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Our File: 294

February 13, 2023

Countryview Centre Ltd.
c/o MSR Solutions Inc
#125 – 662 Goldstream Avenue.
Victoria, B.C.
V9B 0N6

Attention: Mike Seymour, P.L. Eng.

Dear Sirs:

Re: Hydrogeological Assessment of Impact of Sewage Effluent Dispersal Basins on Local Aquifer, at Valleyview Shopping Centre, Cobble Hill, B.C.

As requested, I have conducted a hydrogeological assessment of the area on, and around, the Valleyview Shopping Centre located at 1400 Cowichan Bay Road, Cobble Hill. This report provides a summary of a desk study carried out and sets out my conclusions on the potential for impacts that a proposed new sewage effluent ground dispersal system may have on the local aquifer, and in particular, on two nearby water supply wells..

Background

The Valleyview Centre is located on two parcels of land in the northeast quadrant of the intersection of Cowichan Bay Road and Highway 1, about 2.5 Km north of the village of Cobble Hill, B.C. (see location on Fig. 1). This 4.9 hectare (ha) property includes Lot 1 Plan VIP8038 (PID 0035-633-133) and Lot 2 Plan 18824 (PID 003-778-304) Section 18, Range 5, Shawnigan Land District (the Property). According to the Valleyview Centre's website it has 59 tenants, the biggest of which is a food store, three are restaurants and most of the others are either offices or health related centers.

The original owners of the Property planned to build a commercial centre and retained an engineering firm to design a wastewater treatment system with ground dispersal of the treated effluent. A permit to operate a 87 m^3/day treatment system was issued by the BC Ministry of Environment in July 1994 (PE-3489). This permit allowed for discharge to two drainfields, each with 651 meters (m) of deep infiltration trench. However, in May 1996 BC Environment issued a "bypass" permit which allowed for two 100m long trenches with infiltrators, provided that the discharged effluent met criteria with a Biological Oxygen Demand (BOD) and Total Suspended Solids (TSS) of 10 mg/L and a fecal coliform count of less than 10 MPN/100mL. The installer of the system indicated that it had a 65 m^3/day capacity but could only produce Class B effluent quality, which met the BOD and TSS 10mg/L requirement but would have a higher fecal coliform count (up to 400 MPN/100mL/day).

With future plans to expand, the centre MSR Solutions Inc (MSRS) were retained by the current owners (Countryview Centre Ltd.) to design modifications to the system that would enable them to obtain the necessary permits. MSRS anticipates that the maximum daily design flow is expected to increase from 65 m^3/day to a maximum of 86 m^3/day and has determined that the existing dispersal field is inadequate for this increased discharge. Consequently, MSRS have submitted an application to the BC Ministry of Environment and Climate Change (ENV) to modify the existing wastewater treatment system to ensure that the 86 m^3/day discharge will have effluent quality that meets Class C criteria (i.e. BOD and TSS less than 10 mg/L and fecal coliform less than 400 MPN/100mL/day) and to construct rapid infiltration basins (RIBs) to replace the current effluent ground dispersal field.

B.C. Municipal Wastewater Regulation and Required Hydrogeological Assessments

This hydrogeological assessment is designed to provide background information and to address specific issues set out in the Municipal Wastewater Regulation (MWR) specifically the following:

Discharge - Section 70

(1) In this section, “zone of influence" means the zone around a water well that, in the opinion of a qualified professional, supplies water to the well.

(2) A person must not discharge, within the zone of influence, municipal effluent to ground unless

(a) the requirements set out in this Part are met, and

(b) the discharged effluent is disinfected.

(3) A person must ensure that a discharge of municipal effluent within 300 m of a drinking water source meets class A municipal effluent requirements.

Subsurface travel time – Section 72

(1) In this section, "subsurface travel time" means the actual time, including the time spent in the unsaturated and saturated zones, required for municipal effluent to travel from the disposal site perimeter to the point where the municipal effluent:
(a) surfaces,
(b) reaches a property line, or
(c) is intercepted by a water well.

