PRELIMINARY SERVICING REPORT
FOR
PROPOSED RETREAT CENTRE
PART OF LOT 13, CONCESSION 2, E.G.R.
(FORMER TOWNSHIP OF GLENELG)
MUNICIPALITY OF WEST GREY

GAMSBY AND MANNEROW LIMITED
CONSULTING PROFESSIONAL ENGINEERS
GUELPH – KITCHENER - LISTOWEL - OWEN SOUND

Gamsby and Mannerow
ENGINEERS

November 2009
Our File: C-7274

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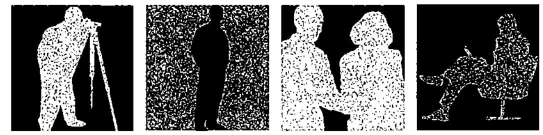

PRELIMINARY SERVICING AND ACCESS STUDY
FOR
PROPOSED RETREAT CENTRE
PART OF LOT 13, CONCESSION 2, E.G.R.
(FORMER TOWNSHIP OF GLENELG)
MUNICIPALITY OF WEST GREY
November 2009
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1.0 INTRODUCTION

A Retreat Centre is proposed for Part Lot 13, Concession 2, E.G.R. (Former Township of Glenelg) in the Municipality of West Grey. The location of the proposed centre is shown on Figure 1. It is proposed to construct a 4,500 m^2 Retreat Centre and Dormitory with associated driving and parking areas. An on-site sewage treatment system, a drilled well and a stormwater management pond are proposed to service the site.

The West Grey Zoning of this land is currently rural and natural environment/hazard lands. The proposed development is only located on lands classified as rural.

The purpose of this report is to undertake a desk-top review of available information, evaluate the feasibility of the proposal with respect to sewage disposal, water supply and stormwater management, and document a conceptual level of design for these systems, to support Planning application for an Official Plan Amendment and Zoning By-law Amendment.

2.0 DOMESTIC WATER SUPPLY – HYDROGEOLOGICAL INVESTIGATION

2.1 PURPOSE AND SCOPE

The purpose of this study is to assess the Hydrogeological information available in the vicinity of the subject site, to characterize the availability of an adequate supply of water required to service the Retreat Centre and Dormitory, and to confirm that the local aquifer can supply sufficient potable water for domestic use without negative impacts to existing features or adjacent properties.

The scope of the investigation includes the following:

1) Review of available reports and Hydrogeological information, including MOE well logs, in the vicinity of the site;
2) Impact assessment based on the existing conditions in the aquifer(s) and the existing water use in the area.

2.2 METHODOLOGY

2.2.1 Background Data Collection/Review

Available information, including MOE well records, MOE publications, geologic publications, previous studies in the area and pertinent maps, was collected and reviewed. The locations of the wells from MOE records, used to obtain lithological cross-sections of the study area, are shown on Figure 2. This information was used in combination with site-specific information for the hydrogeologic assessment.

2.3 RESULTS OF INVESTIGATION

2.3.1 Regional Geology

The subject property is located in the Horseshoe Moraines Region according to "The Physiography of Southern Ontario" (Chapman and Putnam, 1994). The former Township of Glenelg is covered by a complex of till ridges, kame-moraines, outwash plains and spillways, interspersed with more smoothly moulded till plains and drumlinized areas (Chapman and Putnam, 1984). The geology consists of highly variable overburden ranging from sands and gravels to clay tills.

The underlying bedrock in the area is part of the Guelph formation, which consists of buff to brown dolostones (Chapman and Putnam, 1984). Based on the MOE well log data, the depth to bedrock in the area varies from 8 to 25 m bgs.

2.3.2 Regional Hydrogeology

Based on MOE well records, the depth to the water-bearing zone in the bedrock within the region is between 15 and 60 metres bgs (MOE Map 3111-8). Most wells in the area utilize the dolostone bedrock for water supply. The MOE considers the groundwater in this region to be adequate for domestic and livestock purposes (MOE Map 3111-1), with expected well yields to be in the range of 3 to 77 l/min. Static water levels in water bearing units of area wells are reported between 0.6 and 10.6 m bgs.

