clean Water Act*, the Sault Ste. Marie Region Source Protection Authority completed a Conceptual Understanding Report (Kresin Engineering and MacViro, 2006), a Tier 1/Tier 2 Water Budget (Kresin Engineering and GENIVAR, 2008) and a Data Gap Analysis and Hydraulic Testing for a Tier 3 Risk Assessment – East and Central Basins, Sault Ste. Marie, Ontario (Kresin Engineering and Cole Engineering Group Ltd. (CEG), 2012).

The Conceptual Understanding Report provides a basic understanding of the SSMR Source Protection Area. The Tier 1/Tier 2 Water Budget Report quantifies the movement of water within the various elements that constitutes the hydrologic cycle and identifies areas that may have the potential to be stressed from a water quantity perspective.

The SSMR Source Protection Area has a variety of groundwater users; their takings are for domestic, commercial and industrial purposes. However, the most significant use of groundwater in the central and east basin is for municipal drinking water supply. The groundwater system in the SSMR Source Protection Area is divided into West, Central and East Basins (Map 1-1). The analyses conducted for the Tier 1/Tier 2 Water Budget study indicate that water takings from the West Basin range from 2% to 3% of the water supply; following the guidance documents, this is interpreted as a low stress on the groundwater system. The water takings from the Central Basin are predicted to range from 23% to 25% of the available supply, and the water takings from the East Basin are predicted to range from 19% to 23% of the water supply. The Central and East Basins are interpreted to have the potential for moderate stress.

As per the Technical Rules, the Central and East Basins will require a Tier 3 Water Budget and Local Area Risk Assessment to assess the likelihood that the municipality will be able to sustain its allocated (existing, existing plus committed or planned) water supply and to identify threats to the drinking water supply that may influence the municipality's ability to meet its allocated pumping rates.

This report provides a summary of the refined conceptual understanding which will serve as a basis for the Tier 3 groundwater and surface water models and discusses the local area risk associated with current and planned water demands.

1.1 STUDY TEAM

The project team was directed by a team comprising members of the following organizations:

- Ministry of Natural Resources;
- Sault Ste. Marie Conservation Authority;
- City of Sault Ste. Marie, Public Utility Commission;
- Breen GeoScience Management Inc.; and
- Golder Associates.

The project team responsible for the technical preparation of the Tier 3 study includes:

- Kresin Engineering Corporation (Project Manager);
- Cole Engineering Group Ltd. (Technical Lead);
- S.S. Papadopulos & Associates, Inc. (Hydrogeology – Modelling direction);
- AHYDTECH Associates (Hydrology – Surface Water Modelling);
- Schroeter & Associates (Hydrology – GAWSER guidance).

1.2 REGULATORY FRAMEWORK

Water budgets characterize the pathways of water movement within the hydrologic system. The Water Budget Framework as applied by the *Clean Water Act* involves four stages of evaluation:

- Conceptual Understanding;
- Tier One;
- Tier Two; and
- Tier Three.

With each stage improving the understanding of the water budget through refining the spatial scale and increasing the model and technical complexity.

The first stage involves the development of a Conceptual Water Budget. This involves the collection and review of available baseline data and mapping and the analysis of the compiled information linking physiography, geology, surface water, groundwater, climate, land cover, and water taking from a watershed scale.

The Tier One and Tier Two Subwatershed Stress Assessments estimate the potential for hydrologic stress within a subwatershed. The subwatershed stress assessment is dependent on hydrologic parameters estimated in the water budget. Typically, the Tier Two Water Budget confirms the stress assessment established in the Tier One stage through the application of numerical models.

A Tier Three Water Budget and Local Area Risk Assessment is undertaken for Tier 2 subwatersheds that have been assigned a moderate or significant water quantity stress level. The

objective of the study is to estimate the likelihood that a municipality will be able to meet future water quantity requirements. A Tier Three Water Budget uses numerical groundwater and surface water models, which are integrated, for the assessment of the local area of a municipal wellhead. The Tier Three models consider the development of transient simulations and continuous groundwater and surface water models.

1.3 STUDY AREA

The SSMR Source Protection Area delineated in Map 1-1 is situated within the District of Algoma, along the north shore of the St. Marys River and the eastern shore of Lake Superior. The SSMR Source Protection Area encompasses the City of Sault Ste. Marie and the Township of Prince and includes portions of the Townships of Dennis, Pennefather, Aweres, Jarvis and Duncan as well as areas of the Garden River and Batchewana First Nation Reservations. Both Lake Superior and the St. Marys River are shared resources of Canada and the United States. The boundary of the planning region extends to the international border to the south. The land-based area of the planning region is 521 km2.

The SSMR Source Protection Area consists of two distinct landforms; the northern portion, which is referred to as the “Precambrian uplands”, and an area south of this region, which is relatively flat and is referred to as the “lowlands”. There are no mapped aquifers in the Precambrian uplands and groundwater takings are negligible. The groundwater system in the lowlands is divided into three major groundwater aquifers, the “West Basin”, “Central Basin”, and “East Basin”. The stratigraphic sequence in the lowlands area consists of a relatively thick clay-rich overburden unit overlying a layer of coarse-grained glaciolacustrine overburden deposits and the Jacobsville Formation, a regional sandstone bedrock aquifer.

The surface water system includes 12 subwatersheds of which 10 are associated with major creek systems, one discharges directly into Lake Superior and one, associated with an unnamed river, discharges directly to St. Marys River. The major creeks flow across both the Precambrian uplands and the lowlands.

1.4 METHODOLOGY

The approach used to meet the requirements of the Tier Three Water Budget and Local Area Risk Assessment adheres to the Water Budget & Water Quantity Risk Assessment Guide prepared by the Ontario Ministry of Natural Resources (MNR) and Ontario Ministry of the Environment (MOE), dated 2011 and is described as follows.

1. Develop the Tier Three water budget models. The surface water and groundwater models should be based on conceptual models representing detailed conditions around wells and intakes. The models should be calibrated to represent typical operating conditions under average and variable climate conditions.

2. Characterize municipal wells and intakes. The Tier Three assessment requires a detailed characterization of wells and intakes, specifically identifying the low water operating constraints of those wells and intakes.
3. Estimate allocated quantity of water. This task compiles and describes existing, committed, and planned pumping rates for municipal wells.
4. Identify and characterize drinking water quantity threats. Drinking water quantity threats should include municipal and non-municipal consumptive water demands as well as reductions to groundwater recharge.
5. Characterize projected land use. An evaluation of the potential impact of projected land use changes on water supplies should be included; this involves a comparison of official plans with current land use and incorporates assumptions relating to imperviousness of future developments.
6. Characterize other water uses. Identification of other water uses (e.g., provincially significant wetlands) that might be influenced by municipal pumping and their water quantity constraints.
7. Delineate vulnerable areas. The groundwater quantity vulnerable areas, WHPA Q1 and WHPA Q2, should be delineated using the Tier Three water budget models. WHPA Q1 is delineated by computing the drawdown cone for the municipal wells with existing plus committed plus planned rates. WHPA Q2 identifies additional areas, over those within the WHPA Q1, where recharge reductions in those areas results in a measurable impact to water levels at municipal wells.
8. Evaluate risk scenarios. These scenarios consider the allocated quantity of water for each well, average and drought conditions, and projected land use. The scenarios should be evaluated both in terms of the ability to pump water at each well and, where required, the impact to other water uses.
9. Assign risk level. A risk ranking (low, moderate, or significant) should be assigned to each of the vulnerable areas based on the results of the risk scenarios. An uncertainty level (high, low) must accompany each risk ranking.
10. Identify drinking water quantity threats and areas where they are significant and moderate. Drinking water quantity threats, such as consumptive uses or reductions in recharge, within the vulnerable areas must be identified.

This report Tier 3 - Conceptual Understanding Report focuses on addressing items 1 to 6. The Tier 3 - Local Area Risk Assessment addresses items 7 to 10 and is provided under separate cover.

2.0 BACKGROUND INFORMATION AND DATA COLLECTION

In the SSMR Source Protection Area, both surface water and groundwater resources play an important role in the water budget. The surface water system is the main input to the groundwater system and therefore establishes an integrated relationship. Precipitation is the primary driver of the hydrologic cycle. To quantify the volume of water available for groundwater recharge, or that is diverted to the streams as run-off, a good understanding of the land cover, land-use, and underlying soil type is necessary. The following section provides a summary of the background geology and hydrogeology information related to the SSMR Source Protection Area.

2.1 TOPOGRAPHY AND DRAINAGE

The City of Sault Ste. Marie lies within the broad, relatively flat St. Marys River valley, which is about 6 km to 7 km wide between the river and Precambrian uplands to the north.

In the East Basin, the ground elevations range from 180 masl to 230 masl and are slightly lower and flatter than in the Central Basin. In the Central Basin, the surface rises in a series of four or five terraces, representing different glacial lake levels, each about 10 m in height, with elevations from 180 masl to 240 masl. In the northern part, the ground rises steeply to the Precambrian uplands (>300 masl). The surface topography in the SSMR Source Protection Area is illustrated in Map 2-1.

Drainage is via a series of streams flowing southward off the Precambrian uplands, across the lowlands to the St. Marys River. In the East Basin the largest stream is the Root River/Crystal Creek system, which originates in the Precambrian uplands and meanders across the lowlands and ultimately discharges to St. Marys River. The streams in the Central Basin include the East Davignon Creek, West Davignon Creek and Bennett Creek. The surface water features in the SSMR Source Protection Area are further discussed in Section 0.

2.2 PHYSIOGRAPHY

The SSMR Source Protection Area is situated at the border of the Lake Temagami Ecoregion and Georgian Bay Ecoregion. The Lake Temagami Ecoregion covers the northern Precambrian uplands portion of the SSMR Source Protection Area and the Georgian Bay Ecoregion covers the southern lowland area (Crins et al., 2009).

In the Lake Temagami Ecoregion, the bedrock is predominantly granitic and gneissic. The terrain is moderately to strong broken and the substrate covering the bedrock is thin. In the Georgian Bay Ecoregion, much of the bedrock is covered with ground moraine (till) of variable depth, glaciofluvial materials associated with spillways and outwash deposits can also be found (Crins et al, 2009).

2.3 LAND USE

The land use of the SSMR Source Protection Area is presented in Map 2-2. Most development and the majority of the population are in the City of Sault Ste. Marie, along the north shore of the St. Marys River on the lowlands. Other small communities are found along the northern shore of Lake Superior and on the Precambrian uplands, along the Hwy 17 North corridor. Based on the mapping data provided by the SSMRCA (SSMRCA, 2012), it is estimated that the urbanized area accounts for approximately 15% of the overall planning region. This includes residential, industrial, commercial and institutional uses. The remainder of the area is mainly composed of rural areas, woodland, wetland and water bodies.

2.4 LAND COVER

Land cover for the SSMR Source Protection Area is shown in Map 2-3 (Canada Land Inventory, 1996). The key categories include productive woodland, improved pasture and forage crops, and urban areas.

2.5 CLIMATE

Precipitation, evaporation, and temperature have a direct effect on the amount of surface runoff and the amount of water available to recharge the aquifers. Hence, understanding precipitation, evaporation, and temperature and their patterns plays a key role in the water budget analysis. The climate of the SSMR Source Protection Area is affected temporally and spatially by seasonal variations and the physical proximity to Lake Superior. The area is subject to warm summers and cold snowy winters. Lake-effect snow is a common feature of Sault Ste. Marie winters, making it a recognized snow-belt area.

2.5.1 CLIMATE DATA

Climate data are available from several sources for the SSMR Source Protection Area. Environment Canada has had weather stations located at several sites in the Sault Ste. Marie area starting in 1889. Table 2.1 presents a summary of Environment Canada's weather station history in the Sault Ste. Marie region.

Table 2.1 Environment Canada Weather Station Recording History

Station Name	Station ID	Latitude	Longitude	Elevation	Years of Data
Sault Ste. Marie Forestry	6057595	46°30'N	84°22'W	193 masl	1889-1933
Sault Ste. Marie Insectary	6057597	46°28'N	84°28'W	191 masl	1951-1954
Sault Ste. Marie Shingwauk	6057605	46°30'N	84°17'W	183 masl	1954-1955
Sault Ste. Marie (old)	6057589	46°32'N	84°30'W	206 masl	1949-1959

Sault Ste. Marie (new)	6057591	46°29'N	84°31'W	192 masl	2012-2013
Sault Ste. Marie #2	6057590	46°32'N	84°20'W	212 masl	1957-2002
Sault Ste. Marie A (Airport)	6057592	46°29'N	84°31'W	192 masl	1945-2012

Currently there is only one weather station (Sault Ste. Marie Station (new)) in operation in the Sault Ste. Marie region recording both temperature and precipitation data. It is located near the Sault Ste. Marie Airport (Sault Ste. Marie A station in Map 1-1). Sault Ste. Marie #2 station and Sault Ste. Marie A station both have continuous records between 1945 to 2012 and 1957 to 2002, respectively. Table 2.2 and Table 2.3 summarize the available climate data for the SSMR Source Protection Area. It is important to note that both weather stations are located in the lowlands area and the collected climate data may not be representative of the conditions in the Precambrian uplands area.

Table 2.2 Summary of Climate Data for the Period 1945-2012 Recorded at Sault Ste. Marie A Station ID 6057592

Latitude = 46o29'N Longitude = 84o31'W Elevation =192 m

Month	Temperature (°C)	Monthly Average		
		Rainfall	Snowfall	Precipitation
		(mm)	(cm)	(mm)
January	-10.2	9.1	81.2	71.3
February	-9.9	6.3	54.8	49.8
March	-4.5	27.0	35.1	57.7
April	3.3	50.8	15.3	66.0
May	9.7	68.5	1.1	69.6
June	14.7	76.8	1.0	77.8
July	17.7	69.9	0.0	69.9
August	17.2	81.0	0.0	81.0

Month	Temperature (°C)	Monthly Average		
		Rainfall	Snowfall	Precipitation
		(mm)	(cm)	(mm)
September	13.1	99.3	0.1	99.4
October	7.4	84.3	5.4	89.6
November	0.9	55.2	39.9	90.1
December	-6.1	16.4	79.4	77.0
Average	4.4			
Total		644.6	313.2	899.2

Table 2.3 Summary of Climate Data for the Period 1957-2002 Recorded at Sault Ste. Marie #2 Station ID 6057590

Latitude = 46°32'N Longitude = 84°20'W Elevation = 212 m

Month	Temperature (°C)	Monthly Average		
		Rainfall	Snowfall	Precipitation
		(mm)	(cm)	(mm)
January	-9.9	7.6	84.3	91.9
February	-8.7	5.6	48.1	53.6
March	-3.5	24.1	36.2	60.4
April	3.9	51.0	14.2	65.2
May	10.9	72.2	1.0	73.2
June	15.4	81.2	0.0	81.2
July	18.2	71.0	0.0	71.0
August	17.8	87.2	0.0	87.2
September	13.1	106.3	0.1	106.4
October	7.4	88.8	8.0	96.7
November	0.6	55.8	40.2	96.0
December	-6.4	15.2	87.3	102.5
Average	4.9			
Total		666.0	319.4	985.4

A review of Table 2.2 and Table 2.3 show that for the period of record, the average monthly rainfall and snowfall are in close agreement. For the Sault Ste Marie A station, the average annual rainfall was 644.6 mm and the average snowfall was 313.2 cm. Similarly for the Sault Ste. Marie Station #2, the average annual rainfall was 666.0 mm and the average snowfall was 319.4 cm. Monthly average temperature at Sault Ste. Marie ranged from 18.2 °C (July at Sault Ste. Marie #2 station) to -10.2 °C (January at Sault Ste. Marie A station).

Table 2.4 and Table 2.5 show that the average snowfall for December, January, February and March were 79.4 cm, 81.2 cm, 54.8 cm, 35.1 cm, respectively for the Sault Ste. Marie A Station. Similarly, 87.3 cm, 84.3 cm, 48.1 cm, and 36.2 cm of snowfall were recorded at the Sault Ste. Marie Station #2. The snowfall data demonstrate the significant climate variability from year to year in this area.

Table 2.4 Summary of Snowfall Data for the Period 1945-2012 Recorded at Sault Ste. Marie A Station ID 6057592

Month	Average Snowfall (cm)	Maximum Monthly Snowfall (cm)	Year of Maximum Monthly Snowfall	Minimum Monthly Snowfall (cm)	Year of Minimum Monthly Snowfall
December	79.4	207.2	1995	10.9	1994
January	81.2	146.9	1982	25.3	1948
February	54.8	142.3	2006	9.2	1993
March	35.1	162.8	2002	0	1973

Table 2.5 Summary of Snowfall Data for the Period 1957-2002 Recorded at Sault Ste. Marie #2 Station ID 6057590

Month	Average Snowfall (cm)	Maximum Monthly Snowfall (cm)	Year of Maximum Monthly Snowfall	Minimum Monthly Snowfall (cm)	Year of Minimum Monthly Snowfall
December	87.3	244.4	1995	15.7	1968
January	84.3	142.3	1972	24.8	1961

February	48.1	98.7	1968	7.6	1998
March	36.2	120.2	2002	0.0	1973

2.6 SURFACE WATER FEATURES

The St. Marys River is the connecting channel between Lake Superior and Lake Huron, where water exits Lake Superior from Whitefish Bay, flowing in a southeasterly direction. The entirety of the St. Marys drainage basin includes the Lake Superior watershed, as the lake drains directly into the river. The SSMR Source Protection Area consists of twelve subwatersheds with each independently draining into the St. Marys River or to Lake Superior (Map 2-4). Ten out of the twelve subwatersheds are associated with major creek systems. The last two subwatersheds include one system which discharges directly into Lake Superior and another subwatershed, which is located south of the confluence of the Root River and Crystal Creek and discharges directly to St. Marys River.

2.6.1 SUBWATERSHEDS

A description of each subwatershed is presented in the following sections and Table 2.6 presents a summary of subwatershed physical characteristics.

Table 2.6 Summary of Subwatershed Physical Characteristics

Subwatershed	Top Elevation (masl)	Bottom Elevation (masl)	Drainage Length (km)	Catchment Slope	Drainage Area (km2)
Bennett Creek	400	185	15.6	1.4%	25.4
Big Carp River	315	183	12.5	1.1%	51.8
Central Creek	225	189	3.4	1.0%	2.7
Crystal Creek	411	186	19.1	1.2%	51.6
East Davignon Creek	368	184	13.3	1.4%	22.7
Fort Creek	274	176	7.4	1.3%	31.6

Lake Superior	349	190	3.3	4.9%	105.4
Leigh Bay Creek	220	186	4.7	0.7%	15.9
Little Carp River	350	184	15.1	1.1%	20.1
Root River	375	185	21.4	0.9%	123.4
St Marys River	350	180	6.8	2.5%	50.3
West Davignon Creek	360	189	10.8	1.6%	20.3

Big Carp River

The Big Carp River is the first major watercourse east of Lake Superior. The Big Carp River originates at Walls Lake at an elevation of 315 masl in heavily forested terrain in the Precambrian Shield. Walls Lake is a small inland lake rimmed with wetland areas approximately 2 km in length. From the lake, the river flows southeasterly where it is joined by an 8 km long easterly tributary. This confluence is approximately 2.4 km south of Second Line. The river discharges into the St. Marys River just east of Carpin Beach (SSMRCA, 2011).

Little Carp River

The Little Carp River runs approximately 15 km from its headwaters to its mouth just east of the Big Carp River along the St. Marys River. It originates in the Precambrian Shield in Prince Township at a small lake (1.8 ha) north of Third Line. From this point it flows through a steep valley south to Second Line, after which it meanders through the lowlands of the Algonquin and Nipissing Terraces and approaches the Big Carp River before joining the St. Marys River (SSMRCA, 2011). Similar to the Big Carp River, land use within this watershed is mainly undeveloped with some sparse residential and agricultural development.

Leigh Bay Creek

Leigh Bay Creek borders the western edge of the urban area of the city. Its headwaters do not extend to the uplands area but originate in the flat lowland area just north of Second Line. The creek flows southeasterly across Second Line and Leigh's Bay Road. It then crosses Base Line and discharges to the St. Marys River. A diversion channel from the Bennett and West Davignon Creeks joins these two systems with the Leigh Bay Creek just north of the Base Line crossing.

Bennett Creek

The Bennett Creek drainage basin originates in a vast marshy area in the Precambrian Shield. Bennett Creek flows southeasterly from its headwaters for approximately 16 km to its confluence with West Davignon Creek just south of Wallace Terrace (SSMRCA, 2011). Initially, the creek's slope is gentle and it increases as the watercourse drops into the terraced lowlands area within the City. Flow of the creek is restricted within the urban area of the City due to road crossings prior to its confluence with West Davignon Creek. The Bennett-West Davignon diversion channel reduces the creek's flow just north of Wallace Terrace east of the Allens Side Road intersection. The Bennett Creek discharges to the St. Marys River via a constructed channel that terminates at the Essar Steel Algoma ore docks.

West Davignon Creek

The main channel of the West Davignon Creek is approximately 11 km long. Similar to the Bennett system, the West Davignon headwaters are located on the Precambrian Shield. The main source for this system is Allard Lake, a lake edged by wetlands. Other wetland areas in the vicinity also contribute to the flow of this creek. Flow of the creek is generally south until it reaches Second Line at which point it swings southeast. Just north of Second Line, a portion of the flow is diverted south to join Bennett Creek. The remaining flow meanders southeast until it crosses Wallace Terrace. From this point the natural creek bed has been channelled west and then south to its confluence point with Bennett Creek. As previously mentioned, the discharge point of Bennett Creek and West Davignon Creek is at the north end of the Essar Steel Algoma ore docks.

Central Creek

This small watercourse contributes flow to East Davignon Creek and is almost entirely within the urban area of Sault Ste. Marie (SSMRCA, 2011). The creek begins near the intersection of Moss Road and Third Line. It flows south to a continuous concrete aqueduct at Wallace Terrace. Through the aqueduct it is discharged to East Davignon Creek on Essar Steel Algoma property, approximately 1 km upstream of the East Davignon's discharge point to the St. Marys River. Central Creek collects residential and industrial run off from portions of the west end of the city.

East Davignon Creek

The East Davignon Creek headwaters are located north of the city limits within the Precambrian Shield. Nettleton Lake is a small lake (12 ha) located along the main branch of the creek at Fifth Line. The East Davignon flows south through a steep ravine to Rossmore Road where urban development is very close to the creek. South of Second Line, the creek is channeled into a continuous concrete aqueduct that carries the creek across Wallace Terrace and then southwesterly through the Essar Steel Algoma property to the St. Marys River. Along this channel, discharges from Tenaris Algoma Tubes and Essar Steel Algoma contribute to the creek flow as well as the aqueduct carrying Central Creek.

Fort Creek

Fort Creek originates at the northern limit of the Algonquin Terrace and flows through the heart of the urban district, located on the Nipissing Terrace. The Fort Creek dam was constructed in the 1970's upstream of the Second Line crossing, to alleviate flood damage to the urban core. The upper two thirds of the watershed (i.e., upstream of the dam) is steeply sloped and has a number of steep-sided ravines. Downstream of the dam at Second Line, the topography gently slopes south towards the St. Marys River.

Root River

The Root River watershed, which also includes the West Root River, is the largest catchment in the planning area. The basin originates in the northern uplands where a number of swamps, bogs and lakes, including Upper and Lower Island, Aweres and Trout Lakes, feed into the three main tributaries of the river, the Root, the West Root and Crystal Creek. The West Root drains the western portion of the basin and joins the main river west of Highway 17 North near the Root River Golf Course. The Crystal Creek headwaters are in the northeastern region of the basin. Crystal Creek joins the main river north of Highway 17 East, close to the eastern boundary of the Batchewana First Nation Rankin Reserve. The Root River discharges to the St. Marys River at Bell's Point on Little Lake George.

Crystal Creek

Crystal Creek is located at the northeastern corner of the SSMR Source Protection Area and flows to the western boundary of the Batchewana First Nation Rankin Reserve. Prior to discharging to the St. Marys River, it joins with the Root River.

Crystal Creek traverses primarily the Precambrian uplands area but at the downstream area passes through the Algonquin and Nipissing Terraces. The subwatershed is marked by several inland lakes and fairly extensive drainage system. The area is largely undisturbed.

2.6.2 STREAM FLOW DATA

Data for two Environment Canada stream flow gauge stations are available in the SSMR Source Protection Area (Map 1-1). The first gauge station was installed along Root River (02CA002), near the intersection of Old Garden River Road and Landslide Road. The second gauge station was installed on Big Carp River (02BF004), near the Town Line Road and Base Line intersection. Stream flow data were collected from both stations since 1971 and 1979 for the Root River and Big Carp River gauge stations, respectively. Stream flow data collected from the Root River gauge station (02CA002), between 1971 and 2010 were used during the development and calibration of the surface water model.

The stream flow was reviewed to identify trends using the data available from the Environment Canada database. The average flows measured at the Root River Station are greater than Big Carp River due to the larger drainage area associated with Root River. For both stations, flows are higher during the spring months (March to May) and lower during the summer (June to September) and winter months (January, February). The average flow rates for both watercourses are summarized in Table 2.7.

Table 2.7 Average Flow Rate Measured at Root River Gauge and Big Carp River Gauge

Month	Average Flow at Root River Gauge Station (m3/s)	Average Flow at Big Carp River Gauge Station (m3/s)
Data Duration	1971 - 2010	1979 – 2010
January	0.8	0.3
February	0.5	0.3
March	1.9	1.0
April	7.4	2.9
May	2.6	0.8
June	1.0	0.4
July	0.5	0.2
August	0.5	0.2
September	1.1	0.4
October	2.4	1.0
November	2.8	1.1
December	1.6	0.7

Stream flow typically consists of overland flow, interflow and baseflow. Overland flow is defined as surface runoff during precipitation events. It is the dominant part of the stream hydrographs and creates “peaks” in the hydrographs during precipitation events. The percentages of precipitation that becomes overland flow are generally higher in urban area due to higher percentages of impervious area and low evapotranspiration. Interflow usually makes up a minor component of the hydrograph. Interflow represents stormflow moving through a shallow soil horizon without reaching the zone of saturation (Bedient et al., 2008). In general, baseflow

represents flow contributed by groundwater and this amount can vary depending on the characteristics of the watershed.

Baseflow values were estimated for the stream hydrographs at two stream gauge stations (station 02CA002 along Root River and station 02BF004 along Big Carp River). Two baseflow separation techniques were used. The first technique is known as “5-day running average of the 7-day minimum method” and the second technique is known as “slope change method”, these baseflow separation techniques are further explained below.

5-day running average of the 7-day minimum method

The 5-day running average of the 7-day minimum method (DRAFT Interim Water Budget Technical Direction Version 3.0, MNR, December 21, 2005) is designed to link the low points on the hydrograph. The method first takes the minimum daily flow rate for 7 consecutive days (with 3 days ahead and 3 days back). The 5-day moving average of the 7-day minimum is then calculated (with 2 days ahead and 2 days back) as the daily baseflow.

Slope change method

The slope change method (Arnold and Allen, 1995) (Arnold et. al. 1999) estimates the daily baseflow based on the previous day flow. If the present daily flow rate is lower than both the previous day flow rate and previous day baseflow rate, the present day baseflow rate is estimated by adding 0.001 m3/s to the present daily flow rate. If the present daily flow rate is higher than the previous day baseflow rate or previous day flow rate, the present day baseflow rate is estimated by multiplying the previous day baseflow rate by a factor of 1.05.

The estimated average baseflow at the Root River gauge station ranges from 0.8 m3/s (estimated using the slope change method) to 1.2 m3/s (estimated using the 5-day running average of the 7-day minimum method). This corresponds to a range of watershed contribution to baseflow of 250 mm/year and 340 mm/year, respectively.

The estimated baseflow at the Big Carp River gauge station is approximately 0.4 m3/s. This corresponds to a range of watershed contribution to baseflow of 100 mm/year and 130 mm/year, respectively. Results from the estimated baseflow calculations are summarized in Table 2.8.

Table 2.8 Summary of Baseflow Estimation at Root River and Big Carp River

Baseflow	Root River (02CA002)		Big Carp River (02BF004)	
Data Duration	1971 - 2010		1979 – 2010	
Units	m3/s	mm/year	m3/s	mm/year

5 day running average of the 7-day minimum	1.2	340	0.4	130
slope change method	0.8	250	0.4	100

2.6.3 OTHER ECOLOGICALLY SENSITIVE AREAS

Upwelling areas, wetlands and headwaters are known to exist south of the Precambrian uplands, as a result of local-scale discharge of groundwater through the coarse-grained materials. The shallow system provides groundwater flux to the streams and is an essential component to preserving the natural function of the ecosystem.

The extensive rivers and creeks present in the study area are habitat for a multitude of fish species that depend on these watercourses for spawning and sustained health throughout the seasons. Similarly, within the SSMR Source Protection Area, wetlands are habitat for numerous amphibians, flora and fauna. Map 2-5 illustrates the wetlands and conservation areas. The wetlands comprise 7 % (including overlap with water bodies) of the SSMR Source Protection Area. There are several smaller wetland areas in the northern Precambrian uplands of the SSMR Source Protection Area associated with headwater areas of the rivers and creeks, which flow south towards the St. Marys River. Along the shore of the St. Marys River, wetland areas are found at the outlet of the Big Carp River and Little Carp River.

2.7 GEOLOGY

The regional quaternary geology and bedrock geology within the SSMR Source Protection Area is shown in Map 2-6 and Map 2-7, respectively. The geology of the area has been described in reports and maps by the Geological Survey of Canada (GSC) and Ontario Geological Survey (OGS), including Frarey (1977), and Barnett, Cowan and Henry (1991). Geological interpretations were also reviewed in reports by the International Water Supply Ltd. (IWS) (1978, 1979, 1980) and IWC (1971, 1978, 1979, 1995), R.J. Burnside & Associates Limited (Burnside, 2003) and Kresin Engineering Corporation and MacViro Consultants Ltd. (Kresin & MacViro, 2006).

Selected MOE well records were used to delineate the hydrostratigraphy. These records provided greater detail in soil description and extended to greater depths. The municipal supply well records were also included in the geology cross-sections to provide additional confirmation of the hydrostratigraphy. The locations of the geological cross-sections are presented in Map 2-8 and the geological Cross-Sections A-A' to F-F' are shown on Map 2-8A to Map 2-8F. MOE well records used to construct the geological cross-sections are presented in Appendix A-1.

2.7.1 QUATERNARY GEOLOGY

The surficial quaternary geology in the Sault Ste. Marie area (Map 2-6) was mapped by Cowan and Broster (1988), and their map has been used in previous reports by Burnside (2003) and Kresin and GENIVAR (2008). The overburden consists of a sequence of glacial sediments, which reach a thickness of 100 m to 120 m in the East and Central Basins. These sediments include till, glaciofluvial and glaciolacustrine sediments, which are overlain by recent deposits of peat, alluvium and fill.

Map 2-6 shows two types of glaciolacustrine deposits in the area south of the Precambrian uplands: deep-water deposits, consisting mostly of clay and silt (represented by brown), overlain by younger shallow water deposits, consisting mostly of fine sand (represented by purple). Since the last glacial retreat, isolated organic peat deposits (represented by dark purple) have been accumulating in lakes and depressions in the study area. Recent alluvium (represented by dark blue) has been deposited in modern stream channels that have cut into older glacial sediments.

2.7.2 BEDROCK GEOLOGY

The bedrock geology in the Sault Ste. Marie area has been described by Frarey (1977) and others. The oldest bedrock at the site is the Archean granitic/gneissic rocks of the Canadian Shield (Superior Province, 3.1-2.5 billon years old). These crystalline rocks are overlain unconformably by Middle Proterozoic (1.1 billon years old) clastic sedimentary rocks of the Jacobsville Group, primarily sandstone, with minor shale and conglomerate. Map 2-7 presents the bedrock geology in the SSMR Source Protection Area.

The Jacobsville Group rocks were deposited by fluvial processes onto the older exposed crystalline Shield uplands along an east-west contact in the northern part of the study area. Under the flat lowland south of the exposed Shield, where the bedrock is covered by up to about 100 m of overburden; outcrops of the Jacobsville Group are rare. The Jacobsville Group rocks consist of mainly red and grey quartzose sandstone, with minor shale and a basal conglomerate. The sandstone is unfossiliferous and weakly stratified. It is hematitic in the lower part and grades upward to a grey sandstone. The matrix is composed of well-sorted, subrounded grains of mostly quartz. Matrix cement is minor, and is mainly hematite and silica (Frarey, 1977).

Although the Archean granitic/gneissic rocks are strongly folded and faulted due to regional tectonic activity, the Jacobsville sedimentary rocks are relatively undisturbed and essentially flat-lying (Frarey, 1977). The Jacobsville Group extends southward under the St. Marys River and into Michigan (Michigan Center for Geographic Information, 2005). In the Sault Ste. Marie area, the formation is reported to be at least 200 m thick (Frarey, 1977), and has a regional dip southward toward the centre of the Michigan sedimentary basin.

2.7.3 BEDROCK TOPOGRAPHY

Three bedrock basins (or depressions) have been identified based on years of drilling activity, referred to as the East, Central and West Basins. This report focuses on the East and Central Basins. The topography of the bedrock surface is shown in Map 2-9, based on contours from Leahy and Giblin (1979) and Burnside (2003). The basins are pre-glacial erosional features, and

are more properly described as valleys, carved into the sandstone by rivers draining from the north. During the last glacial retreat, the valleys were infilled by sediments deposited by glacial melt waters.