(2) A discharger must ensure that the subsurface travel time is at least,
(a) for class A or B municipal effluent, 6 days, or
(b) for class C or D municipal effluent, 10 days.

Setback requirements – Section 82

(1) For all discharges to ground and standby areas, a discharger must ensure that setbacks from the area into which discharging occurs are at least the distance set out in Table 5.

(4) For the purposes of row 6 of Table 5, if, based on a hydrogeological assessment to determine the minimum distance required to protect the water quality of a water well,
(a) the distance from the water well must be extended in accordance with the hydrogeological assessment, or
(b) the maximum daily flow is more than or equal to 37 m3/d, the distance from the water well may be decreased, if authorized by a director, to a distance of no less than 90 m.

Infiltration basins – Section 83

A discharger must ensure that infiltration basins meet the following requirements:
(a) at least 2 basins must be provided to allow cleaning of one basin while the other is operating and to act as a safety factor for unusual conditions;
(b) for 2 basin systems, each basin must be capable of accepting all the municipal effluent under annual average rainfall conditions;
(c) subject to Division 1 *[General Requirements]*, discharge of municipal effluent to an infiltration basin meets at least class C requirements.

The relevant General Requirements are set out in the above listed Sections 70, 72 and 82 and the following assessments were carried out to provide the information required to address these requirements:

1) Develop a conceptual 3D model of the regional groundwater flow systems by creating hydrogeologic profiles and water table contours.
2) Analyze the hydraulics of the two high yielding production wells located on the Property.
3) Determine the extent of the zone of influence or capture zone when the wells are pumping at present day rates and at the anticipated future rates.
4) Determine the travel time between the RIBs and the nearest well.
5) Identify water quality impacts that that may have resulted from the operation of the existing sewage effluent dispersal fields and any future impacts.

Using this information answer the following questions:

1) If the distance between the RIB's and the nearest well was reduced from 300m to 150m, what impact would it have on the well water quality, both in the short and long term?
2) Do the proposed RIBs have the capacity to discharge up to 86 m^3/day?

Topography and Surface Water Drainage

The Property is located near the bottom of a wide valley at an elevation of about 60 meters above mean sea level (m-asl) and the ground slopes in a northwesterly direction towards Dugan Lake (elevation 45 m-asl). A network of ditches and two small creeks drain towards Dugan Lake (see channels on Fig. 1). Most of these channels are ephemeral with little to no groundwater providing base flow.

Soils.

The regional soils maps (Jungen et al 1985) have mapped three types of soils in the area as indicated on Fig. 1. These include imperfectly drained Fairbridge soils which are derived from marine silts and clay, moderately drained Hillbank soils and rapidly draining sandy Beddis soils which have developed mostly on fluvial marine deposits. The Fairbridge and Hillbank soils are predominant in the low lying areas and Beddis soils are predominant in the hill area located northeast of the Property.

Surficial Geology

Regional mapping of the surficial sediments in the area has identified marine and glacio-marine deposits at surface over much of the area covered by Fig. 1. This unit (7a) on the Halstead 1966 map is described as silt, clay, stony clay and till-like mixture (Vashon till). Below this unit are Quadra Sediments (Unit 3) which are a mixture of sand, silt, clay and minor gravel and peat. Quadra sediments (often called Quadra Sands) were deposited sub aerially on flood plains over a very wide area, which extended across and along the margins of the Georgia Depression, during the transition from non-glacial conditions (the Olympia non-glacial interval), back to glacial conditions (Claque 1977).

In most areas, the Quadra sediments overlie a glacial till unit, with lenses of gravel and silt, (Dashwood Drift (Hammond et al 2019) and below this unit there is a sedimentary bedrock unit belonging to the Nanaimo Group.

The Quadra Sediments form the principal aquifer in the area, and more details of this unit are provided in a following section which describes the aquifer characteristics and its depositional environment.