The MOE has records of only a few wells in the overburden units in the area. Given the complexity and variation of the overburden, well yield is expected to be highly variable in the overburden and, although the aquifer is expected to be able to provide adequate water supply, an on-site pumping test is recommended to verify the quantity and quality of supply in advance of the Site Plan application.

The topography of the site in the location of the proposed development generally slopes downward westerly toward a lowland area dividing the subject property. Two wetland areas exist on the site. Both are located to the west of the development area and are connected to a larger wetland system approximately 500 – 1000 m south of the site. Based on the regional surface drainage patterns, the wetland areas appear to be controlled by the surface water and shallow groundwater systems. The regional drainage is in a westerly direction, where it connects with the Styx River system, approximately 1 km away.

2.3.3 Previous Studies in the Area

In August of 2005 a Servicing and Access Study was prepared by Gamsby and Mannerow Ltd. for a proposed subdivision in the area (S-1353 Servicing and Access Study, Proposed Plan of Subdivision, Part of Lot 6, Concession 1, E.G.R., August 2005). A pumping test was conducted at 91 litres (20 imperial gallons) per minute for a total of one hour. The recharge was monitored for 20 minutes after the test and was measured to be 0.745 mbgs. The report concluded that the potential well yield is likely greater than 100 l/min for the bedrock aquifer in the area.

3.0 WATER DEMAND, STORAGE, SUPPLY, TREATMENT AND DISTRIBUTION

3.1 WATER DEMAND

Calculations for ultimate water demand for the Retreat Centre are based on the Ministry of the Environment Guidelines for Design of Small Residential Developments.

• Number of Persons	=	500 persons
• Average Day Demand per Capita	=	190 L/capita/day
Total Average Day Demand	=	95,000 L/day (20,897 IGPD)
	=	66.0 L/min (14.5 IGPM)
• Maximum Day Demand Peaking Factor	=	3.0
• Maximum Day Demand	=	66.0 L/min x 3.0
	=	198.0 L/min (43.55 IGPM)
• Peak Hour Demand Peaking Factor	=	6.0
• Peak Hour Demand	=	66.0 L/min x 6.0
	=	396.0 L/min (87.1 IGPM)

These flows do not include water for outdoor uses such as lawn watering, car or trailer washing, etc. If lawn watering were to be provided, and assuming that lawn watering is restricted to off-peak water demand times, lawn watering flow should be included in the Average Day Demand to determine the Total Peak Pumping Rate.

3.2 FIRE PROTECTION

It is not proposed to provide water for fire protection due to the close proximity to the surface water east of the proposed building. Confirmation from the Fire Department would be required.

3.3 WATER SUPPLY, TREATMENT AND DISTRIBUTION - OPTION ONE

3.3.1 Water Supply

As described in Section 2.3.3, there is the possibility that the potential well yield for the bedrock aquifer could be greater than 100 l/min. A pumping test is required to determine the site specific potential well yield.

Option One assumes that the bedrock aquifer would be able to provide the Peak Hour Demand of 396 l/min. Two (2) 100 mm (4") wells would be installed on-site to supply raw water. One (1) pump would satisfy up to the Maximum Day Demand (198.0 L/min), while two (2) pumps would satisfy up to the Peak Hour Demand (396.0 L/min). The lead pump would alternate after each cycle.

3.3.2 Water Treatment

Water treatment would be provided by two (2) Trojan UV Max J+ UV filters, or equivalent, which would provide treatment at a rate of 220 L/min with a dose of 30 mJ/cm^2.

3.3.3 Pressure Tanks

The number of 450 L pre-charged pressure tanks required to ensure a minimum cycle time of 10 minutes is calculated as follows:

Drawdown Available for Pressure Tanks pre-charged for 40-60 psi operating range is 122.0 L.

At a demand rate of 198.0 l/min (Maximum Day Demand), one tank will discharge in:

$$\frac{122.0\ \text{L}}{198.0\ \text{L/min}} = 0.62\ \text{min}$$

From the proposed pumping curve, the average pumping rate over the cycle is 315 L/min.