The basins generally widen and deepen from the Precambrian uplands toward the south. Near the St. Marys River, the centre of the East Basin is about 110 m deep and the bedrock surface is at an elevation of about 75 masl (see Cross-section C-C', Map 2-8C). In comparison, the Central Basin is about 50 m deep in the centre, and the bedrock surface is at about 140 masl. The basins are separated by bedrock ridges, where the bedrock is about 20 m below surface (Cross-Section D-D', Map 2-8D).

At a regional scale, the St. Marys River flows mostly within a bedrock valley between Ontario and Michigan. In Cross-Section B-B' (Map 2-8B), the surface of the sandstone bedrock in the East Basin south of the Shannon Well is at approximately 90 masl. On the Michigan side of the St. Marys River, the East Basin ends abruptly against the bedrock escarpment, which rises 100 m from the basin floor to the bedrock plateau on Sugar Island.

Based on an extrapolation of the bedrock surface in Cross-Section A-A' (Map 2-8A) and St. Marys River depth contour map, the sandstone surface in the Central Basin is in contact with the St. Marys River. IWS (1978a) also reported that the St. Marys River "flows on the sandstone aquifer near the locks" in the Central Basin, and that the sandstone "aquifer likely is in contact with the river at the two divides between the three rock basins". In the East Basin, the estimated St. Marys River bottom is at ~157 masl (based on the water level information below the ship canal lock, obtained from Great Lake Information Network and estimated water depth from depth contour map). The St. Marys River bottom in the East Basin is near the interpreted sandstone surface, separated by a layer of clay overburden material with an approximate thickness of 10 m. However, due to lack of detailed bedrock topography and data on the bottom elevation of St. Marys River, it is difficult to assess whether the St. Marys River is in contact with the sandstone in the East Basin.

2.8 HYDROGEOLOGICAL DATA COLLECTION

This section provides a summary of historical hydrogeological data collected in the SSMR Source Protection Area. The information has been used to develop the conceptual understanding and will assist in the calibration of the numerical models.

2.8.1 MONITORING OF MUNICIPAL PRODUCTION WELLS

The municipal wells in the East Basin include the Shannon Well, Lorna Well 1 and Lorna Well 2. The municipal wells in the Central Basin are the Steelton Well, Goulais Well 1 and Goulais Well 2. The details of the municipal wells are summarized in Table 2.9; the locations of the municipal wells are shown in Map 1-1, Map 2-10, Map 2-10A and Map 2-10B. The geological logs of the municipal wells are presented in Appendix A-2.

Table 2.9 Municipal Wells in the SSMR Source Protection Area

Basin	Well	Northing	Easting	Ground Elevation (masl)	Well Depth (m)	Screened Formation	Screen Depth (m)
Central Basin	Goulais Well No. 1	5156958	700733	189.52	55.2	Sandstone	49.1 -55.2
	Goulais Well No. 2	5156958	700737	189.59	54.9		48.8-54.9
	Steelton Well	5157062	701671	189.22	43.0		23.3-43.0
East Basin	Shannon Well	5156234	710260	195.08	100.6	Sand, gravel	94.3-100.6
	Lorna Well No. 1	5154317	710362	183.15	76.2		56.7-76.2
	Lorna Well No. 2	5154321	710364	182.90	75.3		56.7-75.3

The water levels at these municipal wells were mostly monitored by the Sault Ste. Marie Public Utilities Commission (PUC) on a daily to yearly basis as early as August 1966 at Steelton Well. Water level monitoring activities at municipal wells are summarized in Table 2.10 and hydrographs of the municipal wells are presented in Appendix B-1.

Pumpage at the municipal wells was mostly recorded by the PUC. The monthly average pumping rates are also presented in Appendix B-1 along with the monthly average water level data.

Table 2.10 Water Level Monitoring Duration and Frequency at Municipal Wells

Well	Water Level Monitoring Duration	Monitoring Frequency
Goulais Well No. 1	January 1969 to October 1993	Weekly monitoring by PUC
	November 1993 to September 2000	Bi-weekly to yearly monitoring by PUC
	October 2000 to November 2001	Daily monitoring by PUC
	January 2004 to December 2004	Daily to bi-weekly monitoring by PUC
	January 2006 to December 2008	Daily to bi-weekly monitoring by PUC

Table 2.10 Water Level Monitoring Duration and Frequency at Municipal Wells

Well	Water Level Monitoring Duration	Monitoring Frequency
	January 2010 to September 2013	Daily to triennial monitoring by PUC
	March 2011 to October 2011	Weekly to monthly manual monitoring and hourly monitoring using levelogger by Kresin
Goulais Well No. 2	January 1969 to October 1993	Weekly monitoring by PUC
	November 1993 to May 2000	Bi-weekly to yearly monitoring by PUC
	June 2000 to December 2005	Daily to bi-monthly monitoring by PUC
	January 2008 to May 2009	Daily to bi-weekly monitoring by PUC
	January 2010 to October 2010	Monthly to bi-monthly monitoring by PUC
	January 2011 to December 2011	Daily to bi-weekly monitoring by PUC
	March 2011 to October 2011	Weekly to monthly manual monitoring and hourly monitoring using levelogger by Kresin.
	May 2012 to November 2012	Daily to bi-weekly monitoring by PUC
	June 2013 to December	Tri-monthly monitoring by PUC
Steelton Well	August 1966 to October 1993	Weekly monitoring by PUC
	November 1993 to July 2001	Daily to yearly monitoring by PUC
	December 2001 to April 2003	Daily monitoring by PUC
	September 2003 to January 2005	Daily to bi-weekly monitoring by PUC
	June 2005 to February 2008	Daily to bi-weekly monitoring by PUC
	June 2008 to December 2012	Daily to bi-monthly monitoring by PUC

Table 2.10 Water Level Monitoring Duration and Frequency at Municipal Wells

Well	Water Level Monitoring Duration	Monitoring Frequency
	March 2011 to October 2011	Weekly to monthly manual monitoring and hourly monitoring using levelogger by PUC.
	March 2013 to December 2013	Weekly to semi-annual monitoring by PUC
Lorna Well No. 1	January 1980 to October 1993	Weekly monitoring by PUC
	November 1993 to February 2007	Daily to yearly monitoring by PUC
	August 2007 to May 2009	Daily to bi-monthly monitoring by PUC
	January 2010 to November 2011	Daily to weekly monitoring by PUC
	May 2012 to December 2012	Daily to weekly monitoring by PUC
	April 2013 to October 2013	Monthly to bi-monthly monitoring by PUC
Lorna Well No. 2	October 1983 to October 1993	Weekly monitoring by PUC
	November 1993 to January 2002	Daily to yearly monitoring by PUC
	August 2006 to May 2009	Daily to yearly monitoring by PUC
	January 2010 to November 2011	Daily to weekly monitoring by PUC
	October 2010 to February 2011	Frequent manual monitoring and hourly monitoring using levelogger by Kresin and CEG (pumping test)
	May 2012 to December 2012	Daily to weekly monitoring by PUC
	April 2013 to October 2013	Monthly to bi-monthly monitoring by PUC
Shannon Well	July 1973 to October 1993	Weekly monitoring by PUC
	November 1993 to April 1998	Bi-weekly to yearly monitoring by PUC

Table 2.10 Water Level Monitoring Duration and Frequency at Municipal Wells

Well	Water Level Monitoring Duration	Monitoring Frequency
	September 2001 to November 2005	Daily to bi-monthly monitoring by PUC
	July 2006 to February 2008	Daily monitoring by PUC
	December 2010 to November 2012	Daily to bi-weekly monitoring by PUC
	October 2010 to February 2011	Frequent manual monitoring and hourly monitoring using levelogger by Kresin and CEG (pumping test)
	February 2013 to July 2013	Weekly to tri-monthly monitoring by PUC

2.8.2 MONITORING OF PUBLIC UTILITIES COMMISSION MONITORING WELLS

Monitoring wells were installed by the PUC adjacent to each production well. The available information for these wells is summarized in Table 2.11, the location of the PUC monitoring wells are shown in Map 2-10, Map 2-10A and Map 2-10B.

Table 2.11 PUC Monitoring Wells in the SSMR Source Protection Area

Basin	Well	Northing	Easting	Well Depth (m)	Screened Formation	Screen Depth (m)
Central Basin	Goulais Monitoring Well	5156960	700723	47.6	Sandstone	na
	Steelton Monitoring Well 1	5157065	701668	25.7	Sandstone	na
	Steelton Monitoring Well 2	5157053	701666	22.8	Sandstone	na
East Basin	Lorna Monitoring Well	5154308	710357	na	na	na
	Shannon Monitoring Well	5156228	710263	na	na	na

Note:

na = The screen depth information for Goulais Monitoring Well, Steelton Monitoring Well 1 and 2 are not available. The well depth and screen information for Lorna Monitoring Well and Shannon Monitoring Well are not available.

The water level at these PUC monitoring wells was monitored by the PUC and Kresin. Available water level monitoring data are summarized in Table 2.12 and hydrographs of the PUC monitoring wells are presented in Appendix B-2.

Table 2.12 Water Level Monitoring Duration and Frequency at PUC Monitoring Wells

Well	Water Level Monitoring Duration	Monitoring Frequency
Goulais Monitoring Well	January 1969 to October 1993	Weekly monitoring by PUC
	November 1993 to May 2000	Bi-weekly to yearly monitoring by PUC
	March 2011 to October 2011	Weekly to monthly manual monitoring and hourly monitoring using levelogger by Kresin
Steelton Monitoring Well 1	August 1966 to October 1993	Weekly monitoring by PUC
	November 1993 to May 2000	Bi-weekly to yearly monitoring by PUC
	January 2004 to December 2004	Daily to weekly monitoring by PUC
	January 2006 to December 2007	Daily to weekly monitoring by PUC
	March 2011 to October 2011	Weekly to monthly manual monitoring and hourly monitoring using levelogger by Kresin
Steelton Monitoring Well 2	June 2011 to October 2011	Bi-weekly to monthly manual monitoring and hourly monitoring using levelogger by Kresin
Lorna Monitoring Well	January 1980 to October 1993	Weekly monitoring by PUC
	November 1993 to May 2000	Bi-weekly to yearly monitoring by PUC
Shannon Monitoring Well	July 1973 to October 1993	Weekly monitoring by PUC
	November 1993 to May 2000	Bi-weekly to yearly monitoring by PUC

2.8.3 MONITORING OF INTERNATIONAL WATER SUPPLY LTD. TEST WELLS

Several test wells were installed by International Water Supply Ltd. (IWS) in the East and Central Basins, during the groundwater exploration programs of the 1960s and 1970s. Hydraulic testing was also completed in some wells. The available details of these test wells were extracted from historical IWS reports and are summarized in Table 2.13 and their locations were approximated from historical IWS reports as presented in Map 2-10, Map 2-10A and Map 2-10B. The geological logs of the test wells are also presented in Appendix A-3. Data from these historical tests were used to further understand the aquifer-aquitard system and aquifer properties. The results from these historical tests are further discussed in Section 2.8.5.

Table 2.13 IWS Test Wells in the SSMR Source Protection Area

Basin	Well	Northing	Easting	Well Depth (m)	Screened Formation (m)	Screen Depth (m)
Central Basin	TW1/66	5156901	699047	na	na	na
	TW1/65	5156988	700699	na	na	na
	TW2/70	5153000	708813	31.4	Sand, gravel	28.0 - 31.4
	TW2/78	5157035	699908	70.7	Sandstone	65.1 - 70.7
	TW3/78	5157135	699889	57.0	Sand	54.6 - 57.0
	TW4/78	5157853	699023	104.2	Sandstone	79.2 - 104.2
East Basin	TW4/66	5155384	708775	na	na	na
	TW3/70	5153907	710447	46.9	Sand, gravel	50.3 - 46.9
	TW4/70	5155311	711322	60.0	Sand, gravel, sandstone	46.9 - 60.0
	TW5/70	5156221	710292	108.8	Sand, gravel, sandstone	105.8 - 108.8
	TW1/78	5156184	711568	96.0	Sandstone	93.0 - 96.0

Table 2.13 IWS Test Wells in the SSMR Source Protection Area

Basin	Well	Northing	Easting	Well Depth (m)	Screened Formation (m)	Screen Depth (m)
	TW1/75	5154266	710313	76.2	Sand, gravel	71.6 - 76.2
	TW2/72	5156224	710254	107.9	Sand, gravel	97.8 - 107.9

Notes:

na = not available

The locations of the test wells were estimated based on the location maps presented as part of the borehole/well logs.

TW2/70 is located near the boundary separating the East Basin and Central Basin. It was determined as being located within the Central Basin in this table based on Map 2-10.

2.8.4 MONITORING OF CEG/KRESIN MONITORING WELLS

Cole Engineering Group Ltd. (CEG) and Kresin installed a series of monitoring well nests (MW1 to MW6) in the East and Central Basins, to monitor the shallow groundwater zone during the pumping test conducted in 2010. The locations of these monitoring wells are shown in Map 2-10, Map 2-10A and Map 2-10B. The geological logs for these wells are in Appendix A-4. Each monitoring well nest included a deep (denoted by “D”) well and a shallow (denoted by “S”) well.

MW1 to MW4 were installed near the Steelton Well and Goulais Wells and are screened in the clay. MW5 and MW6 were installed near the Lorna Wells and were screened in the surficial silt and sand deposits. All monitoring wells consist of flush-jointed, 50 mm (2 inch) PVC pipe and machine-slotted screens. A sand pack was placed around the screens, and the annulus was sealed with bentonite to the surface. Each well was completed with a steel protective casing. Details of the CEG/Kresin monitoring wells are summarized in Table 2.14.

Table 2.14 CEG/Kresin Monitoring Wells in the SSMR Source Protection Area

Basin	Well	Northing	Easting	Well Depth (m)	Screened Formation	Screen Depth (m)
Central Basin	MW1S	5156622	700480	4.6	Clay	3.1 - 4.6
	MW1D	5156622	700480	12.2	Clay	9.1 - 12.2

Table 2.14 CEG/Kresin Monitoring Wells in the SSMR Source Protection Area

Basin	Well	Northing	Easting	Well Depth (m)	Screened Formation	Screen Depth (m)
	MW2S	5156905	700894	4.6	Clay	3.1 - 4.6
	MW2D	5156899	700898	12.2	Clay	9.1 - 12.2
	MW3S	5156928	700930	4.6	Clay	3.1 - 4.6
	MW3D	5156929	700930	12.2	Clay	9.1 - 12.2
	MW4S	5157103	702270	4.6	Clay	3.1 - 4.6
	MW4D	5157103	702270	12.2	Clay	9.1 - 12.2
	MW5S	5154105	710483	2.1	Silt, sand	0.6 - 2.1
	MW5D	5154106	710481	3.9	Silt, sand	2.4 - 3.9
	MW6S	5156225	711276	2.1	Sand	0.6 - 2.1
East Basin	MW6D	5156224	711277	4.6	Sand	3.1 - 4.6

The CEG/Kresin monitoring wells were monitored on a regular frequency between July 2010 and October 2011. Dataloggers were also deployed in all monitoring wells during the monitoring period to collect hourly groundwater level readings. Table 2.15 summarizes the monitoring activities at the CEG/Kresin monitoring wells; the hydrographs of these monitoring wells are presented in Appendix B-3.

Table 2.15 Water Level Monitoring Duration and Frequency at CEG/Kresin Monitoring Wells

Well	Water Level Monitoring Duration	Monitoring Frequency
MW1D, MW1S, MW2D, MW2S, MW3D, MW3S, MW4D, MW4S	July 2010 to October 2011	Mostly monthly manual monitoring and hourly monitoring using levelogger
MW5D, MW5S	July 2010 to February 2011	Mostly monthly manual monitoring and hourly monitoring using levelogger. Increased monitoring frequency during December 2010 pumping test
MW6D	July 2010 to January 2011	Mostly monthly manual monitoring and hourly monitoring using levelogger. Increased monitoring frequency during December 2010 pumping test
MW6S	July 2010 to December 2010	Mostly monthly manual monitoring and hourly monitoring using levelogger. Increased monitoring frequency during December 2010 pumping test

2.8.5 HISTORICAL HYDRAULIC TESTS

Between the period of 1968 to 2011, multiple hydraulic tests were conducted in the SSMR Source Protection Area. This section provides a brief summary of the historical hydraulic tests, details of each hydraulic test and analysis of test results.

Ontario Water Resources Commission Pumping Test at Goulais Wells, 1968

In August 1968, the Ontario Water Resources Commission (OWRC, 1969) conducted a pumping test of the Goulais Wells. The purpose of the test was to evaluate interference complaints by private well owners in the surrounding area. The Goulais Wells were pumped for 19 days. On the first day of the pumping test, the combined pumping rate was approximately 10278 m3/day (1570 gpm); for the subsequent 18 days, the combined pumping rate was reduced to 7921 m3/day (1210 gpm). Prior to the end of the pumping test, water level measurements were collected from twelve private wells and two test wells. In addition to distance-drawdown data, the OWRC report also provided time-drawdown data for Goulais Wells 1 and 2, TW1/65 and TW1/66; these data were extracted from the OWRC report and are presented in Appendix C-1.

International Water Supply Ltd. Groundwater Investigation in East Basin, 1971

IWS conducted a groundwater investigation program during 1970 in the East Basin. The groundwater investigation program included drilling of four test wells (TW2/70, TW3/70 and TW4/70 and TW5/70). Short-duration (2 hours to 6 hours) pumping tests were conducted in test wells to evaluate the hydraulic properties of the screened zone (sand, gravel and upper portion of the sandstone). In addition, a 24-hour pumping test was conducted at TW5/70, to evaluate the regional characteristics of the sand and gravel and the underlying sandstone. The drawdown and recovery data from the pumping tests were extracted from the report and are presented in Appendix C-2.

International Water Supply Ltd. Groundwater Investigation, 1978

In 1978, IWS conducted a groundwater investigation in both the East Basin and Central Basin. The groundwater investigation program included drilling of one test well (TW1/78) in the East Basin and three test wells (TW2/78, TW3/78 and TW4/78) in the Central Basin. Short-duration pumping tests were conducted in test wells as part of the groundwater investigation program to evaluate the hydraulic properties of the screened formation (sand and sandstone). The pumping test data are presented in Appendix C-3.

R. J. Burnside & Associates Ltd. Goulais Wells and Steelton Well Shutdown Test, 2002

In September 2002, a controlled shutdown test was conducted by R. J. Burnside & Associates Ltd. (Burnside) on the Steelton Well and Goulais Wells. The purpose of the test was to estimate the transmissivity (T) and storativity (S) of the bedrock aquifer. The shutdown period was 1.5 hours and 5.8 hours for the Goulais Wells and the Steelton Well, respectively. During the shutdown test, water levels in the production wells and adjacent monitoring wells were monitored. The aquifer transmissivity and storativity values were estimated using semi-log analyses of the recovery data. The water level recovery data and associated analyses were extracted from the historical report Data Gap Analysis and Hydraulic Testing for a Tier 3 Risk Assessment (Kresin and CEG, 2012) and are included in Appendix C-4.

CEG/Kresin. Shannon Well and Lorna Wells Pumping Test, 2010

CEG/Kresin planned and executed a five-day pumping test in the East Basin in December 2010. During the pumping test, the Shannon Well and Lorna Well 2 were pumped simultaneously for five days (December 8, 2010 to December 13, 2010) at respective average flow rates of 6219 m3/day and 6551 m3/day, which are the maximum permitted rate for each well.

The monitoring well network included the Shannon Well and Lorna Wells, the PUC monitoring wells (Shannon Monitoring Well and Lorna Monitoring Well), the six nested CEG/Kresin monitoring wells (MW1 to MW6) and the following residential wells:

- 620 Old Garden River Rd;

- 834 Old Garden River Rd;
- 871 Old Garden River Rd;
- 1059 Old Garden River Rd Deep;
- 1059 Old Garden River Rd Shallow; and
- 1068 Old Garden River Rd.

The purpose of monitoring the shallow CEG/Kresin monitoring wells was to assess the potential for interconnectivity between the shallow and deep aquifer system. Monitoring the deep wells allowed for measurement of the zone of influence, assessment of boundary effects, and estimation of transmissivity and storativity. The water level monitoring extended six days after the pumping test to capture the aquifer recovery. Water level hydrographs collected during the pumping test were extracted from the historical report (Kresin and CEG, 2012) and are presented in Appendix C-5.

Steelton Well Shutdown Test, 2011

In July 2011, a storm caused a 30-day shutdown (July 11, 2011 to August 10, 2011) of the Steelton Well. Goulais Well 2 had been out of operation since the end of June due to maintenance and Goulais Well 1 was shutdown between June 29, 2011 to July 13, 2011. The recovery of the sandstone aquifer was monitored using the dataloggers previously installed in the Goulais Wells and Steelton Well. Water levels in the CEG/Kresin monitoring wells and nearby residential wells (327 Glasgow Ave and 1500 Korah Rd) were also recorded using dataloggers throughout the shutdown duration. The recovery hydrographs were extracted from the historical report (Kresin and CEG, 2012) and are presented in Appendix C-6.

The shutdown provided a rare opportunity to estimate recovery and static levels around the municipal wells. However, monitoring of the piezometric surface of the sandstone aquifer was challenging due to the artesian flowing conditions at these locations.

2.8.6 GROUNDWATER LEVELS IN THE EAST BASIN

The primary aquifer formation in the East Basin is the sand and gravel aquifer, in which the municipal wells (Shannon Well and Lorna Wells) are screened near the surface of the underlying sandstone. During the pumping test conducted at TW5/70 (IWS, 1971), it was found that the sand and gravel aquifer is hydraulically connected with the underlying sandstone.

Water levels in the East Basin production wells were monitored since the 1970s at the Shannon Well and monitoring at the Lorna Wells started in the 1980s. The monthly average water level in the East Basin wells during production ranged between 144 masl (40 mbgs) to 189 masl (6 mbgs). The monthly average water levels in the East Basin production wells were slightly higher after 1986, likely as a result of reduced municipal groundwater takings. The water levels in the East Basin production wells are summarized in Table 2.16 and the hydrographs are presented in Appendix B-1. It is important to note that the measured water levels at the municipal wells were likely influenced by pumping activities and are lower compared to the static water level.

Table 2.16 Range of Observed Monthly Average Water Level at East Basin Production Wells

Well	Monitoring Duration	Highest Monthly Average Water Level	Lowest Monthly Average Water Level	Overall Average Water Level
Shannon Well	1973 – 2012	189 masl/6 mbgs	157 masl/38 mbgs	168 masl/26 mbgs
Lorna Well No. 1	1980 – 2012	183 masl/0 mbgs	144 masl/40 mbgs	168 masl/15 mbgs
Lorna Well No. 2	1983 – 2012	181 masl/1 mbgs	144 masl/38 mbgs	169 masl/13 mbgs

In addition to the long-term monitoring of the East Basin production wells, residential wells along Old Garden River Road were also monitored during the 2010 Shannon Well and Lorna Wells pumping test. These residential wells are screened in the sandstone and are located approximately 5 km north of the municipal wells (Map 2-10). The water levels at these residential wells ranged from 196 masl to 203 masl, or approximately 2 m to 9 m below the ground surface. These residential wells are located at higher elevations, upgradient of the Shannon Well and Lorna Wells, hence the water level measured at these residential wells were also higher compared to the water levels at the municipal wells. The hydrographs of the residential wells are presented in Appendix C-5.

Groundwater levels in the East Basin CEG/Kresin monitoring wells (MW5D, MW5S, MW6D and MW6S) was monitored from 2010 to 2011. The groundwater levels in these wells were relatively shallow, ranging from 3 mbgs (177.5 masl) to 0.6 mbgs (179.4 masl). The range of water level observed at the East Basin CEG/Kresin monitoring wells are summarized in Table 2.17, the hydrographs of these monitoring wells are presented in Appendix B-3.

Table 2.17 Range of Observed Water Level at East Basin CEG/Kresin Monitoring Wells

Well	Monitoring Duration	Highest Water Level	Lowest Water Level	Overall Average Water Level
MW5D	2010 – 2011	179.3 masl/1.2 mbgs	177.5 masl/3.0 mbgs	178.3 masl/2.2 mbgs
MW5S	2010 – 2011	178.9 masl/1.6 mbgs	177.5 masl/3.0 mbgs	178.4 masl/2.1 mbgs
MW6D	2010 – 2011	178.1 masl/1.9 mbgs	178.6 masl/1.4 mbgs	178.3 masl/1.7 mbgs

MW6S	2010 – 2011	179.4 masl/0.6 mbgs	178.4 masl/1.6 mbgs	178.6 masl/1.5 mbgs

2.8.7 GROUNDWATER LEVELS IN THE CENTRAL BASIN

The municipal wells in the Central Basin are screened in the sandstone bedrock and show artesian flowing conditions when pumping ceases. The monthly average water level in these production wells ranged from 166 masl (24 mbgs) to 191 masl (0.7 m above ground surface). Similar trends were observed in the monthly average water levels in the Central Basin production wells, where the water levels were slightly higher after 1986. The ranges of monthly water levels at the Central Basin production wells are summarized in Table 2.18 and the hydrographs are presented in Appendix B-1. It is important to note that the measured water levels at the municipal wells were likely influenced by pumping activities and were lower than the static water levels. During the shutdown of the Steelton Well and Goulais Wells in 2011, the static water levels at the municipal wells were estimated using the recovery hydrographs, and the extrapolated water levels at the municipal wells ranged from 1 m to 2.5 m above ground surface, corresponding to elevations of 190 masl to 192 masl.

Table 2.18 Range of Observed Monthly Average Water Level at Central Basin Production Wells

Well	Monitoring Duration	Highest Monthly Average Water Level	Lowest Monthly Average Water Level	Overall Average Water Level
Goulais Well 1	1969 – 2012	190 masl/-0.3 mbgs	166 masl/24 mbgs	179 masl/11 mbgs
Goulais Well 2	1969 – 2012	191 masl/-0.7 mbgs	166 masl/24 mbgs	182 masl/8 mbgs
Steelton Well	1966 – 2012	189 masl/-0.1 mbgs	168 masl/21 mbgs	176 masl/13 mbgs

Water levels in two Central Basin residential wells (327 Glasgow Ave and 1500 Korah Rd) were monitored during the Steelton Well shutdown test in 2011. The residential well at 327 Glasgow Ave was screened in the sandstone and showed similar recovery pattern as the Steelton Well. The water levels at 327 Glasgow Ave well ranged between 188 masl (1 mbgs) and 191 masl (2 m above ground surface) throughout the monitoring duration (July 11, 2011 to October 27, 2011). Water levels measured at 1500 Korah Rd well were relatively constant throughout the shutdown period, at 189 masl (near ground surface). The well at 1500 Korah Rd was screened in the coarse grained overburden material shallower than the municipal well. The hydrographs of the two residential wells are presented in Appendix C-6.

Groundwater levels in the Central Basin shallow monitoring wells (MW1D, MW1S, MW2D, MW2S, MW3D, MW3S, MW4D, MW4S) were also monitored from 2010 to 2011. These shallow monitoring wells were screened in the clay overburden material, with depths ranging from 5 mbgs to 12 mbgs. The water level in these shallow monitoring wells ranged from near ground surface (0.1 m above ground surface) to 5.3 mbgs. The range of water level observed at the Central Basin CEG/Kresin monitoring wells are summarized in Table 2.19, the hydrographs of the monitoring wells are presented in Appendix B-3.

Table 2.19 Range of Observed Water Level at Central Basin CEG/Kresin Monitoring Wells

Well	Monitoring Duration	Highest Water Level	Lowest Water Level	Overall Average Water Level
MW1D	2010 – 2011	191.6 masl/0.4 mbgs	190.6 masl/1.4 mbgs	191.1 masl/0.9 mbgs
MW1S	2010 – 2011	190.9 masl/1.1 mbgs	186.7 masl/5.3 mbgs	190.2 masl/1.8 mbgs
MW2D	2010 – 2011	188.0 masl/0.3 mbgs	187.0 masl/1.3 mbgs	187.4 masl/0.9 mbgs
MW2S	2010 – 2011	187.7 masl/0.7 mbgs	186.5 masl/1.9 mbgs	186.8 masl/1.5 mbgs
MW3D	2010 – 2011	188.7 masl/0.5 mbgs	187.3 masl/1.8 mbgs	187.9 masl/1.2 mbgs
MW3S	2010 – 2011	189.3 masl/-0.1 mbgs	185.8 masl/3.3 mbgs	188.1 masl/1.1 mbgs
MW4D	2010 – 2011	188.8 masl/1.0 mbgs	187.2 masl/2.6 mbgs	187.9 masl/1.9 mbgs
MW4S	2010 – 2011	188.5 masl/1.4 mbgs	187.1 masl/2.7 mbgs	187.8 masl/2.1 mbgs

2.8.8 HORIZONTAL GROUNDWATER FLOW

The contoured piezometric surface of the lower sandstone aquifer is presented in Map 2-11. The equipotential contours were interpreted using water level measurements extracted from MOE water well records for wells with depth greater than 15 m. The equipotential contours indicate southward flow from recharge areas close to the Precambrian uplands to the centre of the East and Central Basins.

The contours are spaced relatively close together near the southern edge of the Precambrian uplands. The horizontal hydraulic gradient on the escarpment is approximately 0.05 m/m. The closer spacing of the contours can represent an area of reduced hydraulic conductivity or a significant component of vertical flow. Considering our current understanding of the system, the closely spaced contours are likely attributed to the significant vertical flow component near the southern edge of the Precambrian uplands. Further downstream in the basin areas, the horizontal groundwater gradient reduces to approximately 0.006 m/m.

2.8.9 VERTICAL GROUNDWATER FLOW

In the recharge zone along the margin of the Precambrian uplands, groundwater moves downward through a zone of granular deposits. Southward from the Precambrian uplands toward the St. Marys River, the groundwater vertical hydraulic gradient becomes negative (a negative gradient corresponds to upwards groundwater flow), which is consistent with the observed flowing wells close to the St. Marys River.

Natural vertical hydraulic gradients in the study area were quantified in three areas: the recharge zone, around the Steelton Well and Goulais Wells in the Central Basin, and around the Shannon Well in the East Basin. These natural gradient calculations are discussed below.

Downward Vertical Hydraulic Gradients in the Recharge Zone

Natural vertical hydraulic gradients in the recharge zone were estimated using groups (referred to here as “clusters”) of residential wells in close proximity and screened at different depths. These well clusters can be treated as a large group of nested wells, and the mean vertical hydraulic gradient can be estimated using data from the MOE well records.

Map 2-10 shows two clusters of residential wells, one in the East Basin (Cluster 1) and one in the Central Basin (Cluster 2). Both clusters are in or near the recharge zone, where relatively strong, downward natural gradients would be expected. These well clusters are also relatively far from the municipal wells in each basin, and the natural vertical hydraulic gradients are not significantly affected by municipal pumping.

Using data from the MOE well records for the wells in each cluster, the static water levels were plotted versus well depth, as shown in Figure 2-1 and

Figure 2-2. There are uncertainties inherent in the static level data, which are likely countered to some extent by the large number of wells used in each cluster, and by using a “best-fit”

interpolation technique. Despite the scatter caused by the uncertainties, and also by the lateral separation between the wells, the plots display linear trends that indicate a direction and mean magnitude of the vertical hydraulic gradient.