Local Area Wells

The locations of identified water wells in the area are indicated on Fig. 2 with assigned Well Tag Numbers (WTN). The data on these wells presented on Tables I and II are based on the Ministry of Environment's WELLS database. It is recognized that many of these wells may not be accurately located and the static water levels have changed since they were constructed. Also, many wells are currently not being used and the yields indicated on the table are likely not sustainable over the long term.

A review of the compiled information and statistics led to the following observations:

- There are records of five wells on the Property, one of which had an estimated yield of 37.7 L/s.
- Of the 31 well records listed on Tables I and II, none indicated a "dry hole" but many did not have a record of the estimated yield.
- The estimated well yields ranged from 0.2 to 37.7 litres per second (L/s) and median and average values were 1.6 and 4.2 L/s respectively.
- Well depths ranged from 9.4m to a 85.3m deep and median and average well depths were similar at 46.0m and 45.9m, respectively.
- Recorded depths to static water level at the time of well construction ranged from 4.9 to 41.1m and median and average depths to water were 20.7m and 20.4m, respectively. Many of the reported deep depths to water are not likely true values, as typically they are recorded in low yielding wells which require considerable time for the static water level to establish after construction and some only record a location where water entered the drilled hole.

Two of the wells located on the Property (85452 and 108356) are the principal source of water for the Cowichan Bay Water District (see locations on Fig. 3).

Local Aquifer

Regional aquifer mapping has designated the water bearing Quadra Sediment unit as the Cherry Point Aquifer (Aquifer Number 197).

This aquifer covers an area of approximately 39.5 km^2 and extends southward from Cowichan Bay and pinches out north of Mill Bay. The aquifer extends to Shawnigan Creek in the southwest, and the base of Cobble Hill and the Dougan Lake area in the west. The Quadra Sand unit was deposited by streams and rivers on flood plains extending over part, or all or the Georgia Depression, during late Wisconsin time. It is a well sorted, fine to coarse grained sand, with minor silt and gravel. As such, it is highly stratified and in some areas is cross bedded. In most areas it is overlain by Vashon till and related glacial sediments. In the area near the Property it is underlain by Dashwood drift.

Two profiles which are based on lithology described by water well drillers in their well logs are presented on Figs 4 and 5. While the lithologic descriptions provided by the drillers may not match that of a geologist, the descriptions present a relatively consistent trend in the area, which is a predominantly a relatively silt free sand unit below the Property which trends southwards into a predominantly silty sand unit. While the elevation of the water levels made at the time of construction represent different periods (both seasonal and long term), a relatively consistent trend (with a few exceptions) can be seen in the posted water table elevations on these two figures. The sloping water table lines represent groundwater gradients and from them the groundwater flow directions can be determined. For example: the aquifer gradient in the NE to SW oriented section on Fig. 4 is 6.4m per Km towards the Dougan flats and the gradient on the SE to NW section on Fig. 5 the gradient range from 14 to 11m per Km towards Dougan Lake.

Elevations of the water table at each well are posted on the well map (Fig. 3) and contours of the water table developed. As can be seen, the water table in the south was at an elevation of 60m-asl, around Cowichan Bay Road the water table lowers to 50m-asl. As indicated by the blue arrows, groundwater flow is orthogonal to the water table elevations which indicates that it flows northward, under the Property and likely discharges into Dugan Lake. The slope of the water table between 60 and 50 m-asl has a gradient of about 18m/Km but north of Cowichan Bay Road it flattens out to about 11 m/Km between there and Dugan Lake. This flattening out suggests the permeability of the aquifer in this northern area is higher than that in the south and is consistent with the predominantly silty sand in the south and silt free sand in the north (see profile on Fig. 5).

Aquifer Transmissivity

According to the web based BCMOE information on Aquifer 197 transmissivity values based on pump tests conducted on wells that are screened in the aquifer range 99.36 m^2/day (1 x 10^{-3} m^2/s) to 4,471 m^2/day (2 x 10^{-2} m^2/s) and the median value is about 188.08 m^2/day (2 x 10^{-2} m^2/s.). Pump tests have been conducted on the Braithwaite Estate and Cobble Hill Improvement District wells located about 2.5 Km south of the Property and according to EBA Engineering Ltd (2006) the transmissivity values ranged from 8 x 10^{-4} to 5 x 10^{-2} m^2/s.