Therefore, one tank will fill in $\frac{122.0\ \text{L}}{315.0 - 198.0\ \text{L/min}} = 1.04\ \text{min}$

To ensure a 10 minute minimum cycle time, the number of 451 L pre-charged pressure tanks (n) is determined from

$$(n \times 0.62) + (n \times 1.04) = 10 \text{ minutes}$$

$$\text{Or } n = 6$$

Therefore six (6) 451 L pre-charged pressure tanks would be required.

3.3.4 Overview of Option One

Option One consists of two (2) wells and low lift pumps to provide raw water, two (2) UV treatment units to provide water quality and six (6) pressure tanks to provide sufficient pressure throughout the distribution system.

3.4 WATER SUPPLY, TREATMENT AND DISTRIBUTION - OPTION TWO

3.4.1 Water Supply

Option Two assumes that the bedrock aquifer would not be able to provide the Peak Hour Demand of 396 l/min. Therefore, it would be necessary for water to be stored on-site.

Two (2) 100 mm (4") wells would be installed on-site to supply raw water. The lead pump would alternate after each cycle.

3.4.2 Water Treatment

Water treatment would be provided by one Trojan UV Max J+ UV filter and would provide treatment at a rate of 220 L/min with a dose of 30 mJ/cm^2.

3.4.3 Water Storage

MOE guidelines state that the useable volume of storage should be 25% of the Maximum Day Demand plus 40% of the Average Day Demand (without storage for fire protection).

Therefore, the required storage volume is calculated as follows:

$$\begin{aligned} V_s &= (0.25 \text{ x } 198.0 \text{ L/min x } 60 \text{ min/hr x } 24 \text{ hr}) + (0.4 \text{ x } 66.0 \text{ L/min x } 60 \text{ min/hr x } 24 \text{ hr}) \\ &= 71{,}280 \text{ L} + 38{,}016 \text{ L} = 109{,}296 \text{ L} \\ &= 110 \text{ m}^3 \end{aligned}$$

On-site underground holding tanks would provide 110,000 L of storage for treated water on-site to meet the Maximum Day Demand.

3.4.4 High Lift Pumps

Two (2) high lift pumps would provide flow to the distribution system. One (1) pump would satisfy up to the Maximum Day Demand while two (2) pumps would satisfy up to the Peak Hour Demand. The lead pump would alternate after each cycle.

3.4.5 Pressure Tanks

As calculated in Section 4.2.2, six (6) 451 L pre-charged pressure tanks would be required to ensure a minimum cycle time of 10 minutes.

3.4.6 Overview of Option Two

Option Two consists of two (2) wells and low lift pumps to provide the water supply, one (1) UV treatment unit to provide water quality, storage units to hold 110,000L of treated water, two (2) high lift pumps to provide flow to the distribution system and six (6) pressure tanks to provide sufficient pressure throughout the distribution system.

3.5 SUMMARY

Despite the uncertainty of the site specific potential well yield, Options One and Two are both feasible alternatives to provide a water supply for the proposed development. An on-site pumping test will confirm the available quantity and quality of water from the aquifer.

Since the proposed water supply is greater than 50,000 L/day, the Ministry of the Environment will require a Certificate of Approval for the system, and a Permit to take water. Application for these would follow an on-site pump test at the time of the Site Plan application.

4.0 ON-SITE SEWAGE DISPOSAL SYSTEMS

4.1 SOIL CHARACTERISTICS

There are three types of soil on the site, according to the Grey County Soil Survey (Report No. 17, J.E. Gillespie and N.R. Richards, 1954). Harriston Silt Loam, Parkhill Loam and Wiarton Silt Loam are all present on-site, however, only Harrison Silt Loam and Parkhill Loam are located in the area proposed to be developed. Both soils are described in the Grey County Soil Survey as smooth to gently sloping, moderately stony with poor to good drainage. Both are BC soil types with a minimum infiltration rate of 9 mm/hr and maximum infiltration rate of 162.5 mm/hr according to the MTO Drainage Management Manual.