Figure 2-1 Estimation of Vertical Hydraulic Gradient in Recharge Zone (East Basin)

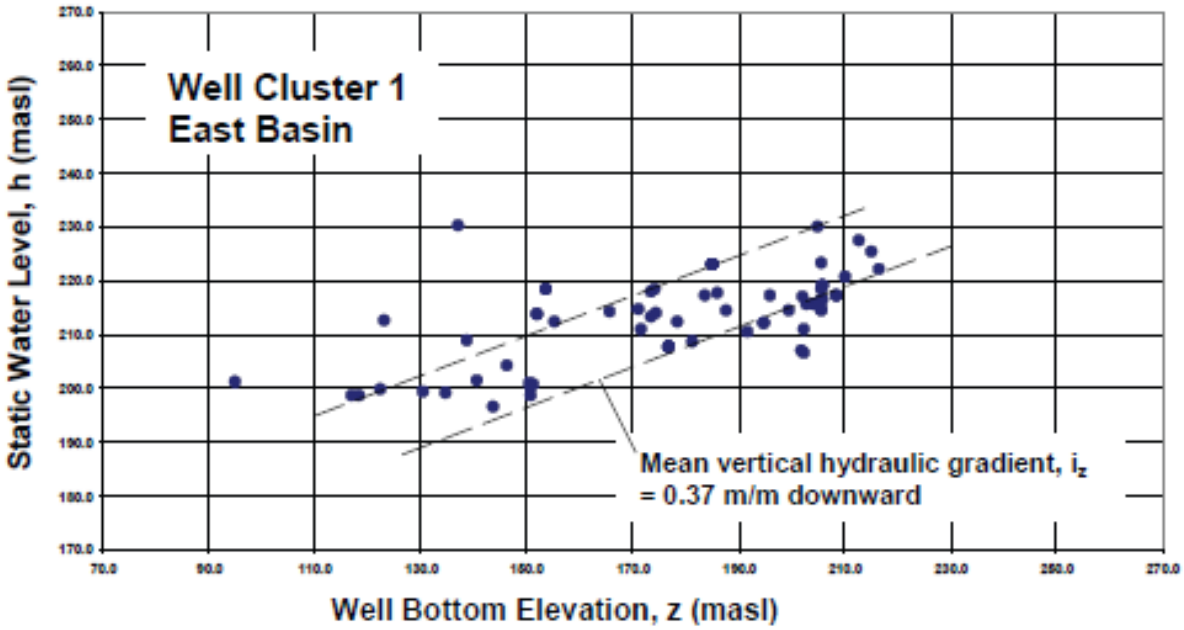
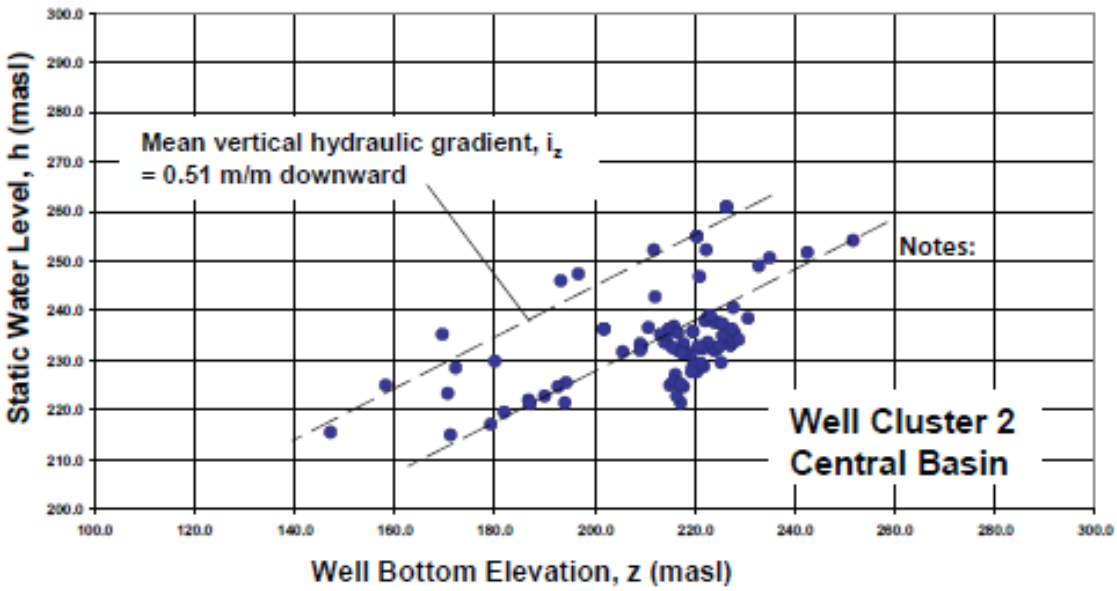


Figure 2-2 Estimation of Vertical Hydraulic Gradient in Recharge Zone (Central Basin)



The results for both clusters showed trends indicating downward vertical hydraulic gradients of about 0.4 m/m and 0.5 m/m. A mean downward vertical hydraulic gradient in the recharge zone is estimated at about 0.45 m/m.

Upward Vertical Hydraulic Gradients in the East Basin

In the East Basin, the natural vertical hydraulic gradient was assessed using the IWS original static levels measured in the Shannon Well, TW3/70 and TW4/70. All three wells are about 100 m from the St. Marys River and the gradient was calculated based on a mean river level of 177 masl below the locks. In this calculation, the bottom of the riverbed was estimated to be 157 masl.

The natural gradient between the river and the sand-gravel aquifer was upward with a magnitude of 0.01 m/m to 0.07 m/m. Calculations are shown in Table 2.20. These values are consistent with the natural gradient measured around the Steelton Well during the 2011 shutdown.

Table 2.20 Natural Vertical Hydraulic Gradients around Shannon Well, 1970 and 1972

River/Well Pair	Distance r (m)	Intake Midpoint	Prior to Operational Startup of Shannon Well	
			1970	1972

			Elevation (masl)	Water Level (masl)	Vertical Gradient Calculation			Water Level (masl)	Vertical Gradient Calculation		
	From River	From Shannon Well			Δz (m)	Δh (m)	iz (m/m)		Δz (m)	Δh (m)	iz (m/m)
Water Table	-	1440	-	180	80.2	na	na	180	60.2	0.44	-0.01
Shannon Well	1440	-	96.8	na				180.44			
St. Marys River	-	1440	-	177	26.2	1.92	-0.07	177	26.2	na	na
TW3/70	88	2694	130.8	178.92				na			
St. Marys River	-	1440	-	177	51.3	na	na	177	51.3	2.83	-0.06
TW4/70	112	1638	125.7	na				179.83			

Notes:
Start of pumping from Shannon Well was October 1974.
The St. Marys River elevation (177 masl) was obtained from 1:20,000 Ontario Base Maps.
The bottom of river bed elevation (157 masl) was used for vertical separation (Δz).
The water table elevation around the Shannon Well (180 masl) was estimated.
Groundwater level data were obtained from the IWS (1978).
na = Not available
Δz = Vertical separation between midpoints of intake intervals
Δh = Difference in hydraulic head
iz = Vertical hydraulic gradient (Δh/Δz)

Upward hydraulic gradient

Downward hydraulic gradient

Upward Vertical Hydraulic Gradients in the Central Basin

In the Central Basin, the natural vertical gradient was assessed around the Steelton Well, using water levels measured during the 2011 shutdown. Calculations are summarized in Table 2.21. During the shutdown period, vertical gradients observed showed an upward trend with a

magnitude of about 0.07 m/m. Table 2.21 shows similar upward vertical hydraulic gradients of 0.04 m/m and 0.06 m/m near the Goulais Well 1 (as measured in MW2D and MW3D).

Table 2.21 Natural Vertical Hydraulic Gradients around Goulais and Steelton Wells, During PUC Down Test, 2011

Zone	Well Pair	Top of Casing Elevation (masl)	Intake Midpoint Elevation (masl)	Shutdown Period 08 August 2011, 14:00-16:00			
				Water Level (masl)	Vertical Gradient Calculation		
					Δz (m)	Δh (m)	iz (m/m)
Shallow	MW2S	193.89	189.2	186.74	6.9	-0.49	-0.07
	MW2D	193.94	182.3	187.23			
	MW3S	194.08	189.2	187.86	6.8	-0.07	-0.01
	MW3D	193.98	182.4	187.93			
	MW4S	192.97	188.2	187.64	6.8	-0.84	-0.12
	MW4D	192.93	181.4	188.48			
Shallow Zone compared to GPW1 Pumped Zone	MW2D	193.94	182.3	187.23	44.9	-2.62	-0.06
	GPW1	190.20	137.4	189.85			
	MW3D	193.98	182.4	187.93	45.0	-1.92	-0.04
	GPW1	190.20	137.4	189.85			
	MW4D	192.93	181.4	188.48	25.4	-1.78	-0.07

Shallow Zone compared to SPW Pumped Zone	SPW	189.25	156.0	190.26			

Notes:

GPW1 = Goulais Production Well 1

SPW = Steelton Production Well

MW = Monitoring Well

Δz = Vertical separation between midpoints of intake intervals

Δh = Difference in hydraulic head

iz = Vertical hydraulic gradient (Δh/Δz)

Upward hydraulic gradient

Downward hydraulic gradient

2.8.10 AQUIFER PROPERTIES IN THE EAST BASIN

During the Shannon Well and Lorna Wells Pumping Test conducted in 2010, the water levels in both municipal wells exhibited a response to the pumping and declined. At the end of pumping, water levels rebounded and asymptotically approached the assumed static level. A similar pattern was also observed in the Shannon and Lorna monitoring wells (Appendix C-5). Unfortunately, the duration of the pumping test was insufficient for the water levels to stabilize; however, the water level responses collected demonstrate drawdowns characteristic of confined aquifer systems.

The CEG/Kresin monitoring wells (MW5S, MW5D, MW6S and MW6D), which are screened in the shallow geologic units, did not show any change in water level associated with the pumping test. This supports the understanding that the clay unit acts as a competent confining layer between the shallow system and the deep sand and gravel aquifer, in which the Shannon Well and Lorna Wells are screened. Alternatively, the duration of the test may have been insufficient for any observable response.

During the Shannon Well and Lorna Wells Pumping Test, some residential wells along Old Garden River Road were monitored. Based on the cross-section B-B' (Map 2-8B), these wells (MOE well ID 1106061 and 1104203) were screened in the sandstone bedrock, which is likely hydraulically connected with the sand and gravel aquifer. These residential wells did not exhibit any response to pumping of the Shannon and Lorna Wells (Appendix C-5).

Hydraulic Parameters

Analyses were conducted on the data collected during the CEG/Kresin Shannon Well and Lorna Wells Pumping Test, 2010 from the various monitoring wells. Comparison of the estimated transmissivities calculated using the Cooper-Jacob method from the monitoring wells show mostly consistent transmissivity values. The estimated transmissivity (T) for the sand and gravel aquifer was found to range from 3.7×10^{-3} m²/s to 6.9×10^{-2} m²/s with an estimated geometric mean of 1.5×10^{-2} m²/s. The hydraulic conductivity (K) values range from 2.0×10^{-4} m/s to 3.7×10^{-3} m/s with an estimated geometric mean of 1.3×10^{-3} m/s. Two storativity (S) values for the sand and gravel aquifer were obtained in the 2010 pumping test, which ranged from 2.0×10^{-2} to 1.3×10^{-1}, with a geometric mean of 2.0×10^{-2}.

Eight T values were also analyzed from historical IWS reports for the sand and gravel aquifer. The values ranged from 2.9×10^{-4} m²/s to 1.7×10^{-2} m²/s. The corresponding K values obtained from historical tests ranged from 8.5×10^{-5} m/s to 2.9×10^{-3} m/s. The geometric mean of the T values was estimated to be 5.5×10^{-3} m²/s and the geometric mean of the K values was estimated to be 4.5×10^{-4} m/s. Five S values, obtained from historical reports, ranged from 3.9×10^{-5} to 5.5×10^{-2}, with a geometric mean of 8.5×10^{-4}.

The geometric means of all the 12 T values, 12 K values and seven S values obtained for the sand and gravel aquifer in the East Basin were estimated as 7.7×10^{-3} m²/s, 6.4×10^{-4} m/s, and 2.1×10^{-3}, respectively.

While the computed geometric mean of the seven S values is generally consistent with S values that are typical of confined aquifers, some of the individual S values that were obtained as part of these analyses are uncharacteristically high for confined aquifer systems. The reason for this phenomenon is unknown; however, may be the physical manifestation of the following:

1. Aquifer heterogeneity;
2. Contribution of water from the overlying clay unit;
3. Contribution of water from the underlying sandstone aquifer; or
4. A combination of the above.

Any of the above scenarios results in a conflict with the fundamental assumptions in calculating the S value (homogeneous and isotropic aquifer, overlain and underlain by impermeable aquitards, infinite extent of aquifer and almost incompressible aquifer). Radial flow is no longer horizontal only and partial penetration effects can sometimes be evident in estimating the S value. A summary of the hydraulic testing results in the East Basin is presented in Table 2.22. The range and geometric mean aquifer parameters are presented in Table 2.24.

Table 2.22 Results of Hydraulic Tests in the East Basin (1970 to 2010)

Data Source	Report Appendix	Test Date	Test Type	Pumped Well(s)	Pumping Rate (m3/d)	Pumping Time	Tested Aquifer	Monitored Well	Analytical Results				
									Analysis Type	T (m^2/s)	b (m)	K (m/s)	S
CEG, 2010+	C-5	Dec-10	Pumping Test	Shannon and Lorna PW	12770 (combined)	5 days	Sand, gravel	Lorna PW2	Time-Drawdown	3.7E-03	18.6	2.0E-04	na
								Lorna MW		6.9E-02	na	3.7E-03	2.0E-02
								Shannon PW		1.5E-02	6.7	2.2E-03	na
								Shannon MW		1.3E-02	na	1.9E-03	2.0E-02
IWS, 2002	C-3	Sep-77	Pumping Test	Lorna PW	7855	1 day		Lorna PW1	Time-Drawdown	9.8E-03	18.6	5.3E-04	na
								TW2/75		8.3E-03	na	4.5E-04	2.0E-02
								TW3/70		5.4E-03	na	2.9E-04	1.4E-04
								TW1/71		1.4E-02	na	7.5E-04	7.3E-05
								All	Distance-Drawdown	1.7E-02	na	9.1E-04	3.9E-05

Table 2.22 Results of Hydraulic Tests in the East Basin (1970 to 2010)

Data Source	Report Appendix	Test Date	Test Type	Pumped Well(s)	Pumping Rate (m3/d)	Pumping Time	Tested Aquifer	Monitored Well	Analytical Results				
									Analysis Type	T (m^2/s)	b (m)	K (m/s)	S
IWS, 1978	C-3	1972 to 1977	Long-Term Drawdown Response	Shannon PW	7626	2.9 years		Shannon PW	Distance-Drawdown	2.8E-03	20*	1.4E-04	5.5E-02
								TW1/71					
								TW2/75					
								TW3/70					
								TW4/70					
								Well 1217					
								Well 1028					
IWS, 1971	C-2	Aug-70	Pumping Test	TW2/70	131	4 hours		TW2/70	Time-Drawdown	2.9E-04^	3.4	8.5E-05	na
		Aug-70		TW3/70	281	8 hours		TW3/70		9.8E-03^	3.4	2.9E-03	na
		Oct-70		TW5/70A	2062	8 hours		TW5/70		1.6E-03^	3.0	5.3E-04	na

Table 2.22 Results of Hydraulic Tests in the East Basin (1970 to 2010)

Data Source	Report Appendix	Test Date	Test Type	Pumped Well(s)	Pumping Rate (m3/d)	Pumping Time	Tested Aquifer	Monitored Well	Analytical Results				
									Analysis Type	T (m^2/s)	b (m)	K (m/s)	S
IWS, 1971	C-2	Oct-70	Pumping Test	TW5/70	2062	1 day	Upper sandstone / sand, gravel combined	TW5/70 Burmaster Well TW3/70	Distance-Drawdown	5.6E-03	8.6	6.5E-04	1.7E-04
IWS, 1979	C-3	Nov-78	Pumping Test	TW1/78	1375	8 hours	Upper sandstone	TW1/78 Burmaster Well	Time-Drawdown	2.4E-03 to 3.3E-03^	3.0	8.0E-04 to 1.1E-03	2.9E-06^

Notes:

IWS = International Water Supply Ltd.

PW = Production well

MW = Observation well

TW = Test well

Table 2.22 Results of Hydraulic Tests in the East Basin (1970 to 2010)

Data Source	Report Appendix	Test Date	Test Type	Pumped Well(s)	Pumping Rate (m3/d)	Pumping Time	Tested Aquifer	Monitored Well	Analytical Results				
									Analysis Type	T (m^2/s)	b (m)	K (m/s)	S

na = Not available

T = Transmissivity

b = Length of screened interval (short tests); thickness of aquifer (long-term pumping)

K = Hydraulic conductivity. K values were approximated from corresponding T and b values. For monitoring/observation wells where b values were not available, K values were estimated based on the b value for the corresponding pumping well.

S = Storativity (dimensionless)

* Mean thickness of aquifer around Shannon Well, based on Cross-Section B-B'

^ Values were extracted from the text of the original test reports which are included in the corresponding appendix.

+ Based on Driscoll (1986), it was inferred that the time-drawdown graphs showed effects of impervious boundaries due to a steepening of the time-drawdown and the illustration of two slopes. As such, the S and T values were calculated from the early test data as per Driscoll (1986).

Table 2.23 Results of Hydraulic Tests in the Central Basin (1967 to 2011)

Data Source	Report Appendix	Test Date	Test Type	Pumped Well(s)	Pumping Rate (m3/d)	Pumping Time	Tested Aquifer	Monitored Well	Analytical Results				
									Analysis Type	T (m^2/s)	b (m)	K (m/s)	S
PUC, 2012	C-6	Jul-11 to Aug-11	Shutdown Test	Steelton PW	6912	14 days*	Upper sandstone	Steelton PW	Time-Recovery	4.1E-03	19.6	2.1E-04	na
								Steelton MW1		3.9E-03	na	2.0E-04	1.1E-01
								Steelton MW2		3.7E-03	na	1.9E-04	2.1E-02
								Goulais MW		3.9E-02	na	2.0E-03	5.3E-04
Burnside, 2003	C-4	Sep-02	Shutdown Test	Steelton PW	5773	30 days*		Steelton PW	Time-Recovery	4.6E-03	19.6	2.3E-04	na
								Steelton MW		4.6E-03	na	2.3E-04	3.7E-02
				Goulais PW2	5095			Goulais PW2		6.3E-03	6.4	9.8E-04	na

Table 2.23 Results of Hydraulic Tests in the Central Basin (1967 to 2011)

Data Source	Report Appendix	Test Date	Test Type	Pumped Well(s)	Pumping Rate (m3/d)	Pumping Time	Tested Aquifer	Monitored Well	Analytical Results				
									Analysis Type	T (m^2/s)	b (m)	K (m/s)	S
								Goulais MW		6.3E-03	na	9.8E-04	1.5E-03
IWS, 1978	C-3	Nov-78	Pumping Test	TW2/78	1140	6 hours		TW2/78	Time-Drawdown	6.9E-03 to 7.2E-03^	3.0	2.3E-03 to 2.4E-03	na
								TW3/78 (served as observation well)			3.0		3.2E-04^
		Nov-78	Pumping Test	TW4/78	1308	8 hours		TW4/78		7.0E-03 to 9.1E-03^	3.0	2.3E-03 to 3.0E-03	na
								TW2/78 (served as observation well)		7.6E-03^	3.0	2.5E-03	1.0E-04^
OWRC, 1969	C-1	Jul-68	Pumping Test	Goulais PW1	8568 (combine	19 days		Goulais PW2	Time-Drawdown	5.3E-03	6.4	8.3E-04	na

Table 2.23 Results of Hydraulic Tests in the Central Basin (1967 to 2011)

Data Source	Report Appendix	Test Date	Test Type	Pumped Well(s)	Pumping Rate (m3/d)	Pumping Time	Tested Aquifer	Monitored Well	Analytical Results				
									Analysis Type	T (m²/s)	b (m)	K (m/s)	S
				Goulais PW2	d initial rate); 6600 (combined reduced rate)			Goulais PW1		4.7E-03	6.1	7.7E-04	na
								TW1/65		5.7E-03	na	9.3E-04 to 8.9E-04	1.0E-03
								Allen's Side Road (TW1/66)		1.1E-02	na	1.8E-03 to 1.7E-03	7.6E-06
								All	Distance-Drawdown	4.0E-03	na	na	9.3E-04

Notes:
PUC = Sault Ste. Marie Public Utilities Commission
Burnside = R.J. Burnside & Associates Ltd.
IWS = International Water Supply Ltd.
OWRC = Ontario Water Resource Commission
PW = Production well
MW = Observation well
TW = Test well
na = Not available

Table 2.23 Results of Hydraulic Tests in the Central Basin (1967 to 2011)

Data Source	Report Appendix	Test Date	Test Type	Pumped Well(s)	Pumping Rate (m3/d)	Pumping Time	Tested Aquifer	Monitored Well	Analytical Results				
									Analysis Type	T (m²/s)	b (m)	K (m/s)	S

T = Transmissivity

b = Length of screened interval (short tests); thickness of aquifer (long-term pumping)

K = Hydraulic conductivity. K values were approximated from corresponding T and b values. For monitoring/observation wells where b values were not available, K values were estimated based on the b value for the corresponding pumping well.

S = Storativity (dimensionless)

* For shutdown tests or long-term response analyses, the pumping time is the period prior to the test over which the pumping rate (Q) value applies.

^ Values were extracted from the text of the original test reports which are included in the corresponding appendix.

Table 2.24 Summary of Transmissivity and Storativity Results of Hydraulic Tests in the East and Central Basins

Basin	Data Source	TRANSMISSIVITY, T (m²/s)						STORATIVITY, S						HYDRAULIC CONDUCTIVITY, K (m/s)					
		Sand and Gravel			Upper Sandstone			Sand and Gravel			Upper Sandstone			Sand and Gravel			Upper Sandstone		
		N	Range	Mean	N	Range	Mean	N	Range/ Value	Mean	N	Range/ Value	Mean	N	Range/ Value	Mean	N	Range/ Value	Mean
East	Historical	8	2.9E-04 to 1.7E-02	5.5E-03	2	2.4E-03 to 3.3E-03	2.8E-03	5	3.9E-05 to 5.5E-02	8.5E-04	1	2.9E-06	2.9E-06	8	8.5E-05 to 2.9E-03	4.5E-04	2	8.0E-04 to 1.1E-03	9.4E-04

	CEG	4	3.7E-03 to 6.9E-02	1.5E-02	-	-	-	2	2.0E-02 to 1.3E-01	2.0E-02	-	-	-	4	2.0E-04 to 3.7E-03	1.3E-03	-	-	-
Central	Historical	-	-	-	14	4.0E-03 to 1.1E-02	6.2E-03	-	-	-	7	7.6E-06 to 3.7E-02	5.4E-04	-	-	-	15	2.3E-04 to 3.0E-03	1.2E-03
	CEG*	-	-	-	4	3.7E-03 to 3.9E-02	6.9E-03	-	-	-	3	5.3E-04 to 1.1E-01	1.1E-02	-	-	-	4	1.9E-04 to 2.0E-03	3.6E-04
Total		12	2.9E-04 to 6.9E-02	7.7E-03	20	2.4E-03 to 3.9E-02	5.9E-03	7	3.9E-05 to 1.3E-01	2.1E-03	11	2.9E-06 to 1.1E-01	7.5E-04	12	8.5E-05 to 3.7E-03	6.4E-04	21	1.9E-04 to 3.0E-03	9.0E-04

Notes:

N = number of measurements

Mean = geometric mean

* Data were collected by CEG during the PUC shutdown test, 2011.

2.8.11 CENTRAL BASIN UPPER SANDSTONE AQUIFER

During the period of the Steelton Well shutdown test (July 4, 2011 to August 8, 2011), the water levels in the Steelton Well rebounded to a static level of approximately 190 masl. After the pump was restarted, drawdown was observed immediately and the water level approached a constant value (~186 masl).

The immediate response in drawdown due to the pumping is representative of a confined aquifer system. A similar pattern was observed in Steelton Monitoring Well 2.

The CEG/Kresin monitoring wells, which are screened in the shallow geologic units, did not show any change in water level associated with the pumping test. This suggests that there is a competent confining layer between the shallow water table and the sandstone aquifer or the duration of the test was insufficient for any observable response.

Hydraulic Parameters

The hydraulic tests conducted in the sandstone aquifer are summarized in Table 2.24. Four T values were obtained from the 2011 Steelton Well shutdown test, which ranged from 3.7×10^{-3} m^2/s to 3.9×10^{-2} m^2/s. The corresponding four K values were found to range from 1.9×10^{-4} m/s to 2.0×10^{-3} m/s.

Analyzing historical tests (Burnside, 2003; IWS, 1979; and OWRC, 1969) provided 14 other T values ranging from 4.0×10^{-3} m^2/s to 1.1×10^{-2} m^2/s for the sandstone in the Central Basin, and two other T values ranging from 2.4×10^{-3} m^2/s to 3.3×10^{-3} m^2/s for the sandstone in the East Basin. The geometric mean of all 20 T values for the sandstone was estimated as 5.9×10^{3} m^2/s.

15 other K values were obtained from the historical tests, ranging from 2.3×10^{-4} m/s to 3.0×10^{-3} m/s. This range of K values was found to be similar to that inferred from the shutdown test. Two K values were also obtained from the IWS, 1979 pumping test for the sandstone in the East Basin, which range from 8.0×10^{-4} m/s to 1.1×10^{-3} m/s. The geometric mean of all these 21 K values was estimated as 9.0×10^{-4} m/s.

Three estimates of S were obtained from the 2011 shutdown test which range from 5.3×10^{-4} to 1.1×10^{-1} for the upper sandstone in the Central Basin. Seven estimates of storativity were obtained from historical tests, ranging from 7.6×10^{-6} to 3.7×10^{-2}. In addition, one S value was obtained from the IWS 1978 pumping test for the sandstone in the East Basin, 2.9×10^{-6}. The geometric mean of all these 11 S values for the upper sandstone was estimated as 7.5×10^{-4}.

Similarly to the analyses conducted for the East Basin, while the computed geometric mean of the 11 S values is generally consistent with S values that are typical of confined aquifers, some of the individual S values that were obtained as part of the analyses for the upper sandstone are uncharacteristically high for confined aquifer systems. The reason for this phenomenon is unknown; however, may be the physical manifestation of the following:

1. Aquifer heterogeneity;
2. Contribution of water from the overlying clay unit;

3. Contribution of water from the underlying sandstone aquifer; or
4. A combination of the above.

Any of the above scenarios results in a conflict with the fundamental assumptions in calculating the S value (homogeneous and isotropic aquifer, overlain and underlain by impermeable aquitards, infinite extent of aquifer and almost incompressible aquifer). Radial flow is no longer horizontal only and partial penetration effects can sometimes be evident in estimating the S value. Furthermore, the large range in S values indicates that the wells are screened in bedrock with varying fracture patterns (in which areas with a lower S value represent tight bedrock, and areas with a large S value represent fractured bedrock) and that the aquifers are inherently heterogeneous.

This reduction in permeability and well yield is likely controlled by decreasing fracture apertures and spacing with depth.

3.0 CONCEPTUAL MODEL

The conceptual surface water and groundwater understanding of the Sault Ste. Marie area has evolved during the last few decades through studies conducted by the OWRC (1969), the IWS (1971, 1978, 2002), Burnside (2003), Kresin & MacViro (2006), Kresin & GENIVAR (2008), and Kresin & CEG (2012). In this section, the information obtained from the previous work has been assessed to develop a comprehensive conceptual model in order to provide the physical framework to be used in the development of the numerical model. Steps critical in building the conceptual model includes:

- Interpretation of the regional geology and hydrogeology through review of geological and water resources information in Ontario and Michigan.
- Validating the hydrogeological understanding through review of:
  - MOE Water Well Records;
  - all recent pumping test data;
  - land development and water usage information;
  - existing groundwater levels; and
  - historical groundwater levels.
- Conducting pumping and aquifer recovery tests in 2012, including a monitoring program to assess stress response conditions within the groundwater/ surface water systems.

The following sections provide an overview of the conceptual models of the groundwater and surface water systems and their interactions as observed through this study. The conceptual model has been used as a foundation for the development of the numerical models.

3.1 CONCEPTUAL SURFACE WATER MODEL

The SSMR Source Protection Area consists of two distinct landforms. The northern portion is referred to as “Precambrian uplands”. South of this region is the relatively flat lying area referred to as the “lowlands”. Drainage is via a series of streams flowing southward off the Precambrian uplands, across the lowlands to the St. Marys River.

The SSMR Source Protection Area consists of twelve subwatersheds with each independently draining into the St. Marys River or Lake Superior (Map 2-4). In the East Basin, the largest stream is the Root River/Crystal Creek system, which originates in the Precambrian uplands and meanders across the lowlands and ultimately discharges to the St. Marys River. The streams in the Central Basin include the East Davignon Creek, West Davignon Creek and Bennett Creek.

In the conceptual model, the surface water system has been characterized by taking into consideration the local land use, land cover, and surficial geology to assess the potential infiltration, runoff and evapotranspiration. For example, a woodland area would typically receive a moderate amount of infiltration; however, since in this case it is located in the Precambrian uplands, an area underlain by intact bedrock material, limited infiltration would occur in this area.

The runoff generated on the Precambrian uplands area flows downgradient through the streams, overland, or through the shallow soils in a southerly direction and continues into the lowlands. Some of the flow infiltrates into the groundwater system through the thick sand and gravel beach deposits located along the southern edge of the Precambrian uplands. Areas of localized groundwater discharge are also observed at/near this interface between the upland and lowlands as a result of potential perched water table associated with localized low hydraulic conductivity zones and/or water table rising above the surface seasonally resulting in discharge in topographically low areas.

The rates of infiltration estimated in the lowlands are relatively low, since the lowlands are mostly covered by glaciolacustrine deep water deposits consisting of fine-grained materials (silt and clay). A significant portion of the lowlands in the Central Basin and East Basin has been urbanized. As a result, the amount of infiltration to groundwater aquifers is expected to be low and the amount of runoff and contribution to the surface water system is relatively higher. Details of the surface water model are provided in the Local Area Risk Assessment report submitted under a separate cover.

3.2 CONCEPTUAL GROUNDWATER MODEL

The conceptual groundwater model shows no mapped aquifers in the Precambrian uplands. Based on the bedrock topography, the groundwater system in the lowlands is divided into three major hydrogeological units, the “West Basin”, “Central Basin”, and “East Basin”. The stratigraphic sequence in all three hydrogeological units in the lowlands area is comprised of a relatively thick clay-rich overburden consisting of glaciolacustrine clays underlain by a layer of coarse-grained glaciolacustrine overburden deposits and the Jacobsville Formation. The deep sand and gravel aquifer is interconnected with the sandstone aquifer, and forms the regional aquifer formation which supplies the municipal wells and other private wells in the Sault Ste. Marie Area. The sandstone aquifer is confined by the fine-grained glaciolacustrine silt and clay deposits in the lowlands area. The groundwater basins are physically separated by bedrock ridges.

The main source of recharge for the aquifers is through the band of coarse-grained sand and gravel deposits immediately south of the Precambrian uplands. The potentiometric surface maps show that the groundwater flows southerly towards the St. Marys River. Based on the interpreted Cross-Sections A-A’ and F-F’ (Map 2-8A to Map 2-8F), the band of coarse-grained glaciofluvial and glaciolacustrine deposits may be hydraulically connected with the underlying sandstone aquifer and has been identified as the “high potential groundwater recharge area” (Map 2-10).

The overburden thickness of the glaciolacustrine clays overlying the aquifer units range from 25 m in the Central Basin to over 50 m in the East Basin. Pumping tests and recovery data confirm the confined nature of the regional sand and gravel aquifers.

Analysis of available geological data suggests that the sandstone aquifer intersects the base of the St. Marys River in the Central Basin and partially in the East Basin. The relationship between the St. Marys River and the regional aquifer is uncertain; however, piezometric levels observed

in the aquifer near the shoreline of the St. Marys River are generally artesian in nature, suggesting the potential for discharge from the confined aquifer to the St. Marys River under natural conditions. Details regarding the groundwater and surface water interaction are discussed further in the Local Area Risk Assessment report submitted under a separate cover.

3.3 GROUNDWATER RECHARGE DISTRIBUTION

In the SSMR Source Protection Area, the majority of the groundwater recharge is expected to occur at the sand and gravel outcrop to the south of the Precambrian uplands. Limited groundwater recharge is expected at the Precambrian uplands due to impervious bedrock and thin overburden material. In the lowlands, groundwater recharge is also expected to be low due to the thick, impervious clay and near-neutral to upward hydraulic gradients.

4.0 WATER DEMAND

One of the key objectives of a water balance is to assess whether the water demand within the watershed or groundwater basin is sustainable. Estimates of consumptive water demand are relevant in water budget assessments to identify areas that may be under hydrologic stress. Consumptive demand is the amount of water that is taken from a water source, and not returned locally to the same source of water within a reasonable amount of time.

Water demand within the SSMR Source Protection Area includes a variety of uses:

- Individual/Domestic;
- Municipal/Public;
- Commercial/Industrial;
- Agricultural; and
- Ecosystem/Recreational.

For the purposes of this investigation, the focus will be placed on evaluating groundwater demands for municipal water supply. All municipal groundwater supply is considered 100% consumptive.

4.1 MUNICIPAL WATER SYSTEMS

4.1.1 EXISTING SYSTEMS

The municipal water supply is managed by the Sault Ste. Marie Public Utilities Commission (PUC). The PUC provides water to the public, commercial and industrial sectors within the municipal services boundary of Sault Ste. Marie and to an area of the Rankin Reserve of the Batchewana First Nation (Map 1-1). Approximately half of the water is supplied from Lake Superior. The remainder of the water is obtained from municipal wells.

4.1.1.1 Surface Water

Gros Cap

The water intake in Lake Superior is located at Gros Cap. It extends 860 m into Lake Superior and is situated at a depth of 17 m. The Gros Cap pumping station delivers water to the Marshall Drive control tanks which then flows into the water treatment plant. The current permitted maximum surface water taking rate at the Gros Cap intake is 75,000 m3/day. The filtration plant is rated at 40,000 m3/day (PUC, 2011).

Lake Superior is a part of the Great Lakes system and is not a part of the watershed. As per the Technical Rules: Assessment Report, water takings from the Great Lakes are not to be considered as part of the water budget at this time.

4.1.1.2 Groundwater

The remaining municipal water supply is obtained from six wells located in the urban areas of the city. The six municipal wells obtain water from the Jacobsville Formation in the Central Basin and overlying sand and gravel unit in the East Basin. There are two (2) wells at the Goulais Well Site and one (1) well at the Steelton Well Site located in the Central Basin. In the East Basin, there are two (2) wells at the Lorna Well Site and one (1) well at the Shannon Well Site. Table 4.1 presents a summary of the municipal well details. The locations of the municipal wells are presented in Map 1-1. Copies of the well logs are provided in Appendix A-2 and associated Permits to Take Water are provided in Appendix D-1. Brief descriptions of the well fields are presented in the following sections.