4.2 DESIGN OF SEWAGE TREATMENT SYSTEM COMPONENTS

4.2.1 Design Sewage Volumes

The OBC indicates that the daily design sanitary sewage flow for a Boarding House, with meal and laundry is 200 L/person/day. With a maximum of 500 people attending the Retreat Centre, the daily sewage flow is estimated at 100,000 L/day.

Since the proposed sewage system would be designed for greater than 10,000 L/day, the Ministry of the Environment (MOE) will require a Certificate of Approval for the system. During the final sewage system design, on-site test pitting and soils sampling and testing will be necessary.

Based on the proximity of the site to wetland area, we expect the MOE would require comment with respect to the potential nitrate loading. Consequently, the MOE's calculation of aquifer security is provided, but further investigation will be necessary in the Hydrogeological Study prior to MOE applications.

4.2.2 Septic Tanks

The OBC states that the working capacity of the septic tank shall be twice (2) the daily design sewage flow for residential uses and three (3) times the flow for commercial establishments. The facility is interpreted to be a residential use for the purpose of sewage generation.

Therefore, based on a design daily flow of 100,000 L/day, it is proposed to provide six (6) – 34,000 litre septic tanks for a total capacity of 204,000 litres.

4.2.3 Holding Tanks

Two (2) 50,000 L holding tanks are proposed to be located downstream of the septic tanks. Each holding tank would contain two (2) alternating submersible effluent pumps. The effluent pumps would be timed to evenly dose six (6) Waterloo Biofilter Wire Mesh Baskets at regular intervals. The holding tanks would provide storage for sewage during times of high flows when the sewage volume being generated is greater than the dosing capacity of the biofilter system.

4.2.4 Treatment Unit

Due to the large volume of wastewater, a Waterloo Biofilter treatment system is proposed to treat the sewage to tertiary effluent quality. There are several types of sewage treatment systems that could be used to provide the same level of treatment. At this stage of conceptual design, it is proposed to have a completely in-ground system. Therefore, the system proposed at this time is the Waterloo Biofilter Wire Mesh Basket System.

It is proposed to provide treatment with twelve (12) 96" x 96" Waterloo Biofilter Wire Mesh Baskets with a total of 133.3 m^3 of Biofilter medium and spray manifolds. There would be two (2) Wire Mesh Baskets contained in a 40,000 L Concrete Treatment Tank. Therefore six (6) - 40,000 L concrete treatment tanks would be required.

4.2.5 Area Beds

The area beds would be located in the easterly field, as shown on the Site Grading and Drainage Plan. The area beds would be partially raised above existing grade. The existing fill and topsoil would be stripped from the area bed location. With the added secondary treatment capabilities of the Waterloo Biofilter, a Waterloo Biofilter Area Bed can be designed based on the guidelines of the Building Materials Evaluation Commission (BMEC) Authorization.

Based on the BMEC Authorization, the area bed consists of a 250 mm deep stone layer overlying a 250 mm deep sand layer.

The minimum area of stone required is:

$$A_{Stone} = \frac{Q}{75} \quad \text{(based on Q< 3000 L/d)}$$

$$A_{Stone} = \frac{100{,}000 \text{ litres/day}}{75 \text{ litres/m}^2\text{/day}} = 1{,}333 \text{ m}^2$$

As shown in the equation below, the minimum area of sand required depends on the "T"-time of the native soils. At this time no invasive testing, such as testpitting, has been conducted to determine the percolation rate of the soils on-site. Therefore, assuming that the soils have a percolation rate of 50 min/cm, the minimum area of sand required would then be:

$$A_{Sand} = \frac{Q * T}{850}$$

$$A_{Stone} = \frac{100{,}000 \text{ litres/day} * 50 \text{ min/cm}}{850} = 5{,}882 \text{ m}^2$$

It is proposed to construct four (4) area beds, each with a 12 m x 30 m stone layer and a 30 m x 50 m sand layer.

4.2.5 Mantle Area

A minimum mantle of 15 metres would be required in the direction of flow from the edge of the piping within the stone layer. A 20 metre mantle from the edge of the stone layer to the edge of the sand layer is included for each area bed.

4.2.6 Dosing System

The filter bed would be dosed based on a preset time to correspond with the operation of the biofilter. A duplex system would be required.

The forcemains from the holding tanks to the biofilter and from the biofilter to the area beds are to be buried 1.5 m below grade. In areas where this cannot be achieved, appropriate insulation must be provided to prevent freezing.

4.2.6 Alarms and Controls

The system would be installed with appropriate duplex controls and alarms located inside the building as designated by the Owner.

4.2.6 Review of Overall Sewage System Conceptual Design

A total of four (4) – 50,000 litre septic tanks, two (2) - 50, 000 litre holding tanks, six (6) - 40,000 litre concrete tanks containing a total of twelve (12) Waterloo Biofilter Wire Mesh Baskets and four (4) - 30 metre wide x 50 metre long area beds are proposed, which would provide capacity for the Retreat Centre to accommodate 500 visitors.

Since the proposed sewage system would be designed for greater than 10,000 L/day, the Ministry of the Environment will require a Certificate of Approval for the system. For final sewage system design, on-site test pitting and soils sampling and testing will be necessary to confirm design parameters.

4.3 REQUIRED MINIMUM CLEARANCES

The OBC requires minimum clearance for treatment units i.e. septic tanks, from built or natural features as follows:

Structure..1.5 m
Well..15 m
Lake..15 m
Property Line...3 m

The minimum clearance required by the OBC for distribution pipes are as follows:

Structure...5 m
Well with a watertight casing to a depth of 6 m......15 m
Any other well...30 m
Lake..15 m
Property Line..3 m

There should be adequate area to locate the leaching bed at these required separation distances.

4.4 AQUIFER SECURITY

The following provides a calculation of the possible nitrate concentration in the groundwater resulting from the development of the Retreat Centre. This is a conservative estimate because only dilution from the infiltration of precipitation is considered. The possible dilution from the groundwater flux or nitrate attenuation is not considered.

Assumptions:

1. On average, the Retreat Centre would discharge 100,000 litres/day of sewage effluent containing 40 mg/L nitrate as NO_3. With a tertiary treatment system designed and installed, the nitrate level can generally be reduced by approximately 50%. This depends on the type of treatment system installed. The system that is proposed is expected to provide a 50% reduction in nitrates after treatment.

2. NO_3 in precipitation: 0.01 mg/L

3. Area of development = 12.60 hectares ~ 126,000 m^2

Calculation of Daily Infiltration Rate of Precipitation

Mean annual precipitation = 40" = 1.02 m

Mean annual actual evapotranspiration = 20" = 0.51 m

Difference = 1.02 m – 0.56 m = 0.51 m

Site Area = 12.6 hectares ~ 126,000 m^2

Runoff coefficient – worst case is sand over gravel, >10% slopes, mown grass~0.40

Runoff = 0.51 m x 0.40 = 0.204 m

Infiltration = 0.51 m - 0.204 m = 0.306 m

Daily rate of infiltration = 0.306 m / 365 days = 8.38 x 10^{-4} m/day

Calculation of Nitrite (NO_3) Concentration in Groundwater:

Nitrate from Daily Sewage Volume (A) [mass / day]	+	Nitrate from Daily Infiltration (B) [mass / day]	=	Total Nitrate Loading (C) [total mass/total volume = concentration]

(A) = 100,000 L/day x 20 mg/L = 2,000,000 mg/day

(B) = 126,000 m^2 x 8.38 x 10^{-4} m/day x 10^3 L/m^3 x 0.01 mg/L = 1,055.9 mg/day

(C) = [100,000 L/day + (126,000 m^2 x 8.38 x 10^{-4} m/day x 10^3 L/m^3)] NO_3
= [100,000 L/day + 105,588 L/day] NO_3
= [205,588 L/day] NO_3

Solving the equation for NO_3:

$$NO_3 = \frac{2{,}000{,}000 \text{ mg/day} + 1{,}055.9 \text{ mg/day}}{205{,}588 \text{ L/day}} = 9.73 \text{ mg/L}$$

This analysis indicates that the resultant nitrate loading in the shallow groundwater for the Retreat Centre would not exceed the Ministry of the Environment Drinking Water Objective limit of 10 mg/L.