Table 4.1 Summary of Municipal Wells

Basin	Well	Installation Year	Northing	Easting	Ground Elevation (masl)	Well Depth (m)	Screened Formation	Screen Depth (m)	Permitted Rate (m3/day)
Central Basin	Goulais Well No. 1	1952	5156958	700733	189.52	55.2	Sandstone	49.1 -55.2	6,606
	Goulais Well No. 2	1952	5156958	700737	189.59	54.9		48.8-54.9	3,407
	Steelton Well	1934	5157062	701671	189.22	43.0		23.3-43.0	8,208
East Basin	Shannon Well	1973	5156234	710260	195.08	100.6	Sand, gravel	94.3-100.6	7,000
	Lorna Well No. 1	1977	5154317	710362	183.15	76.2		56.7-76.2	7,279
	Lorna Well No. 2	1982	5154321	710364	182.90	75.3		56.7-75.3	7,279

Notes:
Well coordinates are in UTM NAD 83, Zone 16 N coordinate system.
Well coordinates were approximated based on the location of the well relatively to the well house on the aerial photo.
Ground elevation was extracted from the well logs provided by the PUC (Appendix A-2).
'Permitted Rate' refers to the maximum allowable daily water taking, as specified on the PTTW (Appendix D-1).

Goulais Wells

The Goulais Wells pumping station is located at Lot 19, McCaig Subdivision, 8 Hare Avenue, on the north side of Hare Avenue, west of Goulais Avenue. The UTM coordinates for the Goulais Well 1 and Goulais Well 2 are summarized in Table 4.1, the municipal well locations are shown on Map 2-10B.

This system consists of two (2) identical 25 cm (10”) inner diameter (ID) wells which extend around 55 m (181 ft) deep. The upper 41.75 m (137 ft) is comprised of a 40 cm (16”) outer diameter (OD) casing. The well screens are 6 m (20 ft) long, constructed out of stainless steel, with No. 1 opening. The wells are screened across the sandstone aquifer.

Figure 4-3 and Figure 4-4 provide schematic diagrams of the Goulais Well No. 1 and Goulais Well No. 2, respectively.

The Goulais Wells were constructed in 1952. Typically, only one well is in operation at a time; however, both wells are permitted to operate concurrently.

Steelton Well

The Steelton Well pumping station is located at 391 Second Line West, on the south side of Second Line West, east of First Ave. The UTM coordinates of the well are presented in Table 4.1. The location of the Steelton Well is shown on Map 2-10B.

The well extends 43 m (141 ft) deep. The upper 23.3 m (76.6 ft) is comprised of a 60 cm (24”) OD casing. The well screen is approximately 6 m (20 ft) long and is followed by a 6” diameter open hole from approximately 28.3 m (93 ft) to 43 m (141 ft). Both the screened portion and the open hole portion of the well are within the sandstone aquifer which supplies water to the well. Figure 4-5 provides a schematic diagram of the Steelton Well.

The Steelton well was constructed in 1934.

Shannon Well

The Shannon Well pumping station is located at the southeast portion of Lot 13, River Range, South of Highway 17 East and East of Dacey Road. The UTM coordinates of the Shannon Well are presented in Table 4.1.

The well consists of a 30 cm (12”) ID well which extends 100.6 m (330 ft) deep. The upper 89 m (292 ft) is comprised of a 60 cm (24”) OD casing. The well screen is 6 m (20 ft) long, 30 cm (12”) diameter, 50 slot stainless steel. The well is screened across a unit of gravel and sand between 94.3 m (310 ft) and 100.6 m (330 ft). Figure 4-6 provides a schematic diagram of the Shannon Well.

The Shannon Well was constructed in 1973.

Lorna Wells

The Lorna Wells pumping station is located at Lot 61 in the Lawrence Subdivision, near Queen Street East and Lorna Drive. The UTM coordinates for the Lorna Wells are presented in Table 4.1. The location of the Lorna Wells are shown on Map 2-10A.

This system consists of two (2) 40 cm (16”) ID wells which extend to approximately 75 m (247 ft) deep. The upper 53 m (174 ft) is comprised of a 60 cm (24”) OD casing. The well screens are 20 m (66 ft) long, constructed with 25 cm (10”) diameter stainless steel with 20 slots. The wells are screened across a unit of sand and gravel. Additionally, Lorna Well No. 2 comprises a steel liner

which was installed in November 2005 due to casing corrosion. The steel liner has a diameter of 30 cm (12”) down to a depth of approximately 49 m (160 ft), after which point its diameter is reduced to 25 cm (10”) for the steel liner to fit in the previously grouted 30 cm (12”) PVC liner. Figure 4-7 and Figure 4-8 provide schematic diagrams of the Lorna Well No. 1 and Lorna Well No. 2, respectively.

Lorna Well No. 1 was constructed in 1977 and Lorna Well No. 2 was constructed in 1982. Typically, only one well is in operation at a time; however, both wells are permitted to operate concurrently.

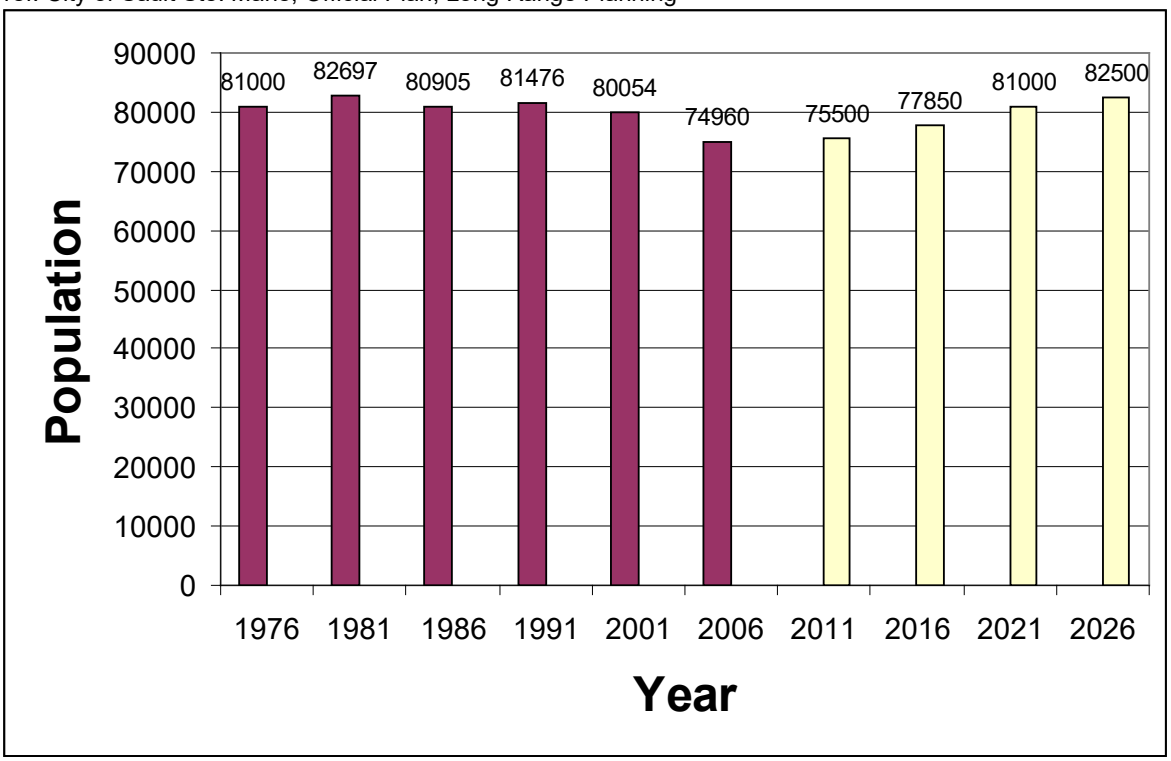
4.1.2 PLANNED SYSTEMS

The City of Sault Ste. Marie Official Plan was prepared in 1996 and was revised in 2006. It is currently under review. As part of the Tier 3 Study, a projection for population growth was considered in order to estimate future water demands. Figure 4-1 illustrates the historical population trends and population projections to 2026 which were generated as part of the recent Official Plan review process. Long-term population projections are expected to be similar to historical levels observed since 1976. Since no long-term growth is anticipated, the PUC is not planning to expand the water supply system. The PUC is currently conducting an assessment of the long term water supply demands and servicing requirements to achieve firm capacity. Firm capacity is defined as the ability of the system to meet demands in the event that the largest water source (the surface water supply in this case) is removed for an extended period of time.

In addition to this program, a series of maintenance programs have been conducted for improved well efficiency.

Figure 4-1 Historical Population Trends and Population Projections

*ref: City of Sault Ste. Marie, Official Plan, Long Range Planning



A review of the Land Use and Zoning mapping from Zoning-By-Law 2005-150 issued in October 2005 suggests that the land use in the SSMR SPA is not changing significantly. Certain areas within the core of the City are to be revitalized and there is some infilling; however, this is not expected to result in a significant change to the overall land use.

4.1.3 WELL OPERATIONS AND MAINTENANCE

The PUC manages the well operations and maintenance. The performance of the Goulais Wells and Lorna Wells were assessed in 2012 by the IWS and upgrades and maintenance to the pumps were conducted. However, overall, the Goulais and Lorna wells were performing effectively. A well work program was also conducted by Well Initiatives Limited at the Shannon Well for pump servicing in November 2005. As part of the program, well maintenance was conducted and a pump upgrade from an oil-lubricated system to a water-lubricated system was implemented. The Steelton Well was inspected more recently in October 2013 by the IWS. Well performance was determined to be effective and improved compared to historic production records, and no well rehabilitation was deemed necessary to improve well performance. However, recommendations were made for the installation of a casing liner in the cased portion of the well to provide medium to long-term security given the age of the well. Records of the upgrades and maintenance activities are provided in Appendix D-2. The Lorna Wells have also had historical issues with bio-fouling from iron-producing bacteria and were treated by chlorination. A Well Operations and

Maintenance Program which involves the collection and regular review of pumping rates and drawdown data to assess well efficiency is currently being reviewed.

4.2 MUNICIPAL WATER DEMAND

4.2.1 HISTORICAL AND EXISTING DEMAND

The existing demand is defined as the average pumping during the study period. However, maximum monthly and daily production should also be estimated from historical trends for the study period.

The PUC maintains pumping records for the water takings. An assessment of pumping rates was conducted on water takings between 2000 and 2012 to represent the existing demand. The existing demand was estimated as the average of the yearly mean daily pumping rates at each of the four municipal well locations (Goulais, Steelton, Shannon and Lorna) and the water treatment plant (WTP) over the 2000-2012 period. Table 4.2 provides the details of the existing demand rates estimation.

Table 4.2 Determination of Existing Demand Rates

Year	Daily Pumping Rate (m3/day)				
	WTP*	Goulais Wells	Steelton Well	Shannon Well	Lorna Wells
2000	19,886	6,679	6,526	2,479	5,063
2001	19,936	5,638	6,501	3,317	4,836
2002	19,544	4,702	6,285	3,885	4,733
2003	23,710	3,825	5,344	3,110	3,658
2004	19,417	4,903	5,927	4,547	2,007
2005	20,062	5,125	6,072	2,739	3,619
2006	17,848	5,289	6,217	2,099	5,049
2007	16,802	4,998	5,768	4,903	3,383
2008	19,706	4,211	4,113	4,502	4,961

Table 4.2 Determination of Existing Demand Rates

Year	Daily Pumping Rate (m3/day)				
	WTP*	Goulais Wells	Steelton Well	Shannon Well	Lorna Wells
2009	23,143	3,021	5,496	1,482	2,191
2010	17,656	4,953	6,155	3,187	376
2011	18,485	4,876	3,705	4,331	1,950
2012	16,826	3,890	5,831	3,394	3,167
Existing Demand (Average)	19,463	4,778	5,688	3,383	3,461

Note:
* The pumping rates represent produced rates from the WTP, not the water taking rates at the Gros Cap Intake.

A summary of the statistics for the water takings from 2000 to 2012 is also provided in Table 4.3.

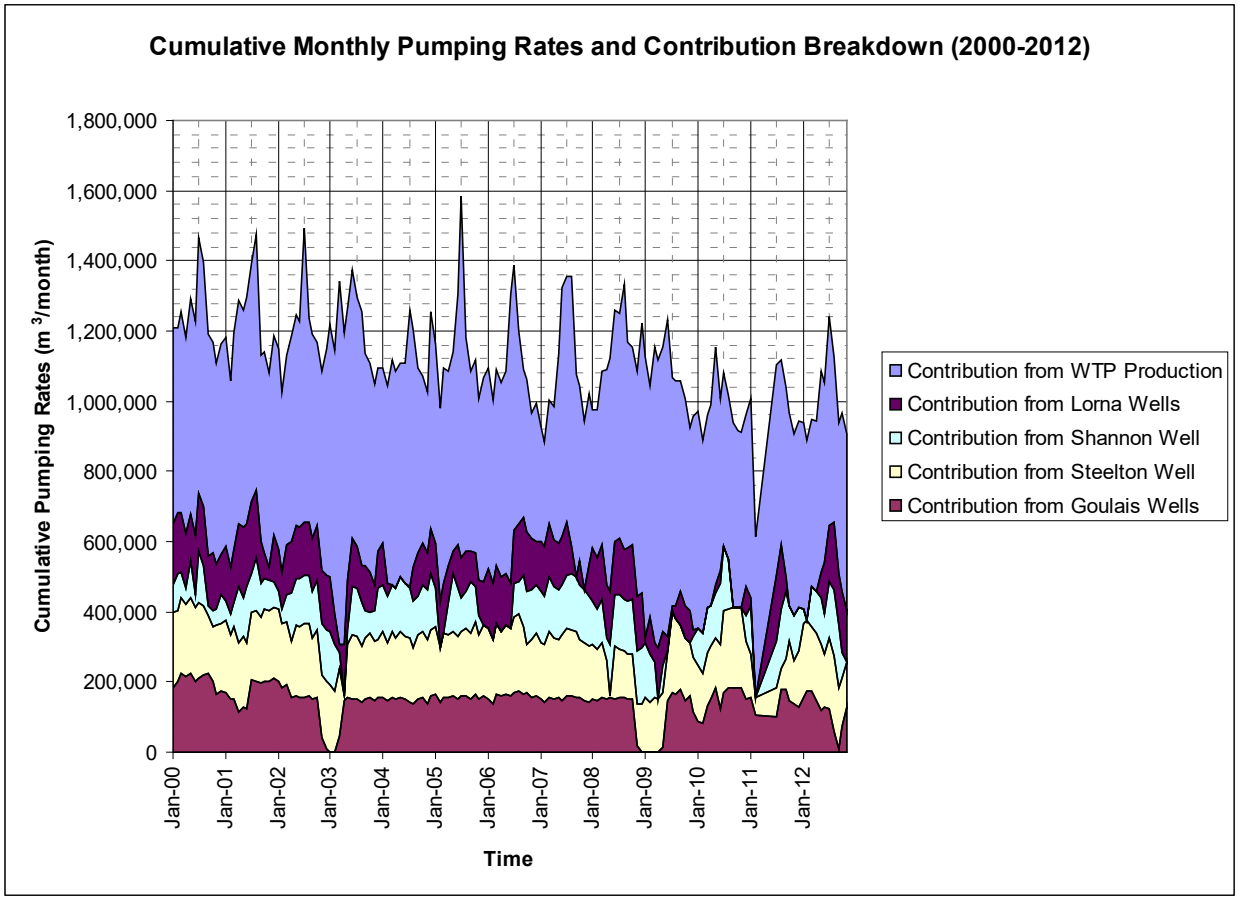
Table 4.3 Daily Pumping Rate Statistics from 2000-2012

	WTP* Production m3/day	Goulais Production m3/day	Steelton Production m3/day	Shannon Production m3/day	Lorna Production m3/day	Groundwater System m3/day	Total System m3/day
Minimum	3,481	0	0	0	0	3,173	13,729
Average	19,511	4,781	5,752	3,350	3,499	17,382	36,890
Maximum	45,571	10,482	8,029	6,883	13,477	32,534	68,819
Permitted Rates	75,000	9,988	8,200	7,000	14,000	39,188	114,188

Note:
* The pumping rates represent produced rates from the WTP, not the water taking rates at the Gros Cap Intake.

Figure 4-2 shows the total monthly water takings from 2000-2012. The data are summarized in Appendix D-3.

Figure 4-2 Cumulative Monthly Pumping Rates and Contribution Breakdown (2000-2012)



In general, the distribution between groundwater sources and the surface water is approximately 50/50; however, during certain operational and maintenance events, one source has been capable of providing sufficient water supply to the municipal system for short-term periods. Some discussions regarding increasing the percentage of groundwater takings versus surface water takings have been raised in the past; however, it is the current intention to continue to operate the water supply system with both sources providing approximately 50%.

Water Conservation Initiatives

Despite the slightly increasing number of water customers supplied by the PUC, the amount of water consumed annually over the last decade or so has declined (PUC, 2008). The PUC and the City of Sault Ste. Marie have undertaken various water conservation initiatives to curtail water consumption in the last five years. As part of these initiatives, the PUC promotes the use of water saving devices in residential homes (PUC, 2013). Additionally, a leak detection expert was

brought in to survey the entire distribution system in 2009. By identifying the potential leaks and addressing loss from the system, the efficiency of the overall system was improved, resulting in reduced overall demand (PUC, 2009).

Furthermore, a water use restriction by-law that imposes limitations on the irrigation of laws and gardens when consumption approaches 80% of in-service capacity was approved in 2011. According to the by-law, when consumption approaches 100% of in-service capacity, all outdoor water uses are restricted (PUC, 2011).

4.2.2 COMMITTED / PLANNED DEMAND

Committed demand is defined as the required increase in the quantity of water provided by a drinking water system if the area serviced by the system were developed in accordance with the official plans for the area to an extent that would result in the greatest use of the drinking water.

The PUC does not have committed or planned demands (as defined in the Technical Rules) for additional groundwater supply as a result of population growth or other new customers, recognized through the City of Sault Ste. Marie Official Plan. However, plans for future development were obtained from the Planning Division of the City of Sault Ste. Marie Engineering and Planning Department for urban expansion plans for the City. Map 2-12 illustrates the anticipated growth areas for the period of 2013-2022.

To estimate projected future water demands for the Tier 3 Water Budget, a review of anticipated future residential developments, which are to take place in the next 25 years, was conducted. As part of this review, proposed developments with the following approval status were identified and considered:

- Draft approved, with phases registered or being registered
- Draft approval pending
- Zoning approved

An estimate of the projected average water demand for the identified future residential developments was then obtained based on the number of individuals to be housed in each subdivision or apartment. The MOE Design Guidelines for Drinking-Water Systems (2008) provides a range of 270 to 450 L/capita/day for estimation of water demands. The water demand calculations for the future residential developments were based on an average per capita rate estimate of 360 L/capita/day.

Table 4.4 presents details of the water demand estimations for the future residential developments.

Table 4.4 Water Demand Estimations for the Future Residential Developments

Development Name	Development Type	Number of Units	Timing of Development / Approvals	Number of Individuals1	Estimated Average Water Demand2 (L/day)
Pine St. Apartments (West side of Pine St. Extension)***	Apartment	136	0-5 years	245	88,128
Windsor Farms*	Subdivision	64		224	80,640
Korah Road Subdivision (West of Cooper St. South of Korah Road)*		15		53	18,900
Central Creek Subdivisions (West side of Cooper St., South of Korah Rd. Subdivisions)*		24		84	30,240
Sherwood Subdivision (East of Peoples Road SE of Fairview)**		84		294	105,840
Denwood Phase VI*		47		165	59,220
Queensgate Subdivision**		94		329	118,440
Greenfield Subdivision*		62		217	78,120
Sherbrooke/Torma Subdivisions (East of Peoples Rd, NW of Fairview)*		38		133	47,880
Northern Brewery's Site		120		420	151,200
Fox Run*		600	0-25 years	2,100	756,000
Snowden Subdivison (East of Cooper St., North of Rossmore Rd.)*		35	5-10 years	123	44,100
Mallards Landing (Top of North Street, South of Third Line)		60		210	75,600

Former Hospital Sites		300		1,050	378,000
Greenfield East		344		1,204	433,440
John Dick Subdivision*		20		70	25,200
Meadowbrook Subdivision (Allens Side Road)*		140		490	176,400
Vezeau Subdivision (East of Dacey/North of Queen St. E)*		85	10-15 years	298	107,100
North side of Old Garden River Road (SW of Windsor Farms)	Future Development Area/Future Subdivisions	108		378	136,080
North side of Rossmore Road ('Garsons Farm')		354		1,239	446,040
West side of Black Road (South of Old Garden River Road)		N/A		700	252,000
West side of Old Garden River Road (North of Greenfield Subdivision)			15-20 years	700	252,000
Total (Developments with "Approved" or "Approval Pending" Status)				5,243	1,887,408

Notes:

* Draft approved, with phases registered or being registered

** Draft approval pending

*** Zoning approved

1 The number of individuals for subdivisions was calculated based on an assumption of 3.5 individuals per unit.
The number of individuals for apartments was calculated based on an assumption of 1.8 individuals per unit.
Where no details regarding the number of units were available, an estimate of 200 units was used for calculation purposes.

2 An average water consumption rate of 360 litres per capita per day was used based on MOE Guidelines for Drinking Water Systems, 2008

It was assumed that future residential developments would be serviced by the groundwater system alone as a worst case scenario. As such, future water demand for the municipal wells was estimated as the sum of the existing demand and the demand from future residential developments which are approved or whose approvals are pending. Since no additional demand from the WTP is anticipated in the future, the future demand from the surface water system was assumed to be equivalent to the existing demand.

Table 4.5 summarizes the overall future water demand estimations.

Table 4.5 Overall Future Water Demand Estimations

Source	Existing Average Daily Demand (m3/day)	Additional Average Daily Demand from Future Residential Developments (m3/day)	Overall Future Average Daily Demand (m3/day)
WTP	19,463	0	11,579
Goulais Wells	4,778	472	5,250
Steelton Well	5,688	472	6,159
Shannon Well	3,383	472	3,854
Lorna Wells	3,461	472	3,933
Total	28,888	1,887	30,775

4.3 SAFE ADDITIONAL DRAWDOWN

To determine the long term sustainability of the groundwater supply system, each well was assessed to determine the safe water level and potential average additional drawdown which could be achieved.

4.3.1 SAFE WATER LEVELS

The safe water levels are determined based on the physical and mechanical characteristic of each well. For each well, the top of the well screen, and pump intake levels were considered. Actual elevations for each are illustrated in Figure 4-3 to Figure 4-8. For all the wells, the top of the aquifer is a few metres above the top of the well intake screen. The pump intakes are also

above the top of the aquifer, with the exception of the Steelton Well. For all wells except for Steelton, the pump intakes can be lowered.

Based on this understanding, the top of the aquifer was established as the safe water level for each municipal well. It represents a realistic scenario for water taking and will preserve the efficient use of the aquifer and well function.

4.3.2 HISTORICAL WATER LEVEL

Water level records from the pumping wells have been collected from various time frames.

Figure 4-3 to Figure 4-8 show the available water level measurements at each municipal well for the whole period of record.

4.3.3 AVERAGE SAFE ADDITIONAL DRAWDOWN

The average safe additional drawdown is estimated by considering the height of water available between the average observed water elevation in the well and the established safe water level.

An analysis of the available water level measurements and corresponding pumping rates at each municipal well for the whole period of record indicates that generally, water demand rates and, correspondingly, water levels differ significantly after 1986. This corresponds to the time during which the existing WTP was brought on-line. As such, the data prior to 1986 are likely not representative of existing conditions. To maintain consistency with the time period used to establish the existing municipal demand in Section 4.2.1, water level data from the period of 2000-2012 were used in assessing the safe additional drawdown.

Table 4.6 shows the calculated safe additional drawdown.

The safe additional drawdown is calculated using observed water levels measured within the pumped wells. When considering these safe additional drawdown values in a groundwater model, it will be necessary to include the calculation of non-linear well losses and convergent head losses in the model calculation. These will be discussed further in the Local Area Risk Assessment report.

Figure 4-3 to Figure 4-8 show the average safe additional available drawdown calculated for each supply well.

Figure 4-3 Goulais Well 1 Average Safe Additional Available Drawdown

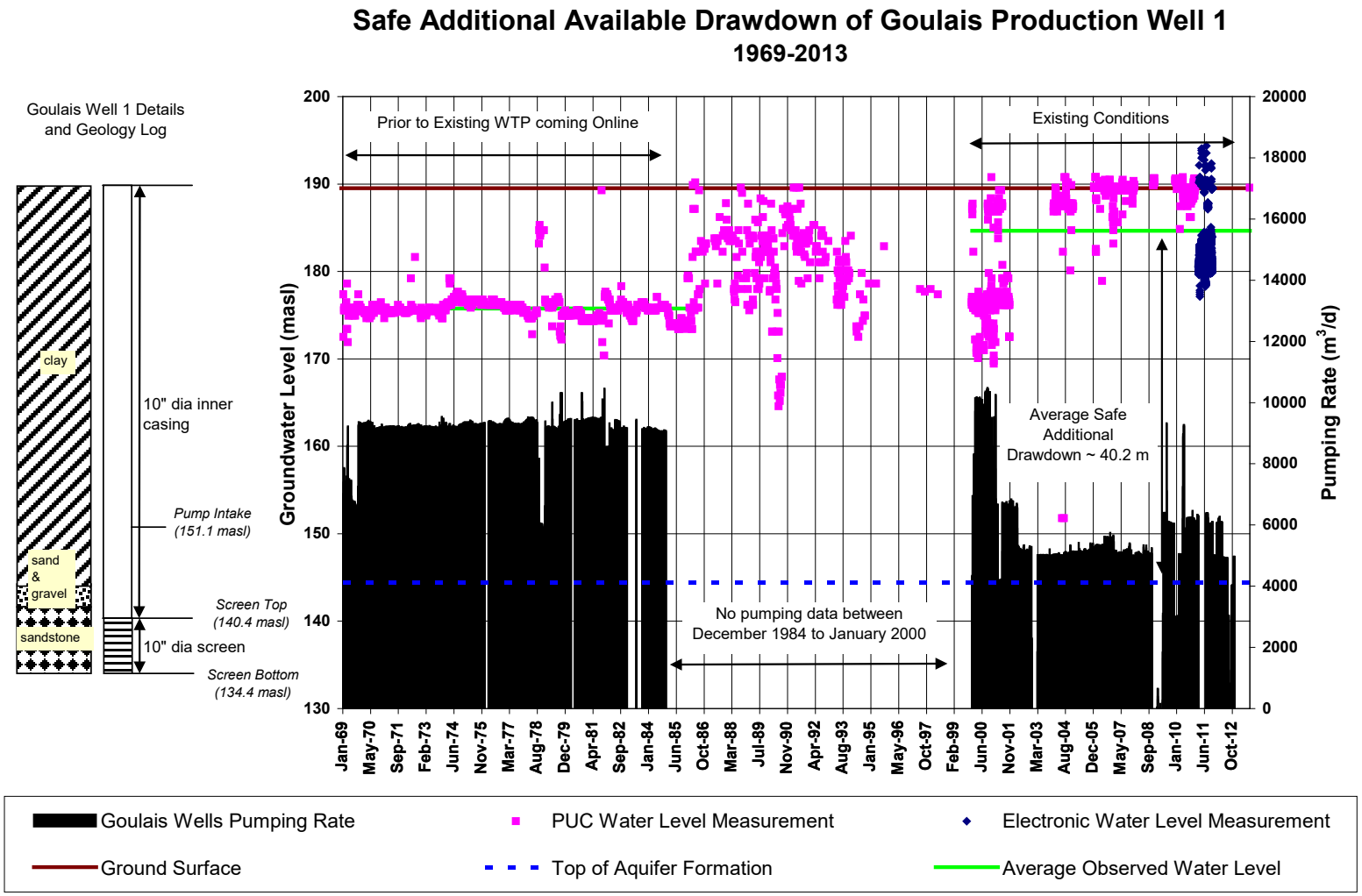


Figure 4-4 Goulais Well 2 Average Safe Additional Available Drawdown

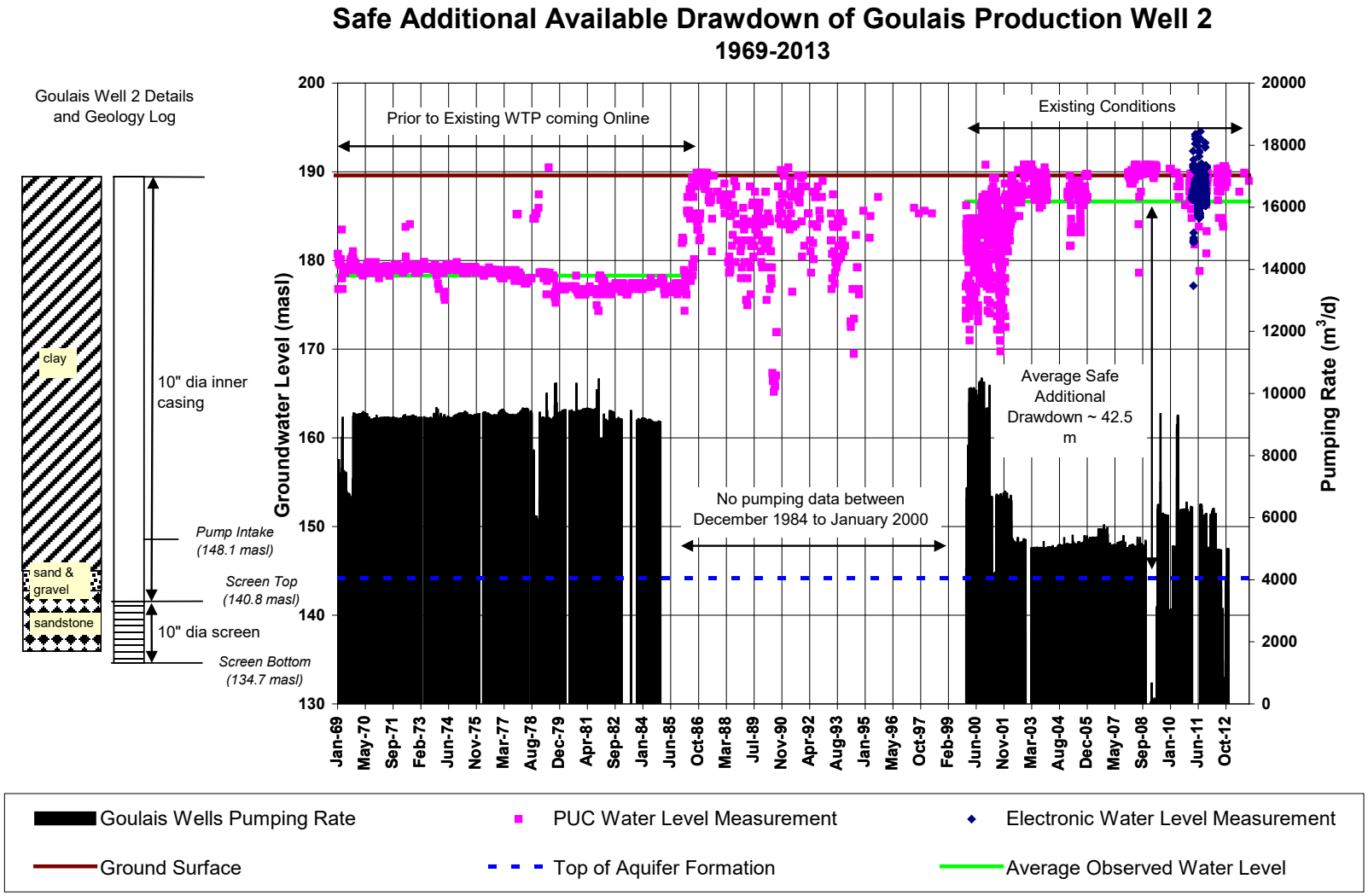


Figure 4-5 Steelton Well Average Safe Additional Available Drawdown

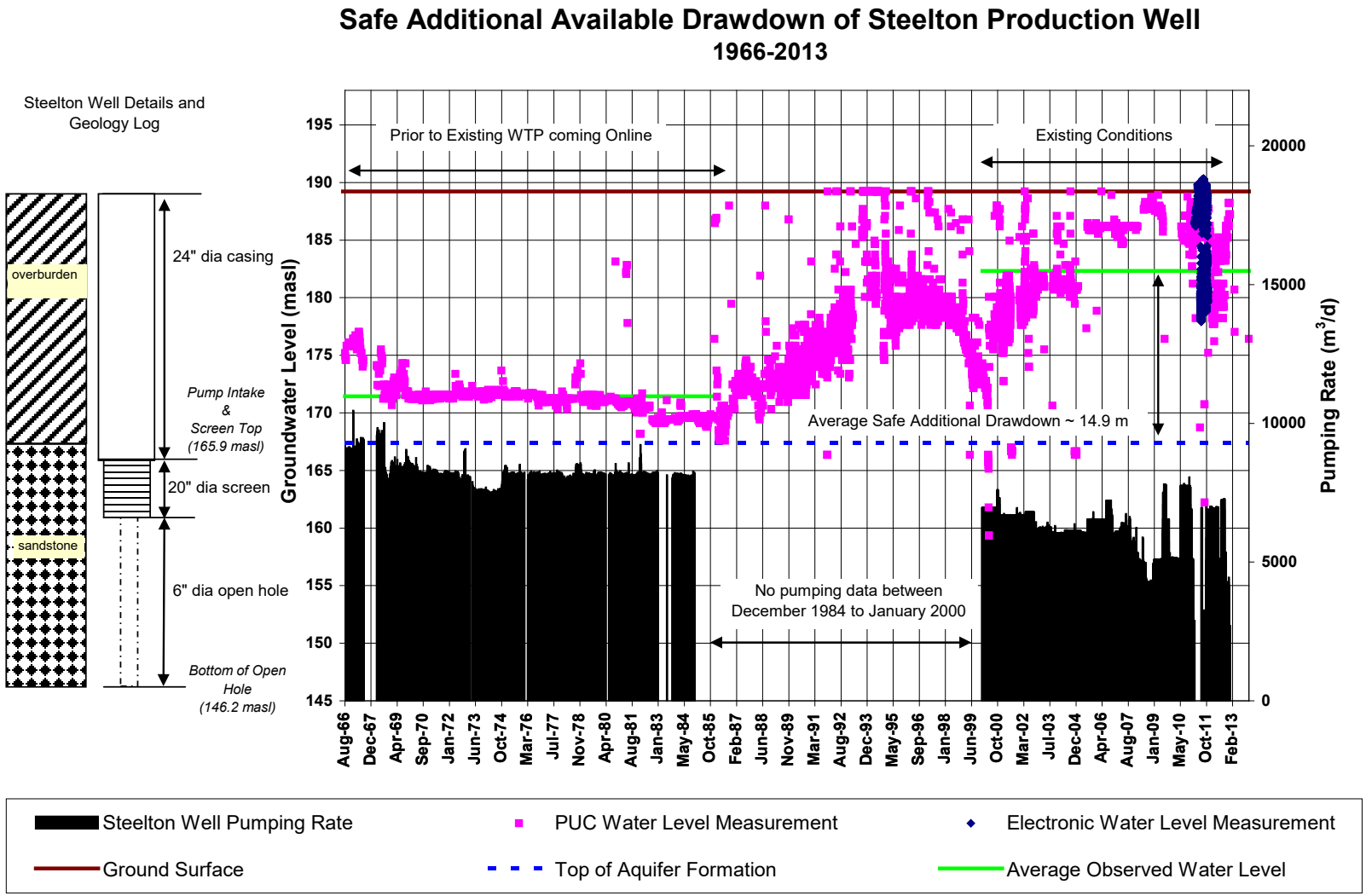


Figure 4-6 Shannon Well Average Safe Additional Available Drawdown

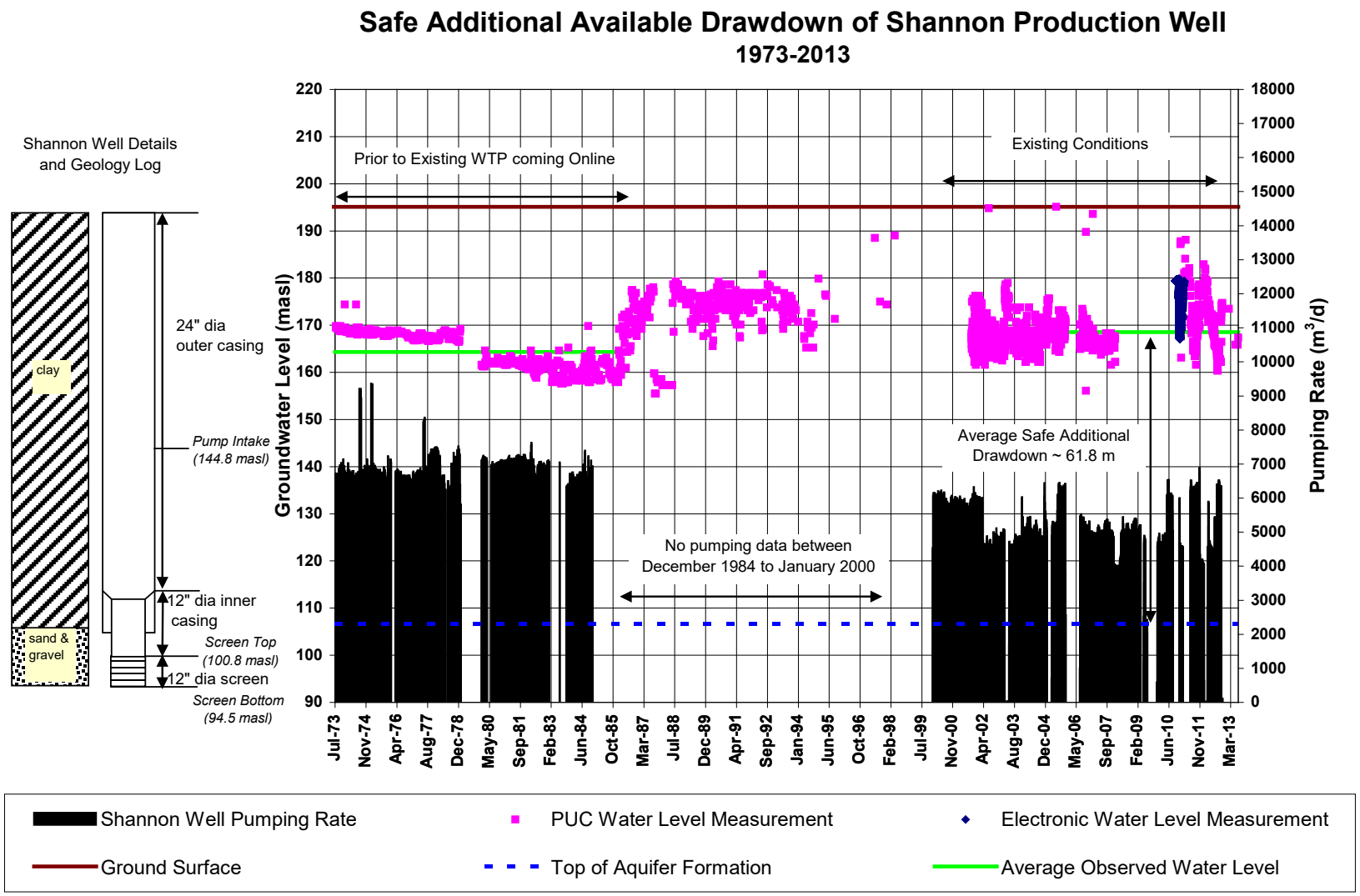
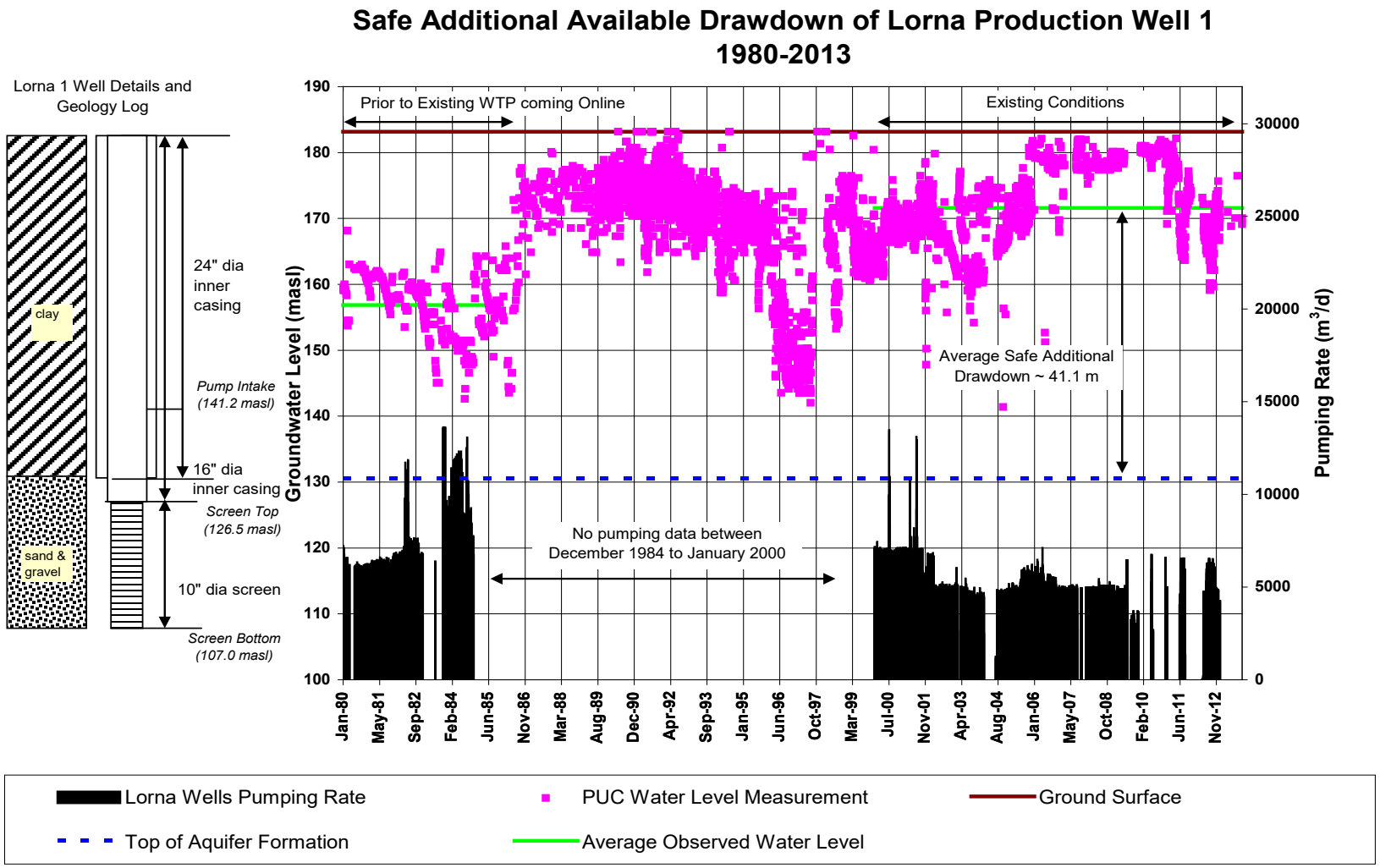


Figure 4-7 Lorna Well 1 Average Safe Additional Available Drawdown



Note: Electronic water level data are unavailable for Lorna Well 1.

Figure 4-8 Lorna Well 2 Average Safe Additional Available Drawdown

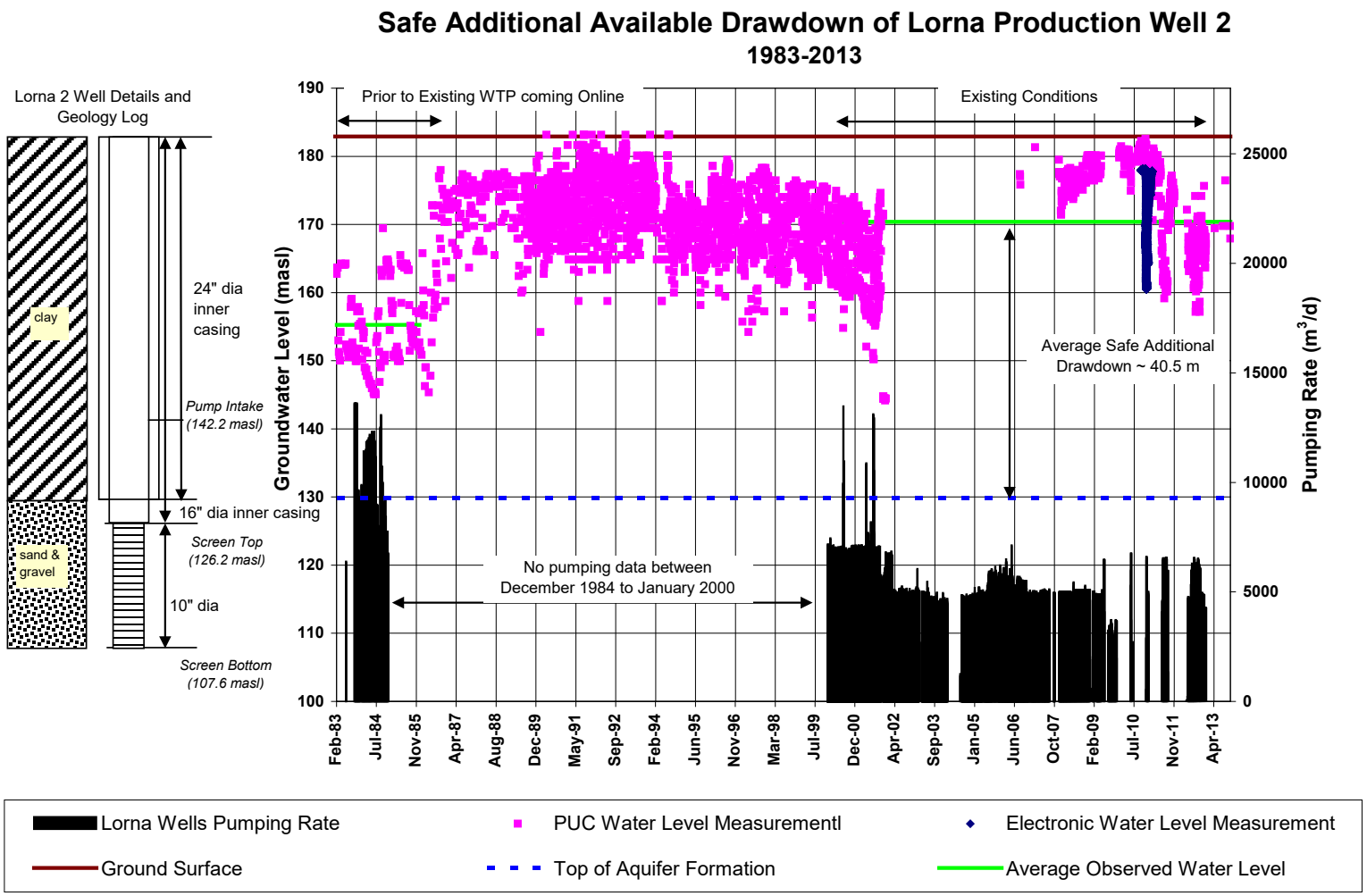


Table 4.6 Safe Additional Available Drawdown

Well Name	Ground Elevation (masl)	Top of Aquifer Elevation / Safe Water Level (masl)	Pump Intake Elevation (masl)	Average Observed Water Level (masl)	Average Safe Additional Available Drawdown (m)
Goulais Well 1	189.5	144.4	151.1	184.7	40.2
Goulais Well 2	189.6	144.2	148.1	186.7	42.5
Steelton Well	189.2	167.4	165.9	182.3	14.9
Shannon Well	195.1	106.7	144.8	168.5	61.8
Lorna Well 1	183.2	130.5	141.2	171.6	41.1
Lorna Well 2	182.9	129.9	142.2	170.4	40.5

Based on observed water level trends and historical pumping records, the Goulais Wells, Shannon Well and Lorna Wells all demonstrate a comfortable Average Safe Additional Available Drawdown value which ranges from 40 m to 62 m. The Average Safe Additional Available Drawdown at the Steelton Well is approximately 15 m.

Average pumping results in water levels which are typically above the top of the aquifer. However, there have been instances where water levels have fallen below the safe water level. Future pumping scenarios will need to ensure pumping rates do not draw down the water level past the safe water level.

4.4 NON-MUNICIPAL WATER DEMAND

Outside of the City of SSM, areas are primarily serviced by individual domestic wells. There are also a number of Permits to Take Water (PTTW) that have been issued for both public and private water takings in excess of 50,000 L per day.

4.4.1 PERMITTED WATER USERS

Results of a search of the MOE Permit to Take Water database in March 2013 identified 34 active permitted water takers in the SSMR Source Protection Area excluding the six municipal wells. However, since then, six of the permits have expired and/or are no longer active. Of the remaining

28 permitted water takers, 17 permitted water takers were identified in the Central Basin and 4 permitted water takers were identified in the East Basin. Table 4.7 provides a summary of the permitted water takers identified within the Central and East Basins. The location of these permitted water takers are presented in Map 2-13

11 non-municipal groundwater takers were identified within the two basins. The groundwater takings are primarily used for remediation purposes.

The search also identified eight surface water takers within the Central and East Basins, of which the majority withdraws water from the St. Marys River. Since the St. Marys River forms part of the Great Lakes system, these surface water takings will not be considered in the water budget analysis. The largest surface water taker is a hydroelectric power generating station with a maximum permitted rate of approximately 85,000,000 m3/day. Other identified surface water takings are for industrial, recreational, water supply, agricultural and commercial purposes. Their associated permitted rates are summarized in Table 4.7.

Finally, two permitted water takers in the East Basin withdraw water from both surface water and groundwater sources for aqua cultural purposes.

Table 4.7 Permitted Water Takers in the Central and East Basins

Permit Number	Easting (m)	Northing (m)	Source Name	Groundwater / Surface Water	Purpose for Water Taking	Permit Expiry Date	Maximum Permitted Rate (m^3/day)	Basin
6316-8K7HVL	704498	5162249	Horizontal Leachate Collector	Ground Water	Other - Remediation	30/07/2021	720	Central
6316-8K7HVL	704464	5162231	PW-2	Ground Water	Other - Remediation	30/07/2021	30	Central
6316-8K7HVL	704466	5162287	PW-3	Ground Water	Other - Remediation	30/07/2021	73	Central
6316-8K7HVL	704473	5162385	PW-4	Ground Water	Other - Remediation	30/07/2021	99	Central
6316-8K7HVL	704465	5162443	PW-5	Ground Water	Other - Remediation	30/07/2021	45	Central
6316-8K7HVL	704460	5162483	PW-6	Ground Water	Other - Remediation	30/07/2021	100	Central
6316-8K7HVL	704445	5162523	PW-7	Ground Water	Other - Remediation	30/07/2021	101	Central
6316-8K7HVL	704442	5162571	PW-8	Ground Water	Other - Remediation	30/07/2021	112	Central
6316-8K7HVL	704465	5162300	PW-9	Ground Water	Other - Remediation	30/07/2021	45	Central
6316-8K7HVL	704476	5162349	PW-10	Ground Water	Other - Remediation	30/07/2021	43	Central
0507-92RPE7	701600	5154711	St Mary's River	Surface Water	Power Production	20/12/2017	138,240	Central

Table 4.7 Permitted Water Takers in the Central and East Basins

1043-7DRRJW	707224	5153206	St. Mary's River	Surface Water	Other - Recreational	13/04/2018	1,000	Central
1402-7REGR5	701613	5154576	St. Marys River	Surface Water	Cooling Water	01/06/2015	1,136,500	Central
1653-7REHM5	701995	5155222	St. Marys River	Surface Water	Cooling Water	31/03/2017	2,766	Central
5502-6AZM3H	694752	5157632	Pond (Western Tributary to Little Carp River)	Surface Water	Nursery	01/10/2015	545	Central
7585-74CKWD	703600	5157409	Fort Creek	Surface Water	Dams and Reservoirs	19/07/2017	95,000	Central
78-P-5110	703376	5154706	St. Mary's River	Surface Water	Power Production	31/03/2028	84,948,300	Central
8126-6QJPWU	705778	5161778	Well PW1	Ground Water	Golf Course Irrigation	01/11/2016	232	East
0225-68DS83	706500	5161700	Thayer Spring	Surface and Groundwater	Aquaculture	18/01/2015	5,472	East
0225-68DS83	707600	5161300	Tarentorus Spring	Surface and Groundwater	Aquaculture	18/01/2015	15,120	East
8754-7WBLFR	709183	5152824	St. Marys River	Surface Water	Golf Course Irrigation	01/11/2019	2,271	East

4.4.2 NON-PERMITTED WATER USERS

Outside of the City of SSM, areas are primarily serviced by individual domestic wells. Approximately 717 well records were identified in the East Basin and Central Basin. The locations of wells identified in an MOE water well records search is shown in Map 2-10. For simplicity, it was assumed that these wells each have a pumping rate of 0.2 m^3/day and that the extraction is 100% consumptive.

4.4.3 CONSUMPTIVE DEMAND ESTIMATION

The consumptive demand for each groundwater use is summarized on an annual basis for each of the two basins in Table 4.8. The demand rate for permitted water takers is estimated from the maximum permitted rates.

Table 4.8 Consumptive Demand for Groundwater Uses

Basin	Water Use Category	Specific Water Use	Demand Rate (m^3/yr)	Consumptive Factor[2]	Consumptive Demand (m^3/yr)
Central	Remediation	PTTW – Landfill Remediation	499,419	100%	499,320
Total (Central Basin)					499,320
East	Commercial	PTTW-Golf Course Irrigation	84,680	70%	59,276
	Commercial	PTTW – Aquaculture[1]	3,758,040	10%	375,804
Total (East Basin)					435,080
Central and East	Individual Domestic	Sparse Rural Population (MOE WWRs	52,341	100%	52,341

Notes:

[1] A 50/50 weighting factor was used to determine the groundwater demand rate associated with the PTTW for aquaculture given that this practice takes water from both surface water and groundwater sources.

[2] Consumptive factors were assigned as per the MOE Guidance Module 7 (MOE, 2007).

4.5 OTHER WATER USES

4.5.1 AQUATIC HABITAT AND PROVINCIALLY SIGNIFICANT WETLANDS

The interaction between groundwater and surface water has not been quantified in terms of extensive baseflow studies. However, areas of recharge and discharge have been identified through groundwater elevations and topographic maps. Upwelling areas, wetlands and headwaters are known to exist south of the Precambrian uplands as a result of local-scale discharge of groundwater through the coarse permeable materials. The shallow system provides groundwater flux to the streams and is essential for preserving the natural function of the ecosystem.

The extensive rivers and creeks present in the study area are habitat for a multitude of fish species that depend on upwellings for spawning and sustained health throughout the seasons. Map 2-5 identifies the natural features in the study area. Similarly, within the planning region, wetlands are habitat for numerous flora and fauna. Map 2-5 illustrates the wetlands within the planning area comprising approximately 4 % of the study area. There are several smaller wetland areas in the northern uplands of the planning region associated with headwater areas of the rivers and creeks, which flow south towards the St. Marys River. Along the shore of the St. Marys River, larger wetland areas are found at the outlet of rivers such as the Big and Little Carp and the Root River.

As a component of this water balance, the water used by these features will be discussed qualitatively since no monitoring data are available at this stage to provide quantitative estimates. The objective of including these features in the assessment is to ensure that they are considered as a part of the system and that flows required to support the natural function of these features are not altered or affected severely as a result of an imbalance of the water budget.

4.5.2 WASTEWATER ASSIMILATION

The City of Sault Ste. Marie manages wastewater with two wastewater treatment plants. The east-end wastewater treatment plant treats about two-thirds of the City's sewage, with the remainder being treated at the west-end wastewater treatment plant. The east-end treatment plant discharges directly to the lower St. Marys River and the west-end treatment plant discharges to Leigh's Bay, in the upper St. Marys River. The withdrawals from the municipal wells are a small fraction of the flow in the St. Mary's River. Therefore, the municipal groundwater withdrawals are not expected to affect the assimilative capacity of the St. Mary's River with respect to the wastewater treatment plants.

4.5.3 RECREATION

The SSMR SPA consists of numerous recreational areas which focus on the naturally beautiful landscape, shorelines, and terrain which consist of campgrounds, trails and Provincial Parks. The SSMRCA has five conservation areas which cover 1865 hectares (4600 acres) of diverse

ecosystems including forest, wetlands and shorelines (Sault Ste. Marie Region Conservation Authority, 2013).

Fort Creek Conservation Area

The Fort Creek Conservation Area covers approximately 77 hectares (191 acres) and is situated off Second Line, east of Peoples Road. It is located near the base of Fort Creek and was developed to address flood control issues. The Fort Creek Dam was constructed between 1968 and 1971 and has a reservoir of approximately 3.24 hectares (8 acres). The conservation area consists of part of the Great Lakes-St. Lawrence forest and wetlands. In addition to its primary purpose of flood control the Fort Creek Conservation Area also provides the ancillary benefit of green space, recreational opportunities and wildlife habitat (Conservation Ontario, 2013).

Gros Cap Conservation Area

The Gros Cap Conservation Area is located along the western limits of the SSMSPA near Lake Superior and exhibits a wide variety of vegetation types including mixed forest, cedar swamp, hardwood stand and significantly rare wildflowers. This diversity in habitat has created plentiful wildlife. Cobbles beaches, rock faces and bluffs highlight the rugged, natural characteristics of Gros Cap. Steep cliffs rise from the water to a height of 60 to 90 metres (200 to 300 feet) overlooking Lake Superior (Conservation Ontario, 2013).

Hiawatha Highlands Conservation Area

The Hiawatha Highlands Conservation Area is located between Fifth and Sixth Line, east of Great Northern Road (Hwy.17 N). The Highlands offer magnificent scenery including breathtaking waterfalls and 35 kilometres (22 miles) of nature trails. Hundreds of hectares of forests with creeks, lakes and wetlands create the ideal habitat for more than 70 species of birds and 18 species of mammals (Conservation Ontario, 2013).

Mark’s Bay Conservation Area

The Mark’s Bay property is a 103 hectare (254 acre) site including 3000 metres (10,000 feet) of shoreline along the St. Marys River. The forest is part of the Great Lakes-St. Lawrence forest region and is home to many different birds and small mammals (Conservation Ontario, 2013).

Shore Ridges Conservation Area

The Shore Ridges Conservation Area comprises 443 hectares (1100 acres) of land including 366 metres (1200 feet) of shoreline. Found within the city limits at the junction of Sunnyside Beach Road and Shatruck Drive, the conservation area consists of hardwood forests, beach terraces and wetland areas. The wetland is a Provincially Significant Wetland and 25% of the wetland is a fen, 74% is swamp and 1% is marsh. Freshwater springs are one of the natural features found in the Shore Ridges Conservation Area (Conservation Ontario, 2013).

Walls Lake Forest

Walls Lake Forest encompasses an area of approximately 171 hectares (423 acres). The general flora of the area is primarily comprised of hardwood forests. Walls Lake Forest includes two significant wetlands located on its south and northeast side and the area has been identified as an important waterfowl staging area (Regen Forestry, 2007).

Headwater Forest

Headwater Forest encompasses an area of approximately 131 hectares (324 acres). The area is characterized primarily by a mixed maple-oak hardwood forest. As its name suggests, the area serves as an important contributor to the Davignon Creek. There are three major wetland areas transecting the boundaries of Headwater Forest, primarily in a northwest to southeast orientation (Regen Forestry, 2007).

Burke Forest

Burke Forest encompasses an area of approximately 121 hectares (299 acres) which is characterized primarily by a mixed maple-oak hardwood forest. Several wetland areas traverse the Burke Forest area including two major headwaters of Canon Creek and Davignon Creek across the northeast side and in the southwest corner. The largest wetland traversing Burke Forest comprises open water measuring 100 m in width and nearly 2 km in length (Regen Forestry, 2007).

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Zoning By-law 2005-150 (October, 2005) – Zoning Map

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BACKGROUND

The Sault Ste. Marie Region Source Protection Area (SSMR SPA) is situated within the District of Algoma, along the north shore of the St. Marys River and eastern shore of Lake Superior. The groundwater system in the SSMR SPA is divided into West, Central and East Basins. The SSMR SPA has a variety of groundwater users; their takings are for domestic, commercial and industrial purposes. However, the most significant use of groundwater in the Central and East Basins is for municipal drinking water supply. The analyses conducted for the Tier One/Tier Two Water Budget study indicate that water takings from the West Basin represent about 2% of the available water supply; following the guidance documents, this is interpreted as a low stress on the groundwater system. The water takings from the Central and East Basins are predicted to represent about 25% of the available supply. The Central and East Basins are interpreted to have the potential for moderate stress.

Considering the assigned stress levels, a Tier Three Water Budget and Local Area Risk Assessment is mandated to assess the likelihood that the municipality will be able to sustain its allocated (existing plus committed) water demand and to identify threats to the drinking water supply that may influence the municipality's ability to meet its allocated pumping rates. This report satisfies the requirements for the Local Area Risk Assessment through the delineation of Vulnerable Areas; the evaluation of various risk scenarios; the determination of Local Area Risk Levels for the SSMR SPA; and the identification of drinking water quantity threats.

VULNERABLE AREAS

This study considered the WHPA-Q1, WHPA-Q2 and Local Area in the delineation of vulnerable areas. The WHPA-Q1 was delineated from the drawdown generated between conditions of (1) existing land use and no pumping (municipal or non-municipal) and that of (2) existing land use and allocated pumping rates along with consumptive rates for all other water users in the study area. The 1-m drawdown contour was established as the limits of the WHPA-Q1 based on observed water level fluctuations in monitoring wells.

The WHPA-Q2 encompasses the WHPA-Q1 and any area where a future reduction in recharge due to future land development may have a measurable impact on the water levels in municipal supply wells. An assessment of the future development plans for the City of Sault Ste. Marie (SSM) was conducted to quantify the impact of the proposed changes in land use on the groundwater recharge rates. The assessment showed a 1% decrease in overall recharge for the

study area. Given the insignificant reduction in recharge, the areas of WHPA-Q1 and WHPA-Q2 were assumed to be identical.

Since the Local Area is, by definition, synonymous with the WHPA-Q2, the Local Area defined for the SSMR SPA was found to be equivalent to the WHPA-Q1. The Local Area includes the cumulative drawdown for the Steelton Well and the Goulais Wells and that for the Shannon Well and the Lorna Wells.

RISK ASSESSMENT SCENARIOS AND MODEL-PREDICTED SCENARIO RESULTS

A set of eight (8) risk assessment scenarios (Scenarios C, G(1), G(2), G(3), D, H(1), H(2) and H(3)) was developed to consider the impact of increases in water demand, drought conditions, and land use change on the sustainability of the municipal water supply. The scenarios were simulated using the Tier Three integrated groundwater and surface water models. Scenario C represents existing municipal pumping, existing land use and average climate conditions. Scenario G(1) assesses the ability of the municipal wells to meet their allocated demand (existing plus committed) under conditions of future land use and average climate, Scenario G(2) assesses the ability of the municipal wells to meet their allocated demand (existing plus committed) under conditions of existing land use and average climate and Scenario G(3) assesses the ability of the municipal wells to meet their existing demand under conditions of future land use and average climate. Scenario D aims to evaluate the ability of the municipal wells to pump at their respective existing rates during a drought period. Scenario H(1) assesses the ability of the municipal wells to meet their allocated demand under future land use and drought conditions, Scenario H(2) assesses the ability of the municipal wells to meet their allocated demand under existing land use and drought conditions and Scenario H(3) assesses the ability of the municipal wells to meet their existing demand under future land use and drought conditions.

The additional drawdowns at each municipal well (relative to the drawdowns generated in Scenario C) for each of the risk assessment model scenarios were simulated and compared to the estimated average safe additional available drawdown, which was established for each municipal well as part of the Water Demand Analysis of the Revised Conceptual Understanding Report (Kresin/CEG, 2013). The model-simulated additional drawdowns were found to be less than the established average safe additional available drawdown at all of the municipal wells for all risk assessment scenarios.

WATER QUANTITY THREATS

A drinking water quantity threat is any activity that reduces groundwater recharge to an aquifer or any consumptive water taking.

Considering the low rates of non-municipal permitted groundwater takings, and relatively low demand from domestic well users, no water quantity threats were identified in the context of consumptive water demands in any of the identified vulnerable areas. Additionally, since land use

changes projected for the SSMR SPA are limited and the proposed developments are not anticipated to significantly affect the groundwater recharge areas, there is low potential for a water quantity threat in terms of reduction in recharge.

SIGNIFICANT GROUNDWATER RECHARGE AREAS

As part of this study, Significant Groundwater Recharge Areas (SGRAs) were delineated by identifying the portion of the study area where groundwater recharges at a rate of 1.15 times greater than the average annual groundwater recharge for the area.

The analysis was conducted based on hydrologic response units (HRUs) which were delineated during the surface water modelling and used to assign recharge rates in the refined groundwater model for the Tier Three assessment. These HRUs represent areas with similar infiltration characteristics delineated on the basis of factors such as land use, land cover, soil type and areas with high potential groundwater recharge.

The major SGRAs for the SSMR SPA were found to be located at the bedrock/overburden contact along the southern border of the Precambrian uplands to the north of the City, with an area of approximately 3750 hectares. This portion of the study area is associated with the gravel-rich glaciolacustrine beaches deposited adjacent to the uplands.

The assessment report was originally developed under the 2008, 2009 and 2013 versions of the Technical Rules and where updates were made, they were carried out under amendments to the 2017 Rules and 2018 addition of pipelines circumstances to the Table of Drinking Water Threats.

6.0 LOCAL AREA RISK ASSESSMENT INTRODUCTION

6.1 OBJECTIVES

The Sault Ste. Marie Region Source Protection Area (SSMR SPA) is situated within the District of Algoma, along the north shore of the St. Marys River and eastern shore of Lake Superior. The groundwater system in the SSMR SPA is divided into West, Central and East Basins (Map 1-1). The SSMR SPA has a variety of groundwater users; their takings are for domestic, commercial and industrial purposes. However, the most significant use of groundwater in the Central and East Basins is for municipal drinking water supply.