5.0 STORMWATER MANAGEMENT

5.1 STORMWATER MANAGEMENT CRITERIA

The stormwater management parameters for the site are expected to be as follows:

1. The stormwater release rate from the site following the proposed development will not exceed the pre-development rates during the various design storm events.

2. All stormwater flows will be directed to the wetlands to the east and west of the site.

3. The development area is 10.20 hectares, parking lot and driveway area is 1.1 hectares and the total area of all buildings is 0.49 hectares.

The MIDUSS software was used to model design rainfall events based on the following Chicago storm input parameters, for the Owen Sound area.

Table 1 – Design Rainfall Events

Coefficient	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
A	861.806	1184.578	142.992	1681.406	1884.680	2085.149
B	7.833	7.833	7.822	7.822	7.833	7.833
C	0.832	0.844	0.850	0.855	0.858	0.860
R	0.375	0.375	0.375	0.375	0.375	0.375
Duration (min)	360	360	360	360	360	360
P_{Total} (mm)	37.926	48.563	55.516	64.594	71.131	77.773

5.2 ANALYSIS RESULTS

Table 2 – Runoff Calculations Summary

	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
Pre-Development (m^3/s)	0.071	0.155	0.226	0.335	0.433	0.541
Post-Development (m^3/s)	0.222	0.315	0.384	0.484	0.565	0.653
Required Storage (m^3)	300	310	335	360	367	328

Based on the results from Table 2, to meet the pre-development discharge rate, a maximum of 367 m^3 of storage is required. This volume of stormwater storage can be achieved through a stormwater management pond, subsurface stone storage gallery, rooftop or parking lot ponding, or a combination of the above.

Given the available land area, a 40 metre long x 15 metre wide x 1 metre deep stormwater management pond is proposed. The pond would attenuate the stormwater runoff rate to pre-development levels while continuing to discharge to the wetlands east and west of the site.

5.3 WATER QUALITY

All runoff from the site would be routed over grassed surfaces prior to discharge from the site. Catchbasins located in the parking lot would be designed with goss traps, to provide initial water quality treatment for runoff from the parking area, and sumps of 0.60 metres to provide storage for sediment.

During construction, strawbale check dams would be required to filter sediment from runoff water within any swales and geotextile silt fence would be provided along the site boundaries, at downgradient locations, and around stockpiled soils.

6.0 CONCLUSIONS

Based on this "desktop" review of available published information, the servicing of a 500 person Retreat Centre on Part of Lot 13, Concession 2, between Glenelg Sideroads 6 and 3 may be achieved with on-site servicing for water supply, sewage treatment and disposal, and on-site stormwater management. The proposed water supply, sewage disposal and stormwater management should meet applicable guidelines.

Further on-site investigations for water supply (quantity and quality) and sewage treatment and disposal, should be conducted to support the preliminary and detailed design stages for these systems, in advance of Site Plan and Ministry of the Environment approvals applications.

7.0 REFERENCES

The Physiography of Southern Ontario, 3rd Edition
Ontario Geological Survey, Volume 2
L.J. Chapman and D.F. Putnam, 1984
Ministry of Natural Resources (MNR)

Soil Survey of Grey County, 1954
Report No. 17 of the Ontario Soil Survey
J.E. Gillespie and N.R. Richards
Ministry of Agriculture and Food (Agriculture Canada Research Branch)
Guelph, ON

Water Resources Map 3111, 1983
Ground Water Probability
Ministry of the Environment