A Tier Three Water Budget and Local Area Risk Assessment (Tier Three Assessment) is completed to estimate the likelihood that a municipality's drinking water wells or surface water intakes will be able to sustain their allocated pumping rates while considering increased municipal water demand, future land development, drought conditions, and other water uses.

According to the Part III.2 of the Technical Rules: Assessment Report, *Clean Water Act 2006* (MOE, November 16, 2009), a Tier Three Assessment must be completed for all Type I, II, and III drinking water systems where:

1. there have been historical issues with water sources meeting demand; or
2. the Tier Two subwatershed stress level is Moderate or Significant.

The analyses conducted for the Tier One/Tier Two Water Budget study (Kresin Engineering and GENIVAR, 2008) indicate that current water takings from the West Basin represent about 2% of the available water supply; the West Basin is considered to have potential for low stress on the groundwater system. The current water takings from the Central and East Basins are predicted to represent about 25% of the available supply, and the basins are considered to have the potential for moderate stress. A Tier Three Water Budget and Local Area Risk Assessment is therefore required for the Central Basin and East Basin.

6.2 METHODOLOGY

The approach used in this study meets the requirements of the Tier Three Water Budget and Local Area Risk Assessment and adheres to the Water Budget & Water Quantity Risk Assessment Guide prepared by the Ontario Ministry of Natural Resources (MNR) and Ontario Ministry of the Environment (MOE), dated 2011 and is described as follows.

1. Develop the Tier Three water budget models.
2. Characterize municipal wells and intakes.
3. Estimate allocated quantity of water.
4. Identify and characterize drinking water quantity threats.
5. Characterize projected land use.

6. Characterize other water uses.
7. Delineate vulnerable areas.
8. Evaluate risk scenarios.
9. Assign risk level.
10. Identify drinking water quantity threats and areas where they are significant and moderate.

This report satisfies items 4 and 7 to 10 of the Local Area Risk Assessment through the delineation of Vulnerable Areas; the evaluation of various risk scenarios; the determination of Local Area Risk Levels for the SSMR SPA; and the identification of drinking water quantity threats. The development of the Tier Three water budget models (item 1) is discussed in the Conceptual and Numerical Model Development Report which is appended at the back of this report (Appendix A). The remaining items (items 2, 3, 5 and 6) are satisfied in the Revised Conceptual Understanding Report (Kresin/CEG, 2013) which was submitted under a separate cover.

6.2.1 INTEGRATED GROUNDWATER AND SURFACE WATER MODELS

Surface water and groundwater models have been developed (Appendix A) in accordance with the requirements of the Technical Rules: Assessment Report, *Clean Water Act, 2006* (MOE, November 16, 2009). The models have been developed to refine the understanding of the Water Budget within the SSMR SPA and are based on the conceptual understanding of the groundwater and surface water systems (Revised Conceptual Understanding Report, Kresin/CEG, 2013).

The surface water model was constructed for the Root River Subwatershed using the Guelph All-Weather Storm-Event Runoff (GAWSER) model. The model was designed to simulate water budget components in a spatially detailed and temporally dynamic manner using hourly time steps. It was calibrated using flow data from the Root River gauge station located near the downstream limit of the Root River Subwatershed for the period 1971 to 2010.

The groundwater model is based on a MODFLOW model developed initially by Waterloo Numerical Modelling Corporation (WNMC) as part of the 2003 Burnside Groundwater Study (R.J. Burnside & Associates Limited, 2003). The model area includes the lowland portion of the Central Basin and East Basin. The model was refined around the municipal wells to improve the local area understanding.

Groundwater recharge from the surface water model was used as input into the groundwater flow model. Details of the models are presented in the Conceptual and Numerical Development Report included in Appendix A.

The Tier Three Water Budget integrates the numerical groundwater and surface water models to delineate the “Local Area” for the groundwater wells which form the basis for the Local Area Risk Assessment. In this assessment, the water budget models are used to estimate the impact to the wells in response to a series of water demand, climate, and land use scenarios. Where these scenarios identify a potential that wells or intakes will not be able to sustain their allocated rates, the Local Area is assigned a ‘Moderate’ or ‘Significant’ Water Quantity Risk Level.

6.2.2 LOCAL AREA

The first step involves the Delineation of Vulnerable Areas (WHPA-Q1 and WHPA-Q2) and defining the "Local Area" using the Tier Three Water Budget Model. The WHPA-Q1 is delineated by computing the drawdown cone for the municipal wells with allocated pumping rates. For the delineation of the WHPA-Q2, additional areas are identified over those in WHPA-Q1, where recharge reductions result in a measurable impact to water levels at municipal wells. As per the Technical Rules, the “allocated quantity of water” refers to the combined amount of existing and committed pumping rates for the municipal wells up to the current lawful Permit To Take Water Takings. As discussed in the Revised Conceptual Understanding Report (Kresin/CEG, 2013), the Sault Ste. Marie Public Utilities Commission (PUC) does not have committed or planned demands (as defined in the Technical Rules) for additional groundwater supply as a result of population growth or other new customers, recognized through the City of Sault Ste. Marie (SSM) Official Plan. However, plans for future development were obtained from the Planning Division of the City of Sault Ste. Marie Engineering and Planning Department for urban expansion plans for the City. For the purposes of this study, the term “committed”, in relation to pumping rates, water demand or water supply, is used to describe the estimated increase in pumping rates required to meet projected water demands based on the urban expansion plan details provided by the Planning Department of the City.

The term “Local Area” is introduced in the Technical Rules (Part III.2) to focus the water budget assessment around drinking water wells or intakes. Local Areas for surface water or groundwater systems are considered vulnerable areas. Surface water intake(s) are not considered in the assessment of the SSMR SPA as the identifiable intakes draw from the Great Lakes system.

With respect to groundwater wells, the Local Area is the combination of the following areas:

(i) the cone of influence of the well;
(ii) the cones of influence resulting from other water takings where those cones of influence intersect that of the well; and
(iii) the areas where a reduction in recharge would have a measurable impact on the cone of influence of the well.

For one or more wells that draw water from an aquifer, the cone of influence is the projection to the ground surface of the cone of depression created in the water table or potentiometric surface when the wells are pumped at rates equivalent to their allocated rates.

6.2.3 RISK SCENARIOS

A set of scenarios are considered with varying allocated quantities of water for each well, average climate and drought conditions, and future land use. Each scenario is evaluated in terms of its ability to provide water at each well along with the impact to other water uses.

6.2.4 RISK LEVEL

A Risk Level ranking (Low, Moderate, Significant) is assigned to each Local Area based on the results of the risk scenarios. The determination of the Risk Level takes into consideration the

exposure level for the Local Area and the tolerance of the municipal system to compensate for any constraints or limitations.

Municipalities typically implement physical solutions (e.g., storage reservoirs, peaking / backup intakes) and water conservation measures to reduce the amount of instantaneous water demand required from a primary drinking water source or to reduce the community's overall water demand. These types of measures are implemented to increase a municipality's 'tolerance' to short-term water shortages. Tolerance effectively reduces the potential that a municipality will face short or long-term water quantity shortages.

An uncertainty level (e.g., high, low) is considered for each Risk Level ranking.

6.2.5 WATER QUANTITY THREATS

Consumptive water uses and reductions in groundwater recharge within the Local Area are identified as Moderate or Significant drinking water threats. The risk scenarios consider the need to meet the water demand requirements of other surrounding uses, particularly those that are required to be maintained by provincial or federal law such as wastewater assimilation flows or the ecological flow requirements of a coldwater fish habitat.

7.0 DELINEATION OF VULNERABLE AREAS

7.1 WHPA-Q1

WHPA-Q1 areas are delineated by determining the change in simulated heads (drawdowns) within the production aquifers between the following two model scenarios:

1. Steady-state model using existing land use and no municipal or non-municipal pumping, to determine the groundwater levels that would exist without pumping; and
2. Steady-state model using existing land use and allocated municipal pumping rates (existing plus committed) along with consumptive use rates for all other water uses in the study area (Risk Assessment Scenario G(2))

The drawdowns predicted for the Scenario G(2) simulation are used to delineate the WHPA-Q1 with the 1-m drawdown contour as the limit. The 1-m drawdown contour was established based on an examination of hydrographs of the CEG-Kresin observation wells in the study area (Appendix B). Long-term water level records for dedicated observation wells that are beyond the immediate influence of municipal pumping are scarce in the study area. However, the data that are available suggest that groundwater levels exhibit natural fluctuations and that these fluctuations may be on the order of ± 1 m. Therefore, it is not possible to infer whether water level changes of less than 1 m are caused by pumping or by natural variations.

The resulting extent of the WHPA-Q1 area is shown on Map 2-1. Physical observations suggest that the aquifers exhibit characteristics of a heterogeneous and confined aquifer system, thereby propagating the extent of drawdown to significant distances from the municipal supply wells.

7.2 WHPA-Q2

The WHPA-Q2 area is defined in the Technical Rules as the WHPA-Q1 area plus any area where a future reduction in recharge due to future land development may have a measurable impact on the water levels in municipal supply wells. A review of the Land Use and Zoning mapping from Zoning-By-Law 2005-150 issued in October 2005 suggests that the land use in the SSMR SPA is not likely to undergo significant change. Certain areas within the core of the City are to be revitalized through infilling, with some developments in the northern portion of the City extents. Future development plans for the City of Sault Ste. Marie were also obtained from the Planning Division of the City of Sault Ste. Marie Engineering and Planning Department. These plans reflect anticipated urban expansion areas for the period of 2013-2022. Map 2-2 illustrates the anticipated growth areas. An analysis was conducted to quantify the impact of the proposed changes in land use on the overall recharge within the study area. Map 2-3 shows the extent of the anticipated changes in recharge due to future land development based on average climate conditions. As part of the analysis, area-weighted annual recharge rates were calculated for each hydrologic response unit (HRU) delineated as part of the conceptual and numerical model development (Appendix A) for the original and proposed recharge distribution. All proposed areas of

development were assigned a revised annual recharge rate equivalent to that for developed areas (125 mm/year) under average climate conditions. Results of the calculations showed only an overall 1% decrease due to proposed future land development. As such, there is no significant reduction in recharge for the study area and, therefore, the areas of WHPA-Q1 and WHPA-Q2 are expected to be identical. Table 7.1 provides a summary of the area-weighted annual recharge rate calculations.

Table 7.1 Area-Weighted Annual Recharge Rate Analysis for Proposed Land Use Change

HRU Description	HRU Recharge (mm/year)	Area (ha)		Area-Weighted Recharge Rate (mm/year)	
		Original	New	Original	New
Waterbody/Impervious Area	0	36.3	25.7	0.0	0.0
Wetland	40	41.3	41.3	0.1	0.1
Fine Grain Soil/Granite Bedrock	65	2,150.7	1,996.1	10.6	9.8
Developed Area	125	5,191.2	5,537.7	49.1	52.4
Medium/Coarse Grain Soil – not on HPGRA	350	2,898.7	2,760.8	76.8	73.1
Medium/Coarse Grain Soil – on HPGRA	425	1,810.5	1,776.6	58.2	57.1
HPGRA Receiving Overland Flow	2100	1,088.3	1,079.0	172.9	171.4
Total				367.7	364.0
Percent Decrease in Recharge				1.0 %	

Note:

HPGRA = High Potential Groundwater Recharge Area (R.J. Burnside & Associates Ltd., June 2003)

7.3 LOCAL AREA

The Local Area is, by definition, synonymous with the WHPA-Q2. In this study, the WHPA-Q2 is identical to the WHPA-Q1. Therefore, the Local Area is equivalent to the WHPA-Q1 area. There are two distinct areas of drawdown caused by municipal pumping: in the Central Basin associated with pumping from the Goulais and Steelton Wells, and in the East Basin associated with the Shannon and Lorna Wells. As shown on Map 2-4, the cumulative effects of pumping in both basins causes the 1-m drawdown contour to encompass a large area that incorporates all of the municipal wells. The Local Area delineated based on the 1-m drawdown contour is shown on Map 2-4.

8.0 DEVELOPMENT OF RISK ASSESSMENT SCENARIOS

A set of risk assessment scenarios, listed in Table 8.1, has been developed to consider the impact of increases in water demand, drought conditions, and land use change on the sustainability of the municipal water supply. The scenarios are evaluated using the Tier Three integrated groundwater and surface water models.

Table 8.1 Risk Assessment Scenarios

Scenario	Time Period	Model Scenario Details		
		Land Use	Municipal Pumping	Model Simulation
C	Period for which climate and stream flow data are available	Existing	Existing Demand	Steady-state, Average Annual Recharge and municipal pumping
G(1)	Period for which climate and stream flow data are available	Future	Existing + Committed Demand	Steady-state, Average Annual Recharge and Municipal Pumping
G(2)	Period for which climate and stream flow data are available	Existing	Existing + Committed Demand	Steady-state, Average Annual Recharge and Municipal Pumping
G(3)	Period for which climate and stream flow data are available	Future	Existing Demand	Steady-state, Average Annual Recharge and Municipal Pumping
D	10-year drought period	Existing	Existing Demand	Transient, Average Monthly Recharge and Municipal Pumping from each Wellfield
H(1)	10-year drought period	Future	Existing + Committed Demand	Transient, Average Monthly Recharge and Municipal Pumping from each Wellfield

Table 8.1 Risk Assessment Scenarios

Scenario	Time Period	Model Scenario Details		
		Land Use	Municipal Pumping	Model Simulation
H(2)	10-year drought period	Existing	Existing + Committed Demand	Transient, Average Monthly Recharge and Municipal Pumping from each Wellfield
H(3)	10-year drought period	Future	Existing Demand	Transient, Average Monthly Recharge and Municipal Pumping from each Wellfield

Table 8.2 summarizes the input values to the groundwater model with respect to municipal pumping for the steady-state scenarios (Scenarios C; G(1); G(2) and G(3)). The HRU-based distribution of average annual recharge rates used for the two scenarios is shown on Map 3-5 - Recharge Input to the Groundwater Model of Appendix A.

Table 8.2 Groundwater Model Input Summary of Municipal Pumping for Steady-State Scenarios

Scenarios	Yearly Average Pumping Rate (m^3/day)			
	Goulais	Steelton	Shannon	Lorna
C; G(3) (Existing Pumping Demand)	4778	5688	3383	3461
G(1); G(2) (Existing + Committed Pumping Demand)	5250	61560	3854	3933

Table 8.3 summarizes the input values to the groundwater model with respect to municipal pumping for the transient scenarios (Scenarios D; H(1); H(2) and H(3)). The HRU-based distribution of average monthly recharge rates used for the two scenarios is presented in Appendix C.

Table 8.3 Groundwater Model Input Summary of Municipal Pumping for Transient Scenarios

Existing Pumping Demand – Scenarios D; H(3)					Existing + Committed Pumping Demand – Scenarios H(1); H(2)				
Month	Monthly Average Pumping Rate (m³/day)				Month	Monthly Average Pumping Rate (m³/day)			
	Goulais	Steelton	Shannon	Lorna		Goulais	Steelton	Shannon	Lorna
January	4279	5884	3184	3361	January	4319	5924	3223	3401
February	4552	5776	2651	3361	February	4592	5815	2690	3401
March	4587	5913	2742	3121	March	4626	5953	2781	3160
April	4703	5281	2826	3670	April	4743	5320	2865	3709
May	4763	5374	3604	3526	May	4802	5413	3643	3565
June	4983	5942	4135	3915	June	5022	5981	4174	3955
July	5164	6051	4135	3927	July	5203	6090	4174	3966
August	5202	5907	4114	4023	August	5241	5946	4153	4062
September	5261	5803	3651	3481	September	5300	5842	3691	3520
October	5098	5651	2955	3418	October	5137	5691	2994	3458
November	4549.	5630	2910	2880	November	4589	5670	2949	2920
December	4181	5486	3519	3363	December	4221	5526	3558	3403

Sustainability of the municipal water supply is assessed in terms of the simulated change in water level in the wells relative to the average safe additional available drawdown. As discussed in the Revised Conceptual Understanding Report (Kresin/CEG, 2013), the average safe additional available drawdown was determined as the difference between the average observed water level

over the period of 2000-2012 (PUC measurements) and the safe water level (established as the top of the aquifer) for each municipal well. The 2000-2012 period was assumed to be representative of existing conditions. Table 4.6 below summarizes the calculated safe additional available drawdown at each of the six municipal wells. Figure 8-1 to Figure 8-6 present graphical illustrations of the estimated average safe additional available drawdown for each individual municipal well.

Table 8.4 Summary of Average Safe Additional Available Drawdown Evaluation

Well Name	Ground Elevation (masl)	Top of Aquifer Elevation / Safe Water Level (masl)	Pump Intake Elevation (masl)	Average Observed Water Level (masl)	Average Safe Additional Available Drawdown (m)
Goulais Well 1	189.5	144.4	151.1	184.7	40.2
Goulais Well 2	189.6	144.2	148.1	186.7	42.5
Steelton Well	189.2	167.4	165.9	182.3	14.9
Shannon Well	195.1	106.7	144.8	168.5	61.8
Lorna Well 1	183.2	130.5	141.2	171.6	41.1
Lorna Well 2	182.9	129.9	142.2	170.4	40.5