Respectively Submitted,

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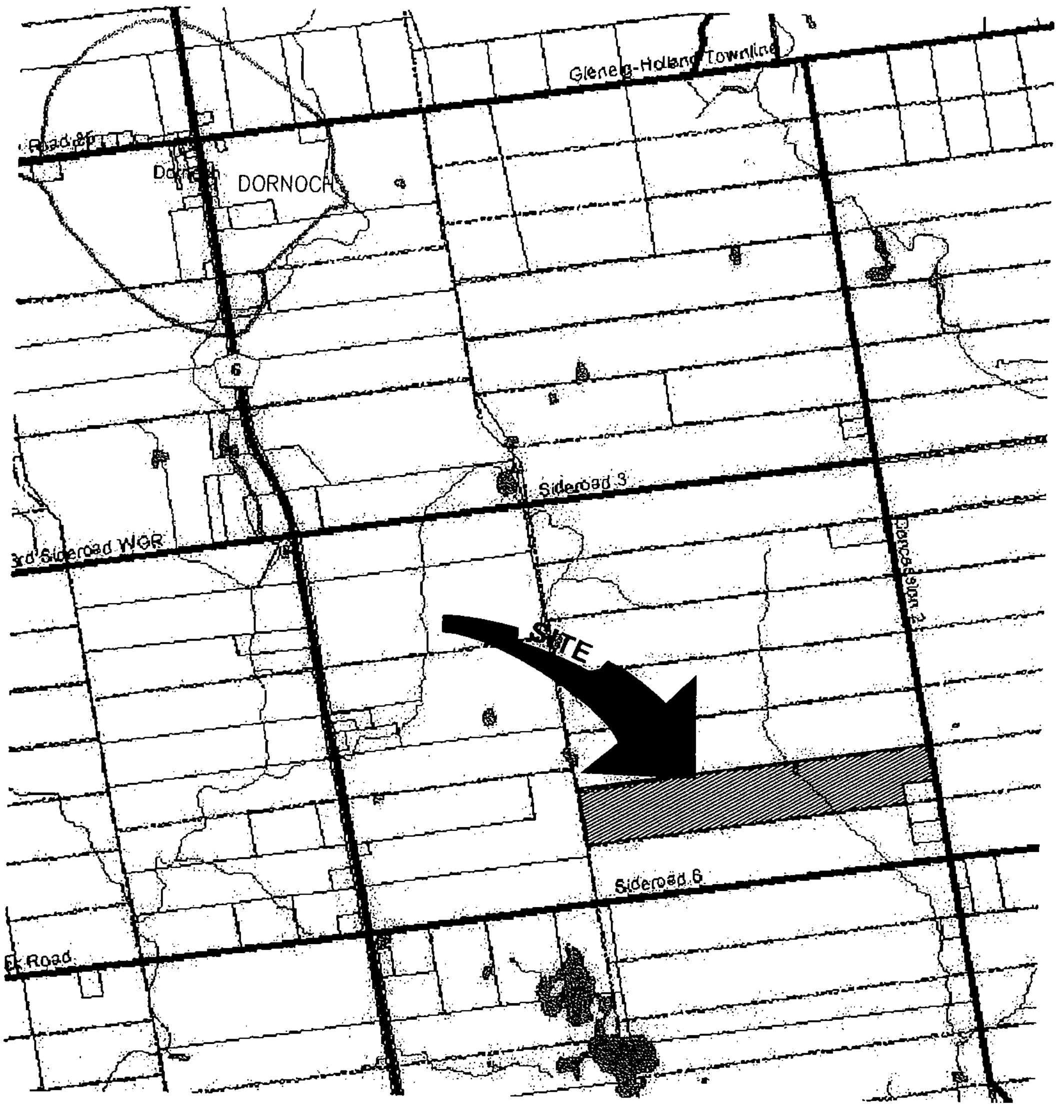
MUNICIPALITY OF WEST GREY

FIGURES

C-7274
Retreat Centre
Municipality of West Grey



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LEGEND

NOT TO SCALE
JUNE 2008

SITE LOCATION MAP

PART LOT 13
CONCESSION 2, E.G.R.
FORMERLY GLENELG TWP.

Figure No. 1

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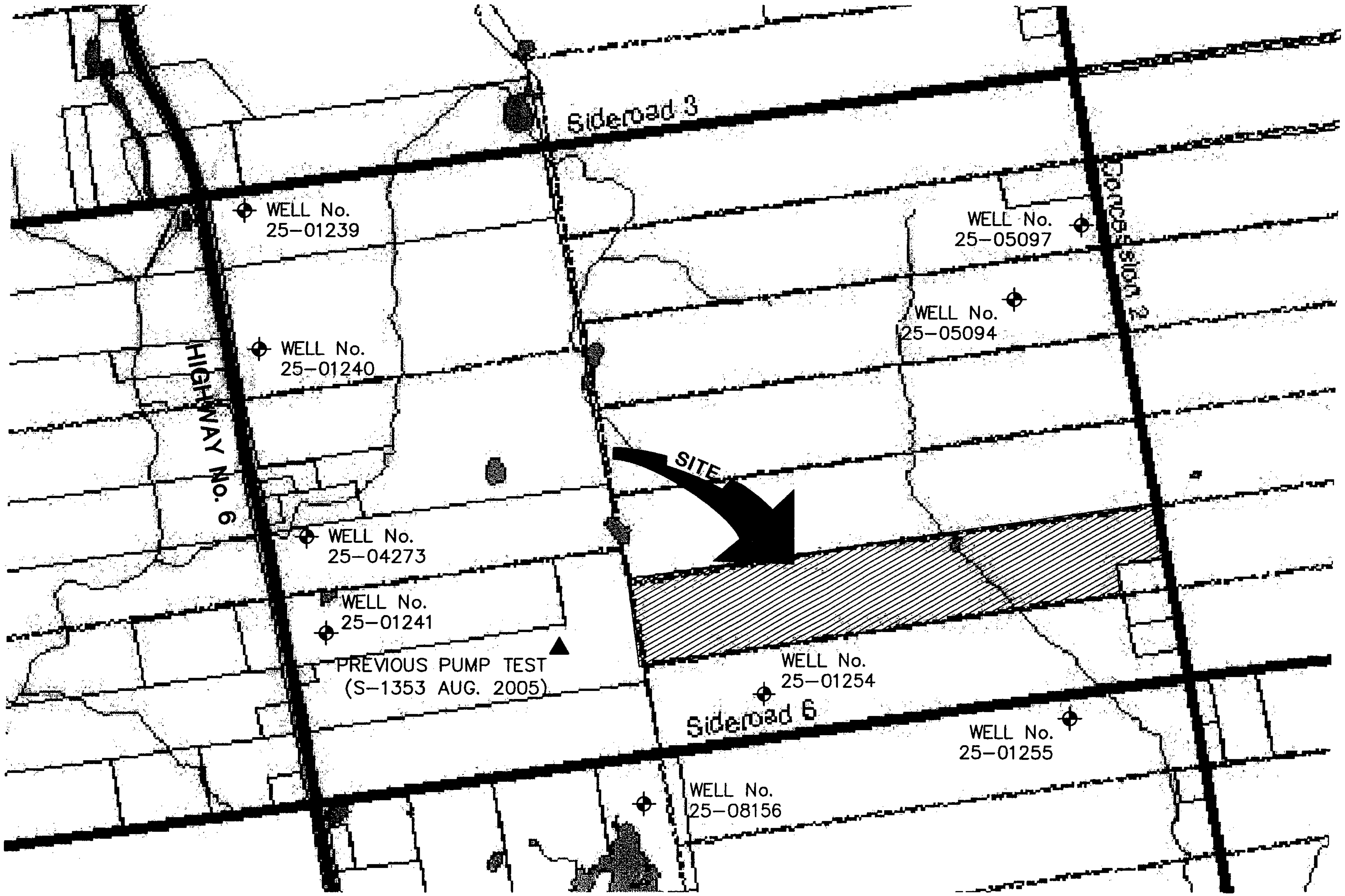

C-7274
Retreat Centre
Municipality of West Grey

LEGEND

EXISTING WELL LOCATION

▲ PREVIOUS PUMP TEST LOCATION

NOT TO SCALE
JUNE 2008

WELL LOCATION MAP

PART LOT 13
CONCESSION 2, E.G.R.
FORMERLY GLENELG TWP.

Figure No. 2

Gamsby and Mannerow
ENGINEERS

Plotted On: JUNE 18, 2008 C7274SPE1.DWG