Figure 8-1 Goulais Well 1 Average Safe Additional Available Drawdown

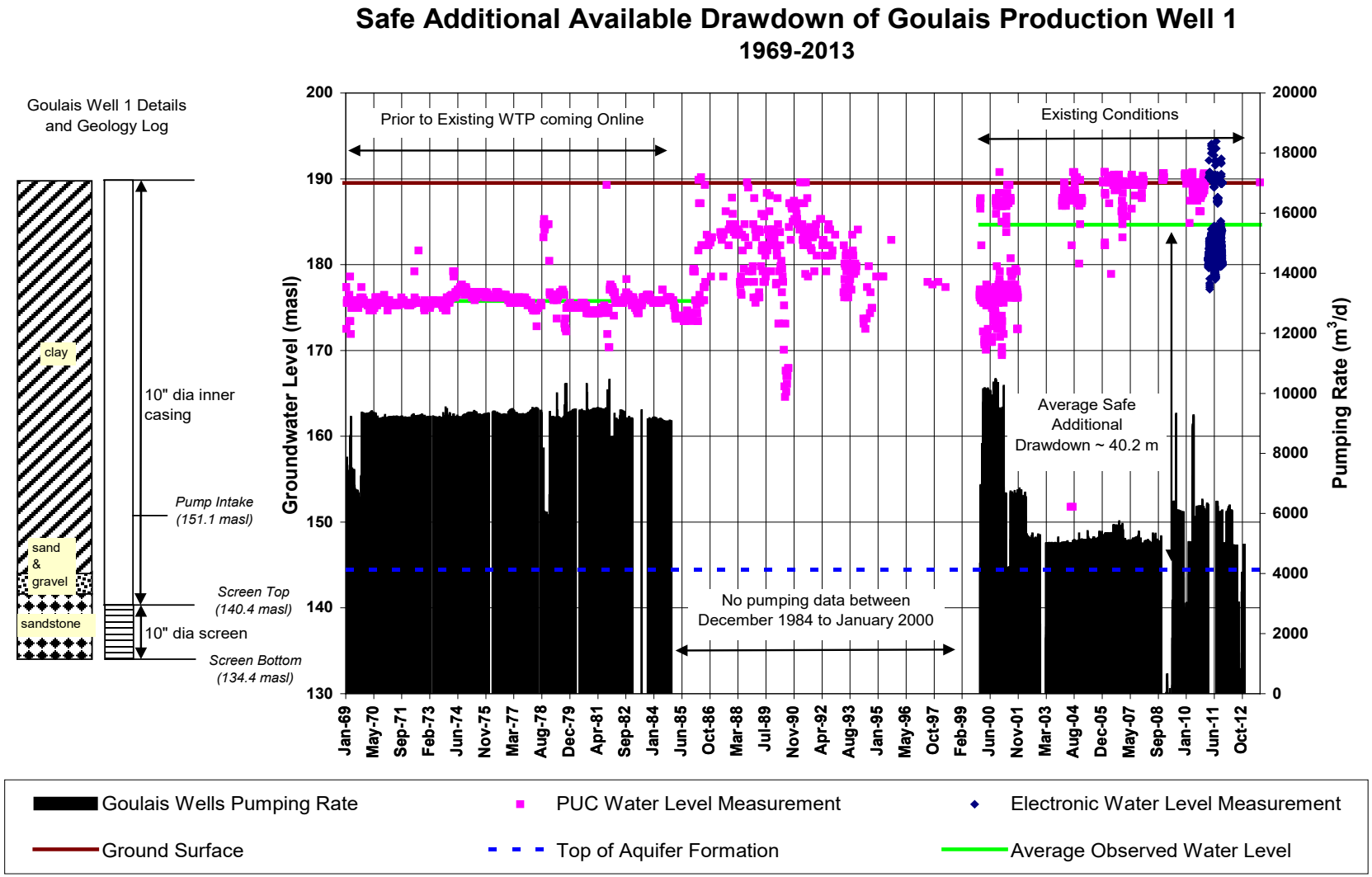


Figure 8-2 Goulais Well 2 Average Safe Additional Available Drawdown

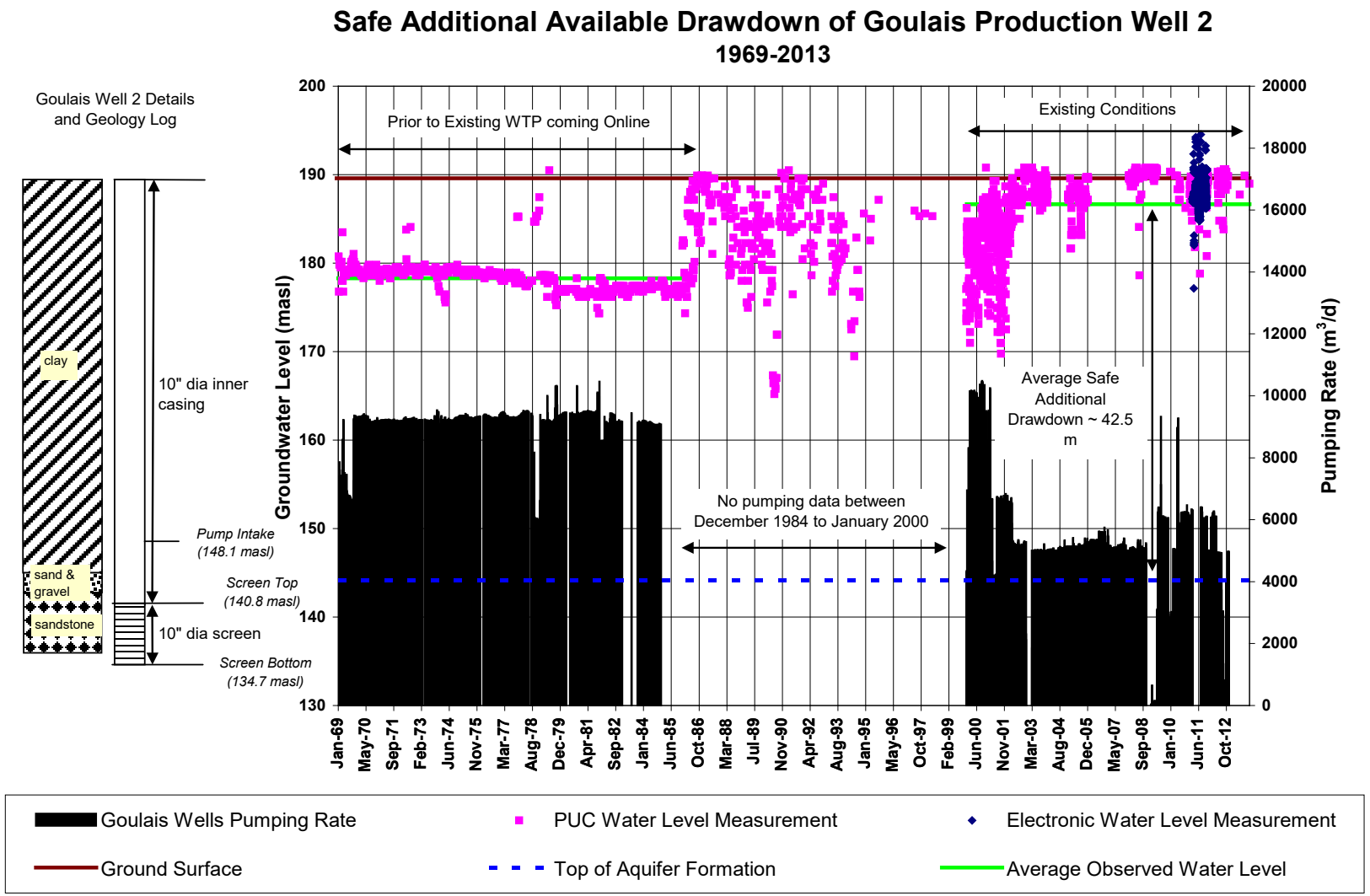


Figure 8-3 Steelton Well Average Safe Additional Available Drawdown

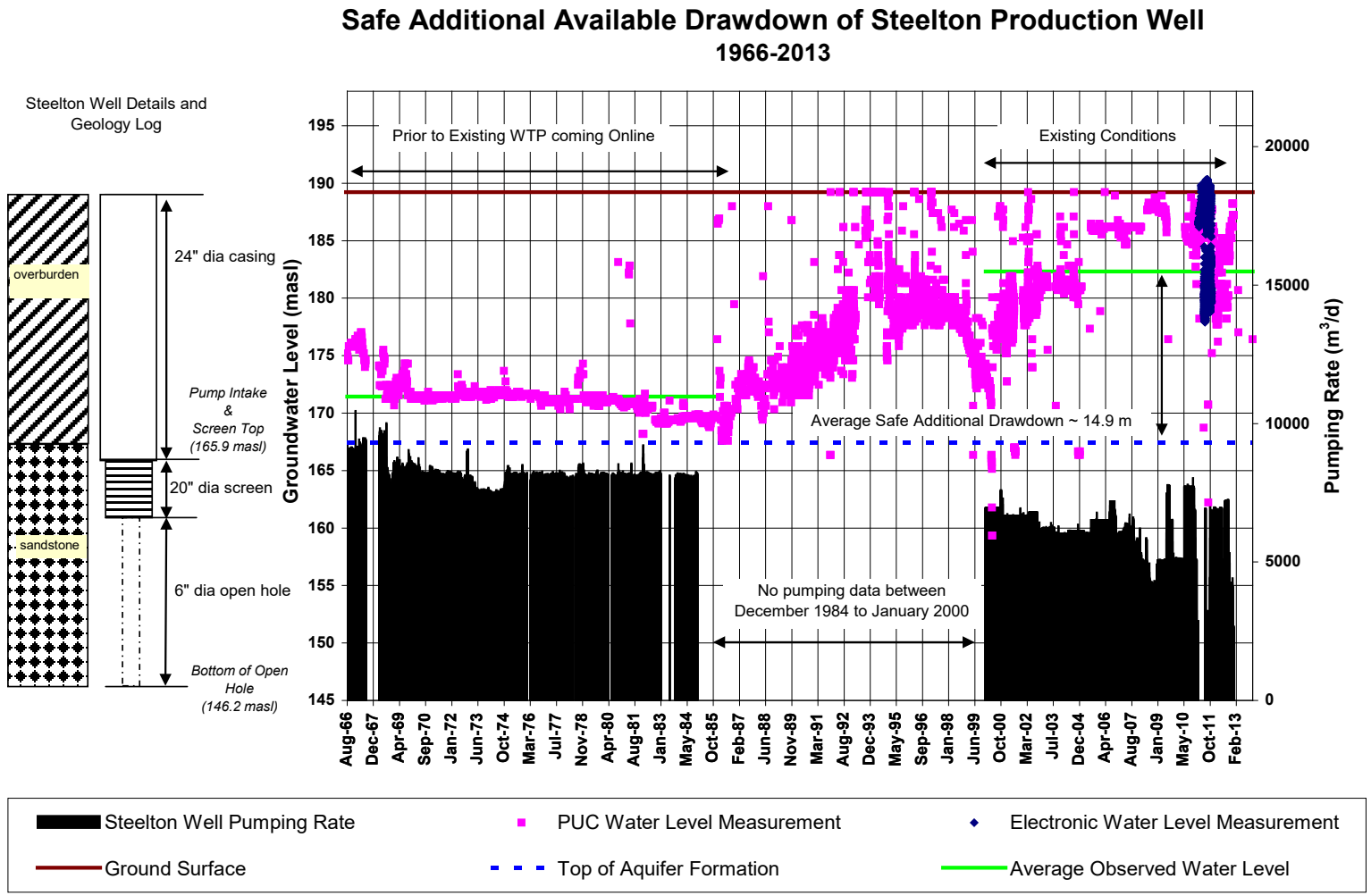


Figure 8-4 Shannon Well Average Safe Additional Available Drawdown

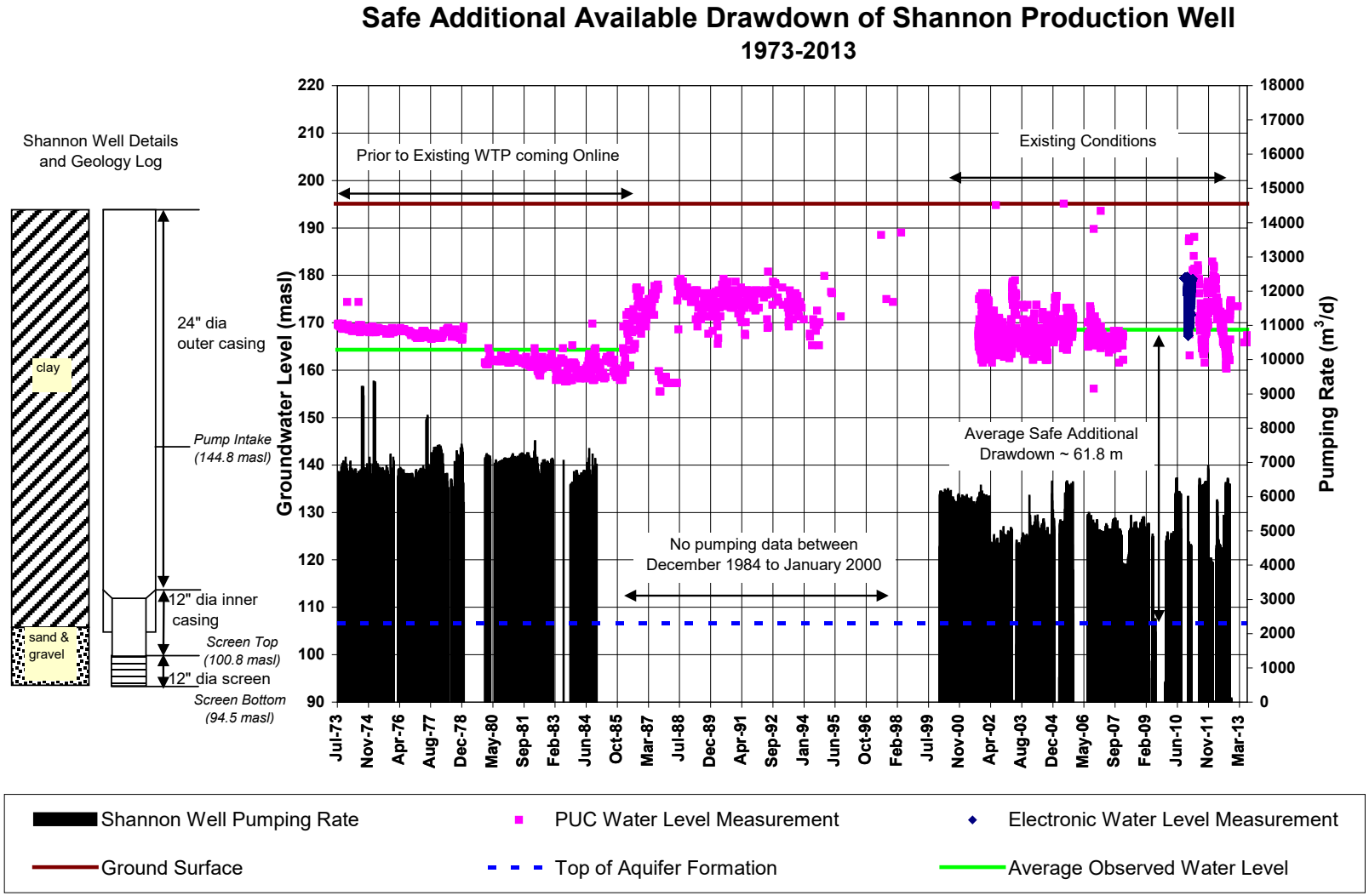
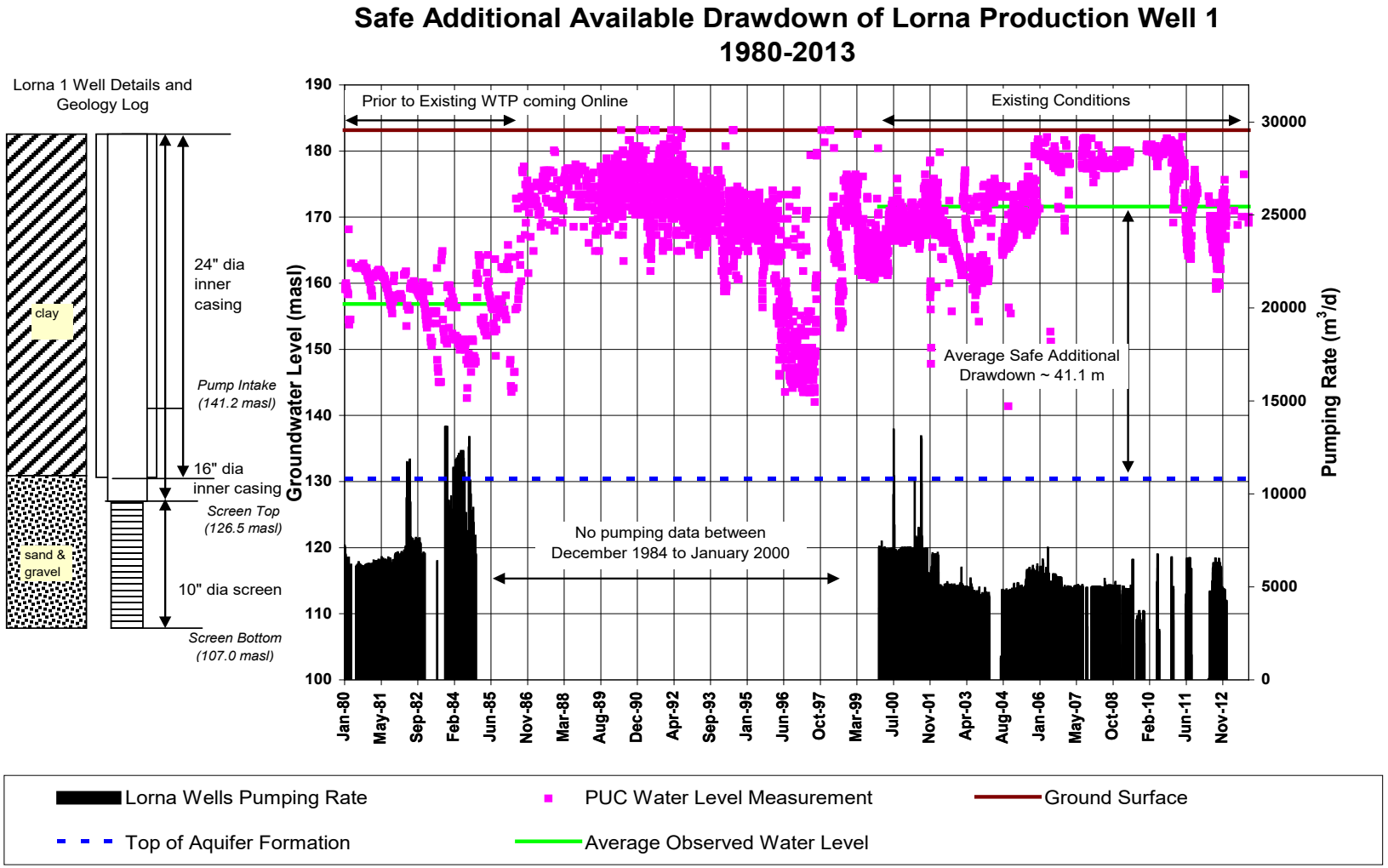
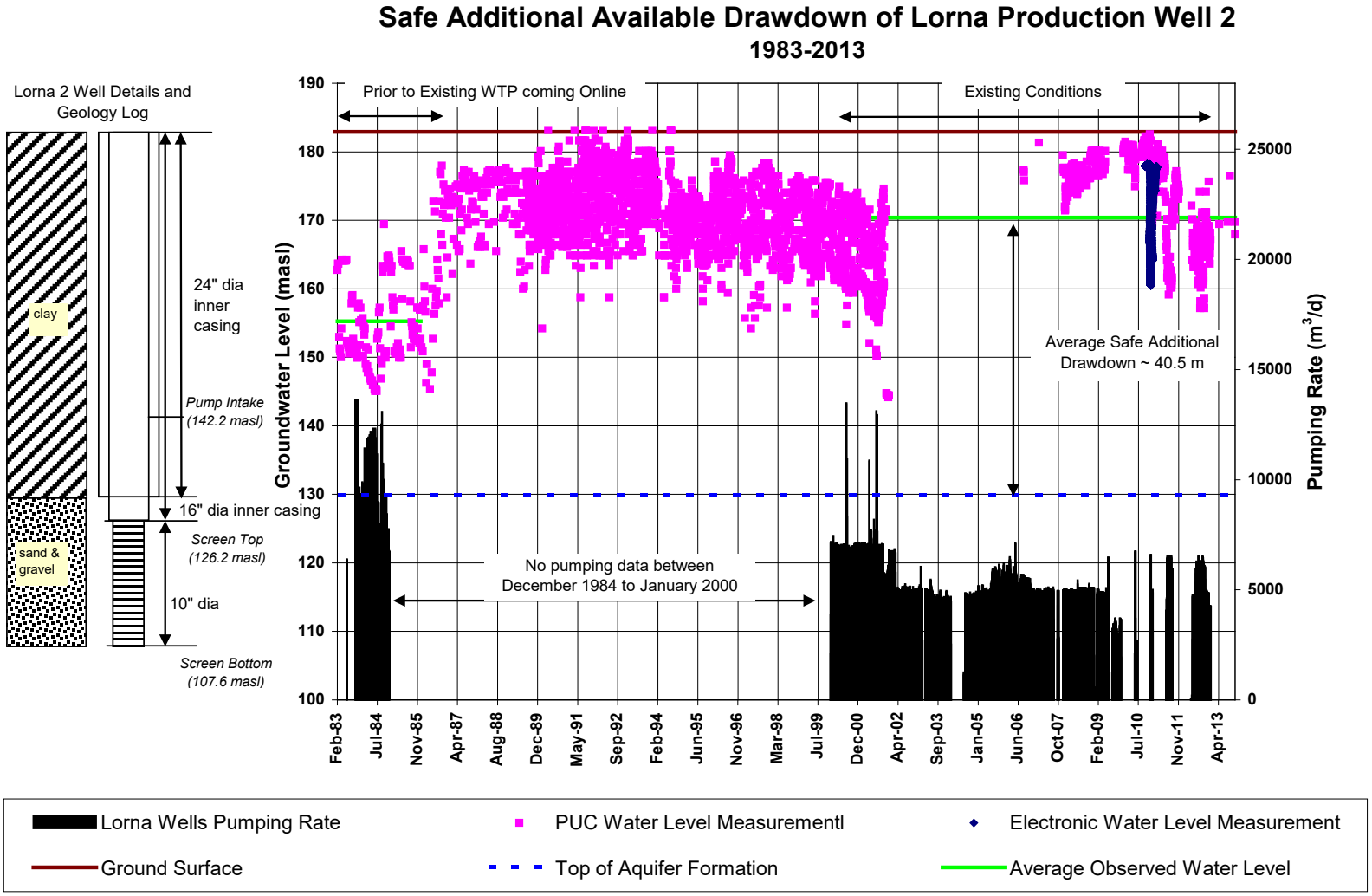


Figure 8-5 Lorna Well 1 Average Safe Additional Available Drawdown



Note: Electronic water level data are unavailable for Lorna Well 1.

Figure 8-6 Lorna Well 2 Average Safe Additional Available Drawdown



8.1 SCENARIO C – EXISTING CONDITIONS, AVERAGE CLIMATE

Scenario C evaluates the ability for existing municipal water supply wells to maintain existing average monthly pumping rates under average climate conditions. This scenario was simulated with the calibrated steady-state groundwater model using existing average annual pumping and the average annual groundwater recharge distribution from the calibrated surface water model. Scenario C therefore represents the same conditions as those which have been used to calibrate the groundwater model.

8.2 SCENARIO G – AVERAGE CLIMATE

The scenarios referred to collectively as Scenario G are designed to assess the ability for existing and planned wells to support existing and allocated pumping rates (existing plus committed) under average climate conditions and with changes in land use. These scenarios are simulated using the calibrated steady-state groundwater flow model using groundwater recharge rates that reflect long-term average climate conditions. Scenario G is subdivided into three scenarios (G(1), G(2), and G(3)) so as to assess the impacts of municipal pumping and land use changes individually.

8.3 SCENARIO G(1) – EXISTING PLUS COMMITTED DEMAND, FUTURE LAND USE, AVERAGE CLIMATE

Scenario G(1) evaluates both the impact of increased municipal pumping rates (existing plus committed rates) on the municipal water supply and other water uses as well as future land use conditions. Average annual pumping rates as well as average annual recharge rates are used for this scenario.

The resulting hydraulic head distribution of Scenario C is used as initial conditions for this scenario.

8.4 SCENARIO G(2) – EXISTING PLUS COMMITTED DEMAND, EXISTING LAND USE, AVERAGE CLIMATE

Scenario G(2) evaluates the impact of increased municipal pumping rates (existing plus committed rates) on the municipal water supply and other water uses. The existing land use is assumed. Average annual pumping rates and average annual recharge rates are used for this scenario.

The resulting hydraulic head distribution of Scenario C is used as initial conditions for this scenario.

8.5 SCENARIO G(3) – EXISTING DEMAND, FUTURE LAND USE, AVERAGE CLIMATE

Scenario G(3) evaluates the impact of a reduction in recharge due to land development on the water levels in the municipal wells. Future land use conditions are assumed. Average annual pumping rates and average annual recharge rates are used for this scenario.

The resulting hydraulic head distribution of Scenario C is used as initial conditions for this scenario.

8.6 SCENARIO D – EXISTING DEMAND, EXISTING LAND USE, DROUGHT PERIOD

Scenario D aims to evaluate whether each municipal well is able to pump at its existing rate during a drought period. This scenario is run as a transient simulation using the calibrated Tier Three groundwater flow model. As per the Technical Rules, a drought period of 10 years was established from available historical precipitation records. The drought period has been determined based on the 10 consecutive years with the lowest precipitation and is found to extend from 1955 to 1964. Average monthly pumping rates for the existing water demand at each individual well field and average monthly recharge rates obtained from the surface water model for the 10-year timeframe are applied in the groundwater flow model throughout the duration of the simulation. The results from Scenario C are used as initial conditions for Scenario D.

8.7 SCENARIO H – DROUGHT PERIOD

The scenarios referred to collectively as Scenario H are designed to evaluate the ability of municipal wells to maintain allocated pumping rates (existing plus committed) under drought conditions (established from the 10-year drought period defined in Scenario D). The groundwater flow model is run in transient mode, similarly to Scenario D, to examine the combined impact of drought conditions, land use change, and additional municipal pumping on water levels at municipal wells.

Scenario H is subdivided into three scenarios (H(1), H(2), and H(3)) so as to assess the impacts of municipal pumping and land use changes individually.

8.8 SCENARIO H(1) – EXISTING PLUS COMMITTED DEMAND, FUTURE LAND USE, DROUGHT PERIOD

Scenario H(1) evaluates the combined impacts of increased pumping and a reduction in recharge on the water levels in the municipal wells under drought conditions. Pumping rate and recharge input parameters for this scenario include average monthly pumping rates for the existing plus committed water demand at each individual well field, and average monthly recharge rates obtained from the surface water model for the 10-year drought period to simulate drought conditions. Future land use conditions are assumed. Again, the results from Scenario C are used as initial conditions.

8.9 SCENARIO H(2) – EXISTING PLUS COMMITTED DEMAND, EXISTING LAND USE, DROUGHT PERIOD

Scenario H(2) evaluates the impact of increased pumping only on the water levels in the municipal supply wells under drought conditions. Pumping rate and recharge input parameters for this scenario include average monthly pumping rates for the existing plus committed water demand at each individual well field, and average monthly recharge rates obtained from the surface water model for the 10-year drought period to simulate drought conditions. Existing land use conditions are assumed. Again, the results from Scenario C are used as initial conditions.

8.10 SCENARIO H(3) – EXISTING DEMAND, FUTURE LAND USE, DROUGHT PERIOD

Scenario H(3) evaluates the impact of a reduction in recharge due to land development on the water levels in the municipal supply wells under drought conditions. Pumping rate and recharge input parameters for this scenario include average monthly pumping rates for the existing water demand at each individual well field, and average monthly recharge rates obtained from the surface water model for the 10-year drought period to simulate drought conditions. Future land use conditions are assumed. Again, the results from Scenario C are used as initial conditions.

9.0 MODEL-PREDICTED SCENARIO RESULTS

The model-predicted scenario results are presented in the following sections. The results for Scenario C are presented in terms of groundwater levels. The results from all subsequent scenarios are presented in terms of additional drawdown relative to the groundwater levels obtained in Scenario C (baseline conditions).

9.1 SCENARIO C GROUNDWATER LEVELS

The steady-state groundwater level distribution for Scenario C in the Model Layer 6 (Layer 6) and Model Layer 7 (Layer 7) is shown on **Map 4-1** and **Map 4-2**, respectively. Layer 6 is the glaciolacustrine aquifer in which the Lorna and Shannon wells are screened and is predominant in the East Basin. Layer 7 is the Precambrian sandstone aquifer in which the Goulais and Steelton Wells are screened. The predominant groundwater flow direction in the Central and East Basins is from north to south and from north to southeast, respectively.

9.2 SCENARIO G(1) ADDITIONAL DRAWDOWNS

The steady-state additional drawdown distribution for Scenario G(1) in Layer 6 is shown on **Map 4-3**. The drawdown distribution around the Lorna and Shannon Wells is shown in more detail on **Map 4-3A**.

The steady-state additional drawdown distribution for Scenario G(1) in Layer 7 is shown on **Map 4-4**. The drawdown distribution around the Goulais and Steelton Wells is shown in more detail on **Map 4-4A**.

Additional drawdown contours are observed in the northern area of the Central Basin on **Maps 4-3** to **4-4** .Given that Scenario G(1) considers future land use conditions relative to baseline conditions, these contours are attributed to the projected reduction in recharge expected to occur in that area.

9.3 SCENARIO G(2) ADDITIONAL DRAWDOWNS

The steady-state additional drawdown distribution for Scenario G(2) in Layer 6 is shown on **Map 4-5**. The drawdown distribution around the Lorna and Shannon Wells is shown in more detail on **Map 4-5A**.

The steady-state additional drawdown distribution for Scenario G(2) in Layer 7 is shown on **Map 4-6**. The drawdown distribution around the Goulais and Steelton Well is shown in more detail on **Map 4-6A**.

In both model layers, it is observed that the additional drawdown contours occur primarily around the municipal wells. This is attributed to a reduction in water level (relative to Scenario C) caused by the increase in pumping rates which is simulated to represent projected water demands.

9.4 SCENARIO G(3) ADDITIONAL DRAWDOWNS

The steady-state additional drawdown distribution for Scenario G(3) in Layer 6 is shown on **Map 4-7**. The steady-state additional drawdown distribution for Scenario G(3) in Layer 7 is shown on **Map 4-8**.

Additional drawdown contours are observed in the northern area of the Central Basin on **Maps 4-7** and **4-8**. Given that Scenario G(3) considers future land use conditions relative to baseline conditions, these contours are attributed to the projected reduction in recharge expected to occur in that area.

No additional drawdown (relative to Scenario C) occurs around the municipal supply wells since both Scenario C and Scenario G(3) consider existing municipal pumping rates.

In general, it can be noted that only localized but no major overall differences in additional drawdown can be recognized when comparing Scenario G(1); Scenario G(2) and Scenario G(3). This suggests that the proposed future pumping rates as well as the proposed future land use changes do not influence the general drawdown distribution in the model domain to a great extent.

9.5 SCENARIO D ADDITIONAL DRAWDOWNS

The model-predicted additional drawdown distribution for Scenario D in Layer 6 is shown on **Map 4-9**. The map illustrates the drawdown distribution for the model date at which the maximum drawdown occurred at the Lorna and Shannon Wells during the simulation. .

Maps 4-10 and **4-11** show the predicted additional drawdown distribution for Scenario D in Layer 7. **Map 4-10** illustrates the additional drawdown contours for the model date at which the maximum additional drawdown occurred at the Goulais Wells. Similarly, **Map 4-11** illustrates the additional drawdown contours for the model date at which the maximum additional drawdown occurred at the Steelton Well.

From the three maps, it is observed that, in comparison with Scenario C, additional drawdown contours are generally observed across the entire model domain (i.e., water levels are lower) due to the reduced recharge used for all drought scenarios. This effect is amplified in the northwestern portion of the model domain due to the large reduction in recharge from long-term average conditions in this area under drought conditions.

9.6 SCENARIO H(1) ADDITIONAL DRAWDOWNS

The model-predicted additional drawdown distribution for Scenario H(1) in Layer 6 is shown on **Map 4-12**. These contours are representative of the drawdown at the model date when the maximum additional drawdown occurred at the Lorna and Shannon Wells during the simulation.

Maps 4-13 and **4-14** show the predicted drawdown distribution for Scenario H(1) in Layer 7. **Maps 4-13** and **4-14** illustrate the drawdown contours for the model date at which the maximum additional drawdown occurred at the Goulais Wells and the Steelton Well, respectively.

Again, it is observed that, in comparison with Scenario C, additional drawdown contours are generally observed across the entire model domain (i.e., water levels are lower) due to the reduced recharge used for all drought scenarios. This effect is amplified in the northwestern portion of the model domain due to the large reduction in recharge from long-term average conditions in this area under drought conditions.

9.7 SCENARIO H(2) ADDITIONAL DRAWDOWNS

The model-predicted additional drawdown distribution for Scenario H(2) in Layer 6 is shown on **Map 4-15**. These contours are representative of the drawdown at the model date when the maximum additional drawdown occurred at the Lorna and Shannon Wells during the simulation.

Maps 4-16 and **4-17** show the predicted drawdown distribution for Scenario H(2) in Layer 7. **Maps 4-16** and **4-17** illustrate the drawdown contours for the model date at which the maximum additional drawdown occurred at the Goulais Wells and the Steelton Well, respectively.

Again, it is observed that, in comparison with Scenario C, additional drawdown contours are generally observed across the entire model domain (i.e., water levels are lower) due to the reduced recharge used for all drought scenarios. This effect is amplified in the northwestern portion of the model domain due to the large reduction in recharge from long-term average conditions in this area under drought conditions.

9.8 SCENARIO H(3) ADDITIONAL DRAWDOWNS

The model-predicted additional drawdown distribution for Scenario H(3) in Layer 6 is shown on **Map 4-18**. These contours are representative of the drawdown at the model date when the maximum additional drawdown occurred at the Lorna and Shannon Wells during the simulation.

Maps 4-19 and **4-20** show the predicted drawdown distribution for Scenario H(3) in Layer 7. **Maps 4-19** and **4-20** illustrate the drawdown contours for the model date at which the maximum additional drawdown occurred at the Goulais Wells and the Steelton Well, respectively.

Again, it is observed that, in comparison with Scenario C, additional drawdown contours are generally observed across the entire model domain (i.e., water levels are lower) due to the reduced recharge used for all drought scenarios. This effect is amplified in the northwestern portion of the model domain due to the large reduction in recharge from long-term average conditions in this area under drought conditions.

In general, it can be noted that only localized but no major overall differences in additional drawdown can be recognized when comparing Scenario H(1); Scenario H(2) and Scenario H(3). This suggests that the proposed future pumping rates as well as the proposed future land use changes do not influence the general drawdown distribution in the model domain to a great extent.

9.9 ADDITIONAL DRAWDOWNS IN MUNICIPAL WELLS

The additional drawdowns at every municipal well for each of the risk assessment model scenarios were obtained and compared to the estimated average safe additional available drawdown. The results from Scenario C were used as the baseline for the calculation of the additional drawdowns. For example, for the steady-state Scenarios G(1), G(2) and G(3), the difference between the water levels at the wells for Scenario C and those at the end of the model simulation for Scenarios G(1); G(2) and G(3), respectively, were recorded as the scenario model additional drawdowns.

For the transient scenarios (Scenarios D, H(1), H(2) and H(3)), the maximum additional drawdown at each municipal well was calculated by subtracting the lowest simulated water level elevation in the aquifer at the location of the well from the corresponding water level under baseline conditions (Scenario C). Table 9.1 summarizes the maximum additional drawdown at each of the municipal wells for the four risk assessment scenarios. The model-simulated drawdowns were then compared to the average safe additional available drawdowns to identify municipal wells where there is a potential for the well to be unable to pump at its allocated rate. Table 9.1 presents the results of this comparison.

Table 9.1 Maximum Additional Drawdown in Municipal Wells for each Risk Assessment Scenario

Wellfield	Average Safe Additional Available Drawdown (m)	Water Level Elevation, Scenario C (masl)	Minimum Simulated Water Level Elevation (masl)							Maximum Simulated Additional Drawdown (m)						
			Average Climate			Drought				Average Climate			Drought			
			G(1)	G(2)	G(3)	D	H(1)	H(2)	H(3)	G(1)	G(2)	G(3)	D	H(1)	H(2)	H(3)
Goulais	40.2 (Goulais Well 1) 42.5 (Goulais Well 2)	183.8	182.3	182.4	183.7	170.5	170.4	170.4	170.5	1.5	1.4	0.1	13.3	13.4	13.4	13.3
Steelton	14.9	177.4	175.5	175.7	177.3	164.9	164.8	164.8	164.9	1.9	1.7	0.1	12.5	12.6	12.6	12.5
Shannon	61.8	179.8	178.8	178.8	179.7	176.3	176.2	176.2	176.3	1.0	0.9	0.1	3.5	3.6	3.6	3.5
Lorna	41.1 (Lorna Well 1) 40.5 (Lorna Well 2)	176.6	175.2	175.3	176.6	171.2	171.1	171.1	171.2	1.4	1.3	0.0	5.4	5.5	5.5	5.4

From the above table, the model-simulated additional drawdown is less than the established average safe additional available drawdown at all of the municipal wells for each of Scenarios G(1); G(2); G(3); D; H(1); H(2) and H(3).

Figure 9-1 to Figure 9-4 show plots of maximum simulated additional drawdown versus time at each of the municipal wells for the transient scenarios (Scenario D, Scenario H(1), Scenario H(2) and Scenario H(3)).

Figure 9-1 Model-Simulated Additional Drawdowns at the Goulais Wells for Scenarios D; H(1); H(2) and H(3)

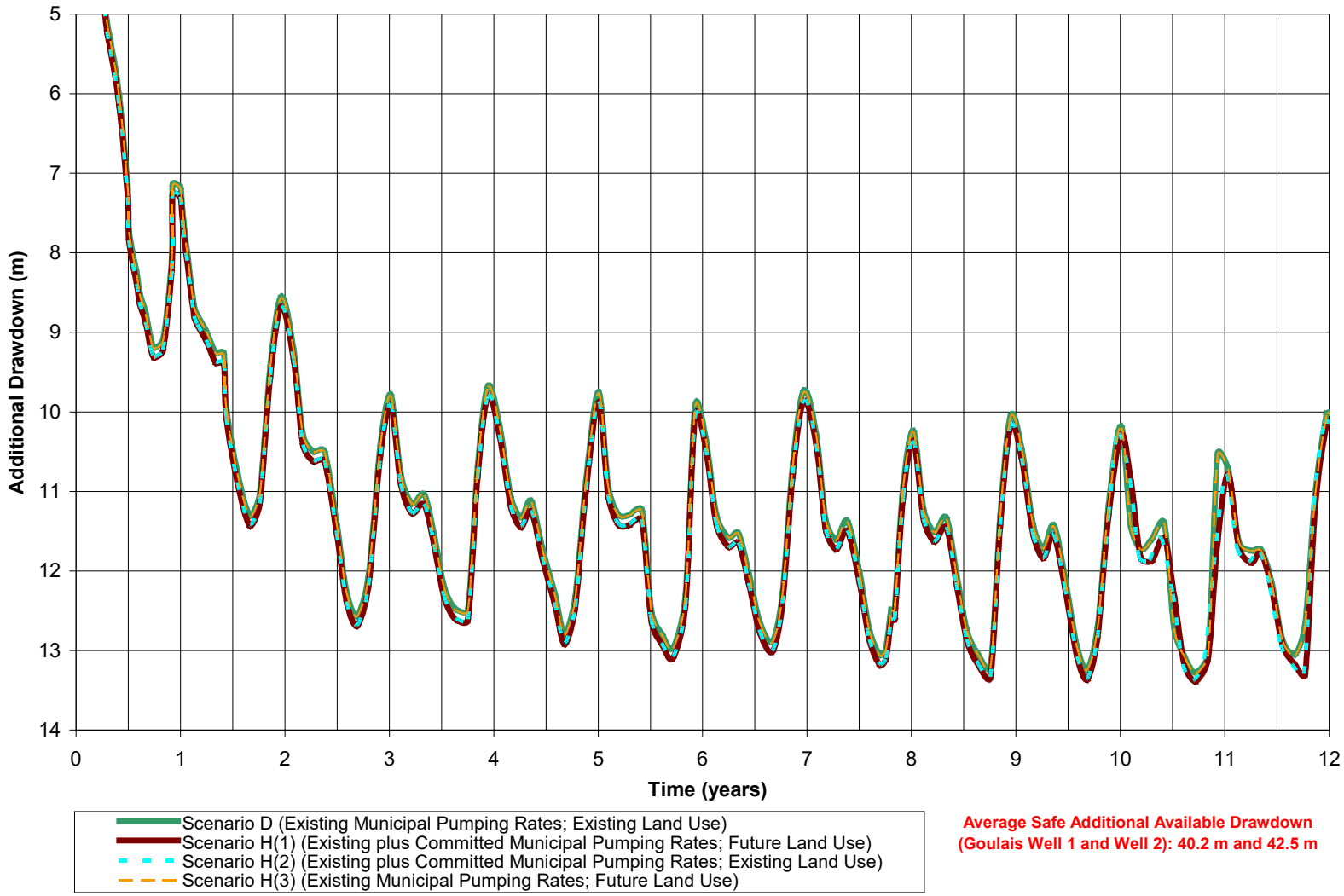


Figure 9-2 Model-Simulated Additional Drawdowns at the Steelton Well for Scenarios D; H(1); H(2) and H(3)

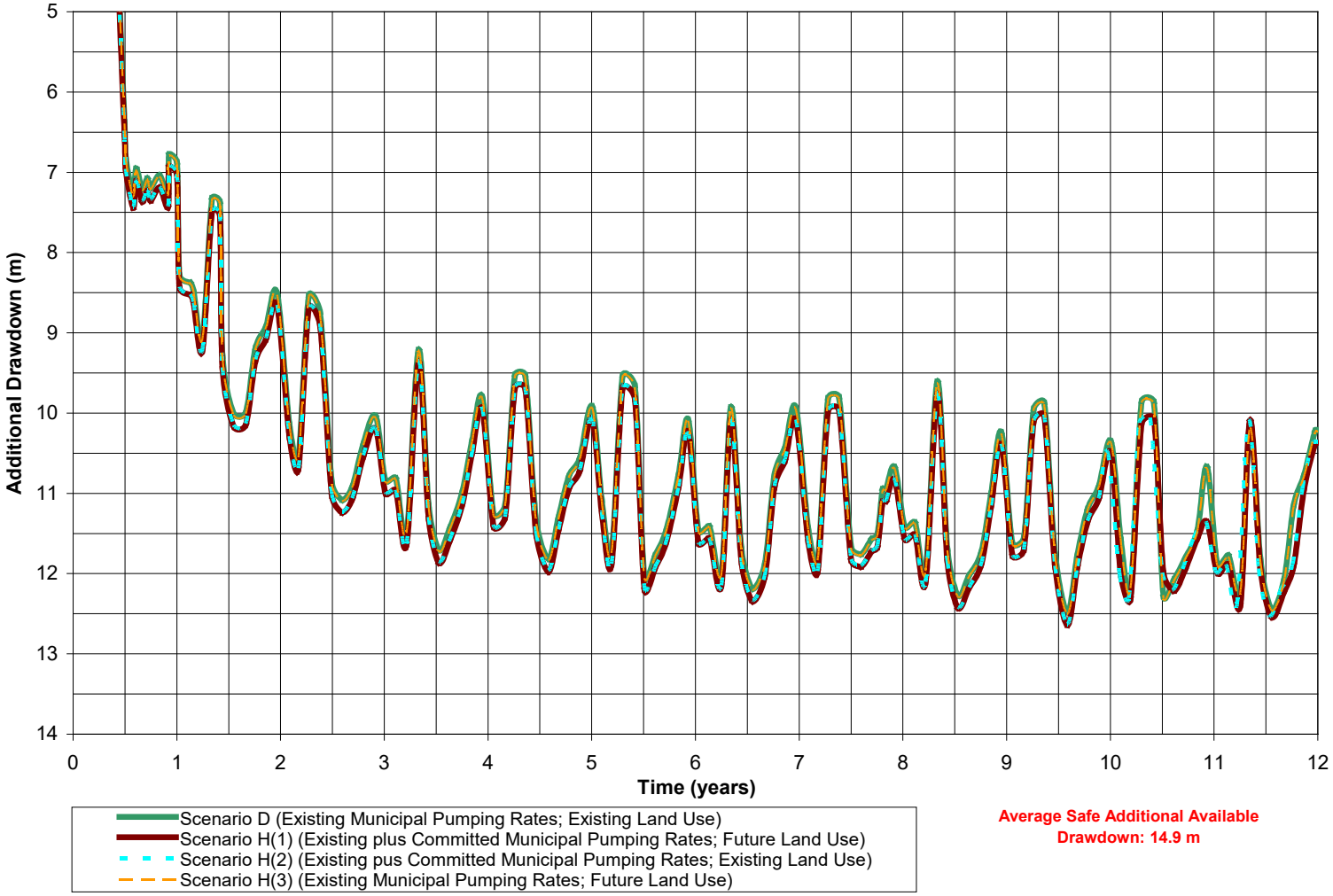


Figure 9-3 Model-Simulated Additional Drawdowns at the Shannon Well for Scenarios D; H(1); H(2) and H(3)

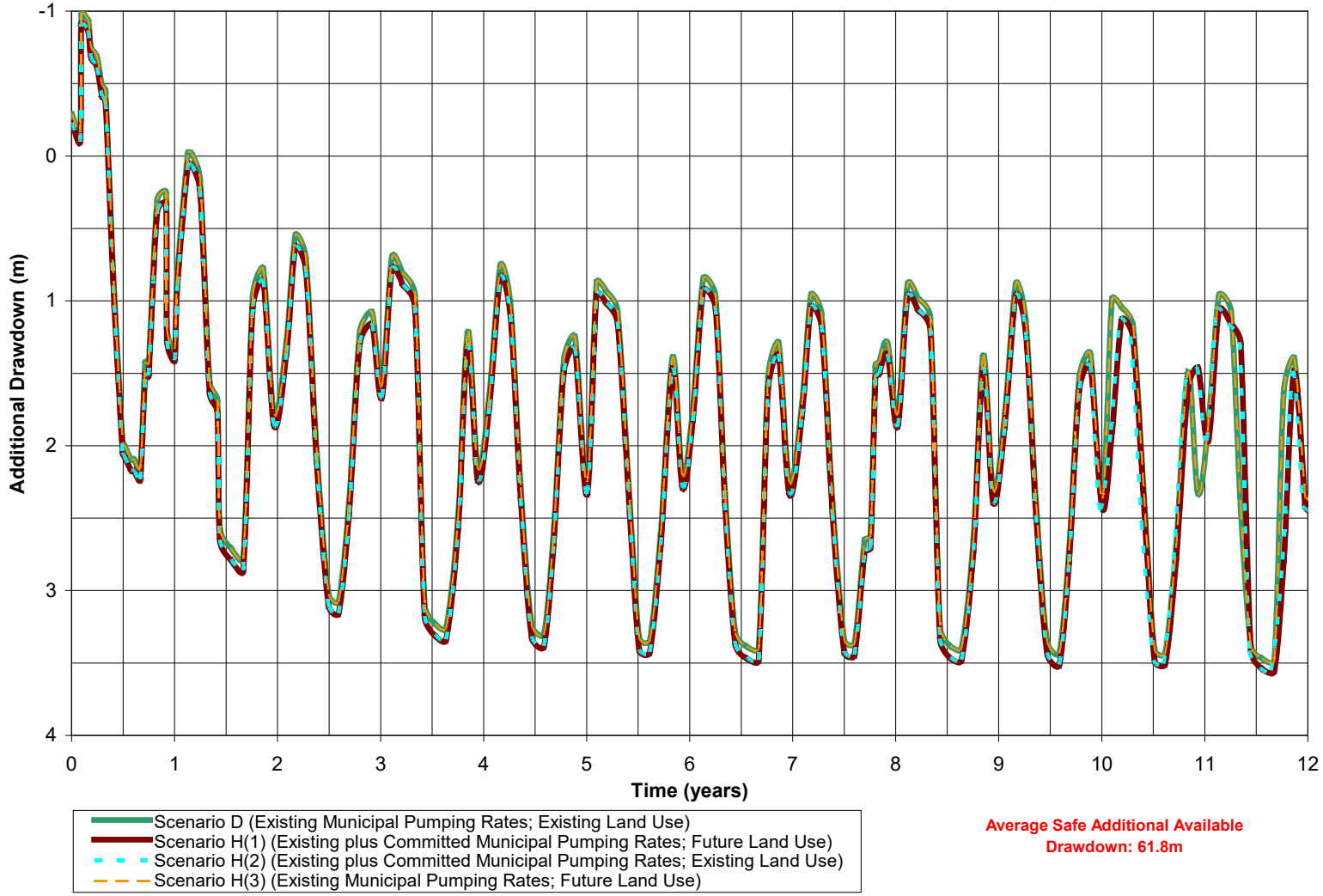
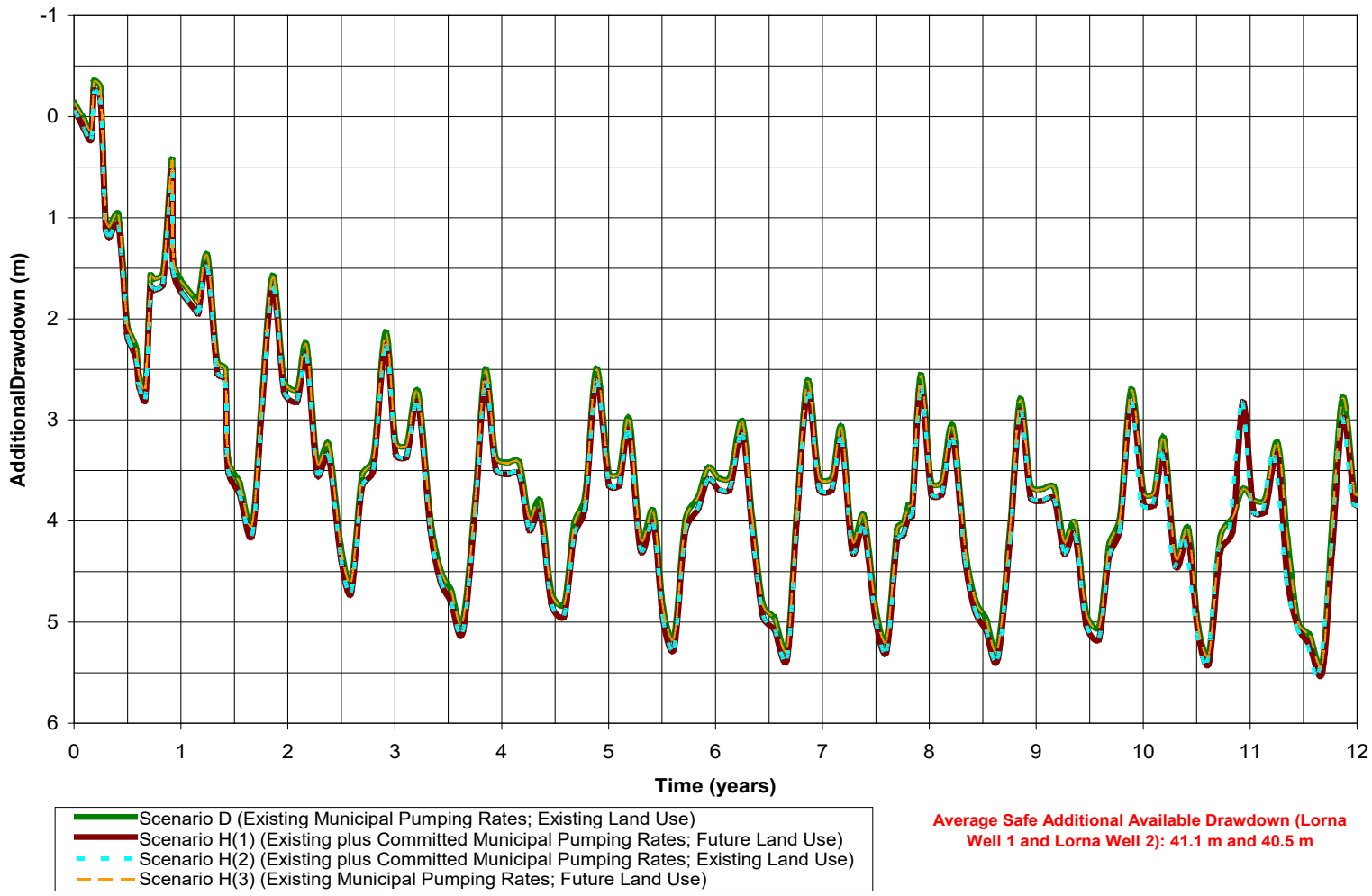


Figure 9-4 Model-Simulated Additional Drawdowns at the Lorna Wells for Scenarios D; H(1); H(2) and H(3)



9.10 CORRECTED ADDITIONAL DRAWDOWNS

Data from step tests conducted on the City of Sault Ste. Marie municipal supply wells have been assembled and analyzed to estimate nonlinear well loss coefficients (C). Estimates of C are required to account for the additional component of drawdown in the pumping wells due to nonlinear flow processes. The estimates of C, summarized in Table 9.2 below, vary over a relatively wide range, but are generally below the Walton (1962) suggested upper limit for a well in good condition (C < 1900 s^2/m^5), with the exception of Goulais Well 2.

Table 9.2 Estimated Nonlinear Well Loss Coefficient

Well	Estimated nonlinear well loss coefficient, C (s^2/m^5)
Goulais Well 1	1480
Goulais Well 2	4810
Steelton Well	370
Shannon Well	~ 0
Lorna Well 1	147
Lorna Well 2	1200

The nonlinear well loss coefficient, C, is estimated from an analysis of step test data. In a step test, a production well is pumped for relatively brief intervals at a sequence of increasing rates. Available step test data for the SSMSPA wells are limited. Therefore, a deliberately simplified approach is adopted to estimate the nonlinear well loss coefficient. The approach for estimating C is standard-practice and is referred to as a Hantush-Bierschek analysis (Hantush, 1964; Bierschenk, 1964). Details on this analysis can be found in Appendix D.

Simulated water levels in the Sault Ste. Marie municipal wells are adjusted in the following way:

$$s_{well-adjusted} = s_{model} + \Delta s_{wellblock} + \Delta s_{nonlinear}$$

Where $\Delta s_{wellblock}$ accounts for the fact that the model yields the average water level in the numerical grid block that contains the well, and $\Delta s_{nonlinear}$ represents turbulent head losses that are not considered in a standard numerical simulator (nonlinear near-well or in-well losses).

The wellblock correction can be estimated as (Rushton and Herbert, 1966; Prickett, 1967; Peaceman, 1978):

$$\Delta s_{wellblock} = \frac{Q}{2\pi T} \ln\left\{\frac{0.2\Delta x}{r_w}\right\}$$

Where Q is the pumping rate, T is the transmissivity of the grid block that contains the well, Δx is the size of the grid block and r_w is the actual radius of the pumping well. If the well is located in a rectangular grid block with dimensions Δx and Δy, the above equation is generalized as (Peaceman, 1983):

$$\Delta s_{wellblock} = \frac{Q}{2\pi T} \ln\left\{\frac{0.14\left[(\Delta x)^2 + (\Delta y)^2\right]^{0.5}}{r_w}\right\}$$

An estimate of the nonlinear well loss coefficient, C, is required to account for the nonlinear head losses (Jacob, 1946):

$$\Delta s_{nonlinear} = CQ^2$$

For any predictive (scenario) simulations, the focus is on the differences in water levels between any two simulations, rather than absolute water levels. The results of the numerical simulations have to be adjusted for changes in water levels in pumping wells.

The pumping rate for base case conditions (Scenario C) is denoted as Q_1, the simulated water level in the grid block that contains the well as $WL(Q_1)$, the pumping rate for a predictive scenario (e.g G(1); G(2); G(3); D; H(1); H(2); H(3)) as Q_2, and the corresponding water level in the well as $WL(Q_2)$.

The adjusted water levels in the pumping wells for the base case (Scenario C) and one of the predictive scenarios (Scenario G(1); G(2); G(3); D; H(1); H(2); H(3)) are:

$$WL(Q_1)_{adjusted} = WL(Q_1)_{simulated} - \frac{Q_1}{2\pi T}\ln\left\{\frac{0.14\left[(\Delta x)^2 + (\Delta y)^2\right]^{0.5}}{r_w}\right\} - CQ_1^2$$

$$WL(Q_2)_{adjusted} = WL(Q_2)_{simulated} - \frac{Q_2}{2\pi T}\ln\left\{\frac{0.14\left[(\Delta x)^2 + (\Delta y)^2\right]^{0.5}}{r_w}\right\} - CQ_2^2$$

The corrected additional drawdown (AD) for a predictive scenario (G(1); G(2); G(3); D; H(1); H(2); H(3)) with respect to the base case (Scenario C) is:

$$AD(1 \rightarrow 2) = WL(Q_1)_{adjusted} - WL(Q_2)_{adjusted}$$

Substituting for the adjusted water levels and simplifying yields the corrected additional drawdown calculation including the modelling block correction (Peaceman) and the nonlinear well loss correction (Jacob):

$$AD(1 \rightarrow 2) = WL(Q_1)_{simulated} - WL(Q_2)_{simulated} + \underbrace{\frac{(Q_2 - Q_1)}{2\pi T}\ln\left\{\frac{0.14\left[(\Delta x)^2 + (\Delta y)^2\right]^{0.5}}{r_w}\right\}}_{Peaceman\ \ correction} + \underbrace{C(Q_2^2 - Q_1^2)}_{Jacob\ \ correction}$$

In Table 9.3, the resulting sum of the Peaceman correction term plus the Jacob correction term are summarized for each well and each risk assessment scenario. All these

calculations have been carried out with respect to the existing pumping rate for Scenario C.

Table 9.3 Sum of Peaceman Correction and Jacob Correction

Wellfield	Sum of Peaceman Correction and Jacob Correction (m)						
	G(1)	G(2)	G(3)	D	H(1)	H(2)	H(3)
Goulais	2.2	2.2	0.0	0.0	2.2	2.2	0.0
Steelton	0.6	0.6	0.0	0.0	0.6	0.6	0.0
Shannon	0.2	0.2	0.0	0.0	0.2	0.2	0.0
Lorna	0.6	0.6	0.0	0.0	0.6	0.6	0.0

The amount of Peaceman correction and Jacob correction, which both depend on differences in pumping rates for two scenarios, are equal to zero for Scenario G(3); Scenario D and Scenario H(3) as pumping rates are identical to Scenario C (i.e., existing demand). Also, the amount of Peaceman correction and Jacob correction is always identical for Scenario G(1); Scenario G(3); Scenario H(1) and Scenario (H2), respectively, as allocated pumping rates for these four (4) scenarios are identical (i.e., existing plus committed).

To calculate maximum corrected simulated additional drawdowns for each risk assessment scenario, the amount of Peaceman correction and Jacob correction is added to the maximum simulated additional drawdown as presented in Table 9.4. The resulting maximum corrected simulated additional drawdown is shown in Table 9.4.

Table 9.4 Maximum Corrected Additional Drawdown in Municipal Wells for each Risk Assessment Scenario

Wellfield	Average Safe Additional	Maximum Corrected Simulated Additional Drawdown (m)

	Available Drawdown (m)	Average Climate			Drought			
		G(1)	G(2)	G(3)	D	H(1)	H(2)	H(3)
Goulais	40.2 (Goulais Well 1) 42.5 (Goulais Well 2)	3.7	3.6	0.1	13.3	15.6	15.6	13.3
Steelton	14.9	2.5	2.3	0.1	12.5	13.2	13.2	12.5
Shannon	61.8	1.2	1.1	0.1	3.5	3.8	3.8	3.5
Lorna	41.1 (Lorna Well 1) 40.5 (Lorna Well 2)	2.0	1.9	0.0	5.4	6.1	6.1	5.4

Even when considering the modelling block correction (Peaceman) and nonlinear well losses (Jacob) it can be seen, from the above table, that the model-simulated corrected maximum additional drawdown is less than the established average safe additional available drawdown at all of the municipal wells for each of Scenarios G(1); G(2); G(3); D; H(1); H(2) and H(3).

9.11 IMPACTS TO GROUNDWATER DISCHARGE/SUPPLY

Baseflow reductions arising from land use changes are independent of any baseflow reductions caused by increased groundwater pumping. Since only impacts associated with groundwater pumping are used to evaluate the water quantity risk level, the Tier Three Assessment only considers baseflow impacts associated with Scenario G(2) when evaluating the risk level placed on the Local Area. The Local Area is assigned a Significant risk when groundwater discharge is reduced by more than 20% of the existing monthly baseflow (MOE and MNR, 2010).

The simulated impact on groundwater discharge to rivers and streams within the study area was assessed for Scenario G(2) by comparing the simulated groundwater discharge

for Scenario G(2) to the simulated groundwater discharge for Scenario C. The groundwater model's estimate of baseflow (discharge) to rivers for the entire model domain and the Root River Zone, using the model's mass balance calculations, is summarized in Table 9.5. The percentage reduction of discharge to rivers, as compared to Scenario C (existing conditions) is also presented in this table.

For this study, the predicted baseflow reductions for the entire model domain as well as the Root River Zone are negligible (0.36% and 0.37%, respectively). The small reduction in baseflow to rivers can be associated to increased pumping rates at the municipal wells in Scenario G(2) (existing plus committed) compared to only existing pumping rates in Scenario C. As such, there is no Significant risk associated with baseflow reduction.

Table 9.5 Baseflow (Discharge) to River Model Estimates

	Scenario C Model Mass Balance (m^3/day)	Scenario G(2) Model Mass Balance (m^3/day)	Percent Reduction (%)	Scenario C Root River Zone Mass Balance (m^3/day)	Scenario G(2) Root River Zone Mass Balance (m^3/day)	Percent Reduction (%)
Baseflow (Discharge) to Rivers	68623	68376	0.36	9155	9121	0.37

9.12 IMPACTS TO OTHER WATER USES

The following other water uses have been identified in the Revised Conceptual Understanding Report (Kresin/CEG, 2013). Possible impacts of increased pumping on these water uses are discussed below.

Provincially Significant Wetlands

Wetlands are known to exist south of the Precambrian uplands as a result of local-scale discharge of groundwater through the coarse permeable materials. The shallow system provides groundwater flux to the streams and is essential for preserving the natural function of the ecosystem (Kresin/CEG, 2013).

Map 4-21 illustrates the wetlands within the planning area, which comprise approximately 4 % of the study area. There are several smaller wetland areas in the northern uplands of the planning region associated with headwater areas of the rivers and creeks, which flow

south towards the St. Marys River. Along the shores of the St. Marys River, larger wetland areas are found at the outlet of rivers such as the Big and Little Carp and the Root River (Kresin/CEG, 2013).

As shown on **Map 4-21**, only a very small portion of the wetland areas mentioned above lie within the groundwater model limits and within the WHPA-Q1. Given that increased pumping simulated in Scenarios G(1) and G(2) only affects the hydraulic head distribution in the deeper aquifer units to a large extent, significant adverse impacts on the shallow aquifer system which is connected to the wetland areas are not anticipated. Therefore, adverse effects on the wetland areas are expected to be minimal when increased pumping occurs (Scenarios G(1) and G(2)) compared to Scenario C.

Recreation

Five conservation areas are located within the SSMR SPA. These areas are used for recreation and consist of campgrounds, trails and provincial parks. The conservation areas, shown on **Map 4-21**, cover 1865 hectares of diverse ecosystem including forest, wetlands and shorelines (Sault Ste. Marie Region Conservation Authority, 2013).

Only two conservation areas lie within the model limits, namely the Hiawatha Highlands and Fort Creek. Of these two areas, only Fort Creek is located within the WHPA-Q1 (**Map 4-21**). Given that increased pumping simulated in Scenarios G(1) and G(2) only affects the hydraulic head distribution in the deeper aquifer units to a large extent, significant adverse impacts on the shallow aquifer system connected to the conservation areas are not anticipated. Therefore, adverse effects on the conservation areas located within the groundwater model limit are expected to be minimal when increased pumping occurs (Scenarios G(1) and G(2)) compared to Scenario C.

Wastewater Assimilation

The City of Sault Ste. Marie manages wastewater with two wastewater treatment plants. The east-end treatment plant discharges directly to the lower St. Marys River and the west-end treatment plant discharges to Leigh's Bay, in the upper St. Marys River. The withdrawals from the municipal wells are not expected to affect the assimilative capacity of the St. Marys River with respect to the wastewater treatment plants. Also, since municipal well water takings take place in the deeper layers, no direct impact on the St. Marys River baseflow is expected.

Non-Municipal Water Demand

A combined total of 21 active non-municipal permitted water takers were identified in the Central and East Basins, with 17 identified in the Central Basin and four identified in the East Basin (Kresin/CEG, 2013).

Of the above permitted water takers, a total of 11 groundwater takers were identified within the two basins. The groundwater takings are primarily used for remediation purposes. The remaining identified permitted water takers were found to consist of eight surface water takers and two other water takers which withdraw water from both groundwater and surface water sources for aquacultural purposes (Kresin/CEG, 2013).

As shown on **Map 4-22**, all of the identified non-municipal permitted groundwater takings are situated outside of the WHPA-Q1. As such, adverse impacts to these non-municipal groundwater takers are expected to be minimal under increased pumping conditions (Scenarios G(1) and G(2)) relative to Scenario C.

10.0 LOCAL AREA RISK ASSESSMENT RESULTS

When assigning a water quantity risk level classification to a Local Area, a Significant risk level is used if any of the following circumstances applies for a model risk assessment scenario (MOE and MNR, 2010):

Scenario C and Scenario D:

- The quantity of water that can be taken from the groundwater in the local area is insufficient to meet the allocated quantity of water (existing plus committed demand) of the well at any time.
- The tolerance level of the drinking water system is considered Low.

Scenario G

- For any of Scenarios G(1), G(2) and G(3), the quantity of water that can be taken from the groundwater in the local area is insufficient to meet the allocated quantity of water of the well at any time.
- For either of Scenarios G(1) and G(2), a period of time exists where:
  a) the allocated quantity of water of the well results in a reduction to the flow or level of water that constitutes an unacceptable impact to other water uses, or
  b) in relation to aquatic habitat that is classified as a cold water stream, the allocated quantity of water of the well results in a reduction in groundwater discharge by an amount that is greater than:
     i. 20% of the existing estimated stream flow that is exceeded 80% of the time, or
     ii. 20% of the existing estimated average monthly base flow of the stream

Scenario H

- For any of Scenarios H(1), H(2) and H(3), the quantity of water that can be taken from the groundwater in the local area is insufficient to meet the allocated quantity of water of the well at any time.

A Moderate risk level applies only in the case of Scenario G (Scenarios G(1) and G(2)) and is assigned if either of the following two conditions are predicted (MOE and MNR, 2010):

- The allocated quantity of water of the well results in a reduction to the flow or water level which constitutes a measurable and potentially unacceptable impact to other water uses; or
- With respect to aquatic habitat classified as a coldwater stream, the allocated quantity of water of the well results in a reduction in groundwater discharge by an amount that is

- At least 10% but not greater than 20% of the existing estimated stream flow that is exceeded 80% of the time, or
- At least 10% but not greater than 20% of the average estimated monthly base flow of the stream

In the absence of all of the above circumstances, the Local Area is assigned a Low risk level.

10.2 RISK LEVEL

For all risk assessment scenarios considered, the Goulais Wells, Steelton Well, Shannon Well and Lorna Wells have demonstrated their ability to meet existing and allocated water demands under various land use and climate conditions. Therefore, the Local Area, which encompasses all six municipal wells, was assigned a Low risk level.

10.3 TOLERANCE

The municipal water supply includes one surface water intake at Gros Cap, six municipal wells and three reservoirs having capacities of 15,000 m3 (water treatment plant reservoir), 27,300 m3 (Zone 1 reservoir) and 9,000 m3 (Zone 2 reservoir). The six municipal wells obtain water from the Jacobsville Formation in the Central Basin and the overlying sand and gravel unit in the East Basin. There are two wells at the Goulais Well Site and one well at the Steelton Well Site located in the Central Basin. In the East Basin, there are two wells at the Lorna Well Site and one well at the Shannon Well Site.

All of the groundwater that is pumped from the municipal wells is treated prior to introduction into the water distribution system, and the raw surface water is directed to the water treatment plant and reservoir prior to distribution to the City of SSM. The Zone 1 and Zone 2 reservoirs float on the distribution system in their respective pressure zones. The system was designed so that all sources are interconnected to the same system for redundancy, and, therefore offers a high level of tolerance. Permitted rates exceed any historical takings, and projected takings are lower than historical demands.

Operational issues, however, are an ongoing challenge for the Public Utilities Commission (PUC), who needs to manage a mixed water supply system and aging infrastructure. To improve the tolerance for the Sault Ste. Marie water supply system, consideration should be given to upgrading the existing infrastructure.

10.4 UNCERTAINTY ASSESSMENT

Based on the reported calibration results and the sensitivity analysis of the groundwater flow model, discussed in Section 3.4 of the Conceptual and Numerical Model Development Report (Appendix A), it can be stated that predictions made by the model

produced consistent model results. The model has been calibrated for multiple stress conditions and, when applied to assess the potential changes between scenarios, there is high confidence in its reliability.

In contrast, the limitations in the available data mean that calculations of absolute water levels in any particular simulation are likely to be less reliable. The extent of the Local Area is based on the delineation of the WHPA-Q1, which in turn is sensitive to the projected drawdowns with respect to conditions of no pumping for which little data are available.

Through review of the model calibration trends, the simulated water level drawdowns tend to be higher than observed drawdowns. This would result in an over-estimate of the WHPA-Q1 area which ensures that the actual WHPA-Q1 falls within the simulated boundaries. Therefore, the uncertainty associated with the Local Area is Low.

11.0 WATER QUANTITY THREATS

As outlined in the Technical Rules, a drinking water quantity threat is any activity that reduces groundwater recharge to an aquifer or any consumptive water taking. Consumptive water takings are activities that extract water from an aquifer or surface water body without returning that water to the same aquifer or surface water body.

11.1 CONSUMPTIVE WATER DEMANDS

Within each vulnerable area (i.e., WHPA-Q1 and WHPA-Q2) identified under clause 15 (2) (d) or (e) of the *Clean Water Act, 2006*, drinking water quantity threats must be identified. Considering the low rates of non-municipal permitted groundwater takings, and relatively low demand from domestic well users, no water quantity threats were identified.

11.2 REDUCTIONS IN RECHARGE

The Technical Rules specify that reductions in groundwater recharge represent water quantity threats. Land use changes projected for the SSMR SPA are limited, and the proposed developments are not anticipated to significantly affect the groundwater recharge areas. As such, there is low potential for a water quantity threat in terms of reduction in recharge.

12.0 SIGNIFICANT GROUNDWATER RECHARGE AREAS

The Technical Rules require that Significant Groundwater recharge Areas (SGRAs) be delineated for each source protection area. SGRAs are one of four types of vulnerable areas that are used in water quality vulnerability assessments; the other vulnerable areas are wellhead protection areas, intake protection zones, and highly vulnerable aquifers.

12.1 METHODOLOGY USED TO DELINEATE SGRAS

The Technical Rules (MOE, 2009) require that SGRAs be delineated by identifying the portion of the study area where groundwater recharges at a rate of 1.15 times greater than the average annual groundwater recharge for the area (Rule 44(1); MOE, 2009). This methodology was used to delineate the SGRAs in this Tier Three Assessment.

12.2 SIGNIFICANT GROUNDWATER RECHARGE AREA DELINEATION RESULTS

12.2.1 TIER ONE AND TIER TWO ASSESSMENT SGRAS

Recharge Areas have been delineated and are presented in the Sault Ste. Marie Area Groundwater Management & Protection Study (R.J. Burnside & Associates, 2003). The recharge areas are based on large sand and gravel deposits identified in quaternary soils maps that are located between the Precambrian uplands and the lowlands (refer to Appendix A, Map 4-15 of the Burnside report). According to Appendix B of Guidance Module 7, these areas are classified as a “High Volume Recharge Area.”

Taking into consideration the location of the sand and gravel deposits and their significance as the only major formation to allow recharge to the deep aquifer unit to the East and Central Basins, which are the primary groundwater resources for the community, these units have been considered Significant Recharge Areas in the SSMR SPA.

12.2.2 TIER THREE ASSESSMENT SGRAS

In the refined groundwater model for the Tier Three assessment, recharge is assigned based on hydrologic response units (HRUs) delineated during the surface water modelling. These HRUs represent areas with similar infiltration characteristics. The HRUs were delineated by considering multiple factors, including: land use, land cover, soil type (quaternary geology) and areas with high potential groundwater recharge. Recharge rates based on these HRUs have been specified directly in the groundwater model. Recharge varies from 0 mm/yr over impervious areas to 2,100 mm/yr over the high potential groundwater recharge area (HPGRA) adjacent to the Precambrian granite uplands on the

northern edge of the model under average climate conditions. Recharge values obtained from the surface water model were adjusted during groundwater model calibration (refer to Map 3-5 of the Conceptual and Numerical Model Development Report (Appendix A)).

An average annual recharge rate was calculated based on the distribution of recharge rates over the study area (that is, the limits of the groundwater flow model) of seven different categories based on the HRUs. The seven categories which were evaluated, with their associated HRUs and recharge rates are listed below:

- Waterbody/Impervious Area (HRUs 1 and 2; 0 mm/year);
- Wetland (HRU 3; 40 mm/year);
- Fine Grain Soil/Granite Bedrock (HRU 4; 65 mm/year);
- Developed Area (HRU 5; 125 mm/year);
- Medium/Coarse Grain Soil – not on HPGRA (HRUs 6 and 8; 350 mm/year);
- Medium/Coarse Grain Soil – on HPGRA (HRUs 7 and 9; 425 mm/year); and
- HPGRA Receiving Overland Flow (2,100 mm/year).

Here, HPGRA refers to the “High Potential Groundwater Recharge Area” immediately adjacent to the Precambrian Uplands (defined as the "protected recharge area" in Burnside, 2003).

To calculate the average annual groundwater recharge rate for the study area (groundwater model domain) the following approach was followed:

1. The surface area for each category inside the model domain was calculated.
2. The surface area was divided by the total surface area of the model area.
3. The resulting percentage for each category was multiplied with the associated recharge rate to obtain weighted recharge rates.
4. The average annual groundwater recharge rate was obtained by computing the total of the seven weighted recharge rates.

The average annual groundwater recharge rate was found to be 370 mm/year. Table 12.1 summarizes these results.

Table 12.1 Calculation of Average Annual Groundwater Recharge Rate

HRU Category	HRU #	Recharge rate (mm/year)	Area (km^2)	Area (% of total)	Weighted Recharge Rate (mm/year)
Waterbody/Impervious Area	1; 2	0	0.36	0.3	0
Wetland	3	40	0.41	0.3	0.1

Table 12.1 Calculation of Average Annual Groundwater Recharge Rate

HRU Category	HRU #	Recharge rate (mm/year)	Area (km^2)	Area (% of total)	Weighted Recharge Rate (mm/year)
Fine Grain Soil/Granite Bedrock	4	65	21.51	16.3	10.6
Developed Area	5	125	51.91	39.3	49.1
Medium/Coarse Grain Soil – not on HPGRA	6; 8	350	28.99	21.9	76.7
Medium/Coarse Grain Soil – on HPGRA	7; 9	425	18.11	13.7	58.2
HPGRA Receiving Overland Flow		2,100	10.88	8.2	172.2
Total			**132.17**	**100**	**366.9**
1.15 times Average Annual Groundwater Recharge Rate:					**421.9**

To delineate the SGRAs, the portion of the study area where groundwater recharges at a rate of 1.15 times greater than the average annual groundwater recharge for the area was determined. Based on the recharge polygons applied in the MODFLOW groundwater flow model, all recharge values greater than 420 mm/year were used to delineate the SGRAs for the Tier Three Assessment. The recharge polygons were used in this estimation to maintain consistency with the input parameters used in the Local Area Risk Assessment. Two of the seven HRU categories shown in the above table have average recharge rates greater than 420 mm/year and were, therefore, used to define the extent of the SGRAs:

- Medium/Coarse Grain Soil – on HPGRA (HRUs 7 and 9; 425 mm/year); and
- HPGRA Receiving Overland Flow (2,100 mm/year).

Map 7-1 illustrates the SGRAs based on the Tier Three assessment. The major SGRAs are at the bedrock/overburden contact along the southern border of the Precambrian uplands to the north of the City, with an area of approximately 3750 ha. This larger zone is associated with the gravel-rich glaciolacustrine beaches deposited adjacent to the uplands.

13.0 SUMMARY AND CONCLUSIONS

Given the potential for moderate stress on the groundwater system for the Central and East Basins of the SSMR SPA, this Tier Three Assessment was undertaken to assess the likelihood that the SSMR SPA will be able to sustain its water demand and to identify threats to the drinking water supply that may influence its ability to meet its allocated pumping rates.

As part of the study, numerical groundwater and surface water models were developed in accordance with the requirements for the Technical Rules based on the conceptual understanding of the groundwater and surface water systems of the SSMR SPA. The models were integrated to delineate the “Local Areas” for the groundwater wells which form the basis for the Local Area Risk Assessment. The surface water model was constructed for the Root River Subwatershed using the Guelph All-Weather Storm-Event Runoff model to simulate water budget components in a spatially detailed and temporally dynamic manner using hourly time steps. The groundwater model is based on a MODFLOW model developed initially by Waterloo Numerical Modelling Corporation as part of the 2003 Burnside Groundwater Study.

A set of eight (8) risk assessment scenarios (Scenarios C, G(1), G(2), G(3), D, H(1), H(2) and H(3)) were developed to consider the impact of increases in water demand, drought conditions, and land use change on the sustainability of the municipal water supply. The scenarios were simulated using the Tier Three integrated groundwater and surface water models. Scenario C represents existing municipal pumping, existing land use and average climate conditions. Scenario G(1) assessed the ability of the municipal wells to meet their allocated demand (existing plus committed) under conditions of future land use and average climate, Scenario G(2) assessed the ability of the municipal wells to meet their allocated demand (existing plus committed) under conditions of existing land use and average climate and Scenario G(3) assessed the ability of the municipal wells to meet their existing demand under conditions of future land use and average climate. Scenario D evaluated the ability of the municipal wells to pump at their respective existing rates during a drought period. Scenario H(1) assessed the ability of the municipal wells to meet their allocated demand under future land use and drought conditions, Scenario H(2) assessed the ability of the municipal wells to meet their allocated demand under existing land use and drought conditions and Scenario H(3) assessed the ability of the municipal wells to meet their existing demand under future land use and drought conditions.

In order to assess the ability of each municipal well to meet its allocated pumping rate under various climate, water demand and land use conditions, the maximum model-simulated additional drawdown at each well for each risk assessment scenario was compared to the average safe additional available drawdown for the well, which was established as part of the Conceptual Understanding. For each of Scenarios G(1), G(2),

G(3), D, H(1), H(2) and H(3), the average safe additional available drawdown was found to exceed the maximum model-simulated additional drawdown for all six municipal wells. Therefore, the Local Area defined for the SSMR SPA, which includes the cumulative drawdown for the Steelton Well and the Goulais Wells and that for the Shannon Well and the Lorna Wells, was assigned a Low risk level.

In terms of the uncertainty associated with the model results, it can be stated that, based on the reported calibration results and the sensitivity analysis of the groundwater flow model, predictions made by the model produced consistent results and that there is high confidence in the model reliability when assessing the potential changes between scenarios. However, the limitations in the available data mean that calculations of absolute water levels in any particular simulation are likely to be less reliable. Model calibration trends show that the simulated water level drawdowns tend to be higher than observed drawdowns which would result in an over-estimate of the WHPA-Q1 area. Since this ensures that the actual WHPA-Q1 falls within the simulated boundaries, the uncertainty associated with the Local Area was determined to be Low.

As the municipal supply system integrates the groundwater sources and surface water sources into the distribution system, the tolerance level for the water supply system is high overall; but needs to manage challenges from a mixed system and aging infrastructure. To improve the tolerance of the system, it is necessary to consider upgrades to the existing infrastructure.

The Water Quantity Threats within the Local Area are limited given that most water takings are by domestic well users, and that non-municipal groundwater permits are at relatively low rates and are insignificant in comparison to the municipal takings.

Two conservation areas and only a very small portion of the wetland areas located in the SSMR SPA lie within the groundwater model limits, with only the Fort Creek Conservation Area and very few wetland areas being located within the WHPA-Q1. Given that increased pumping simulated in Scenarios G(1) and G(2) only affects the hydraulic head distribution in the deeper aquifer units to a large extent, significant adverse impacts on the shallow aquifer system connected to the conservation areas and the wetland areas are not anticipated.

The Significant Groundwater Recharge Areas were delineated and correspond with the sand and gravel outcrop to the south of the Precambrian uplands. A portion of the Significant Groundwater Recharge Areas falls within the Local Area and can pose a threat to the water quantity if significant changes in recharge to that area are proposed. At this time, there are no major development plans which are expected to significantly affect this aspect.

14.0 RECOMMENDATIONS

Based on the findings of the Tier Three Water Budget and Local Area Risk Assessment, the following recommendations are made:

- Long-term monitoring of water levels at the CEG/Kresin monitoring well to improve data quality and certainty.
- The Provincial Groundwater Monitoring Network (PGMN) should be upgraded to include wells within the Sault Ste. Marie Source Protection Area closer to the Significant Groundwater Recharge Areas but sufficiently far from the municipal wells to allow for the measurement of background water levels.
- Long-term monitoring of water levels at the municipal wells.
- To improve the tolerance of the groundwater system, consider potential upgrades to the aging infrastructure.

15.0 LOCAL AREA RISK ASSESSMENT REPORT REFERENCES

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