Production Wells on the Property

As indicated earlier there are two production wells on the Property, both of which are owned and operated by the Cowichan Bay Waterworks District (CBWD). According to Associated Engineering (BC) Ltd. (2021) (Associated) the first well (Production well No. 2) was constructed in 1996. In 2009 responsibility for this well (WTN 85452) was taken over by the CBWD and was connected to their existing water distribution system. The well was developed and when tested it had a capacity of 31 L/s. However, it currently operates at a reduced rate (6 L/s) due to issues with sand production. In 2013, due to the decline in the Well 2 capacity, a second well (WTN 108356) was constructed at a distance of about 30m from Well 2. A 48 hour pumping test was conducted on this well (Well 1) and the results confirmed that its long-term capacity was 38 L/s (Thurber Engineering Ltd, 2013.) However, the well is currently pumped at 12 L/s due to the sizing of the installed pump. A nearby pump control building includes a disinfection system that was upgraded in 2021 to allow for individual dosing of each of the wells.

Associated Engineering Ltd. was retained by the CBWD to prepare a water system master plan and a draft plan was issued in 2021. In this plan Wells 1 and 2 are to be the principal source of water and a well located near the CBWD offices (6.3 L/s capacity) is to be used as a backup. Their analysis of water consumption indicated that the current average daily demand (ADD) for the CBWD was 8.2 L/s and that the projected 2041 ADD would be 11.3 L/s.

As indicated on Tables I and II and on Figs. 4 and 5, Well 1 is 60.4 m deep and Well 2 is 61m. The driller's logs indicated that after penetrating a layer of "clay" (3 to 10m thick) and for the remainder of each hole mostly medium to coarse sand was penetrated. A coarse sand and gravel was encountered in the bottom of each hole, where a stainless steel continuous wire wound well screen was installed.

A summary of the available depths to water measured in these two wells is provided on Table III. As can be seen the water levels have not changed much over the years indicating that the aquifer can easily sustain current pumpage from the well field. This data also demonstrates that the drawdown when the wells are in production is relatively small (a few metres).

Pump Testing of Well 1

The measured depth to water level and flow rate measured during the 46 hour pump test run on Well 1 was used to plot the graphs on Fig. 6 and the water level recovery data was used to plot the graph on Fig. 7. As can be seen on Fig. 6 during the period 1 to 10 minutes after the test started the rate of drawdown when plotted on a time log scale was relatively constant. The slope of this line (Leg 1) was used to calculate an aquifer transmissivity of 9×10^{-3} m^2/s, using the commonly used Jacob method. The slope of the next part of the graph (Leg 2) was much flatter and using the Jacob method an apparent transmissivity of 4×10^{-2} was calculated. This is interpreted as an apparent transmissivity as the well only screens 3m out of the 45m thickness of the aquifer and as such there is likely to be vertical leakance from the sand unit above down into the relatively permeable sand and gravel unit below. The drawdown curve for water levels measured in Well 2 is also presented on Fig 6 and as can be seen the transmissivity values calculated from this graph are similar to that of Well 1.

The water level recovery in Well 1 is presented on Fig 7 and as can be seen the Leg 1 values were similar to the pumped graphs and the Leg 2 values were about half. It is noted the water level trend on this recovery graph does not trend to a t/t' value of 1, which further confirmed that the sand and gravel aquifer is acting as a semi-confined aquifer with leakance from the sand unit above.

A distance – drawdown graph is presented on Fig. 8 and as can be seen the calculated transmissivity values are similar to those of Leg 2 in the other graphs. As there was no 1 to 10 minute data for Well 2, it was not possible to plot a distance graph of this period.

Using the Theis equation an attempt was made to simulate the measured drawdowns measured in both wells at the end of the pump test. The best fit simulation was for an assumed 2×10^{-2} m^2/s and a storativity of $10^{-4.}$ This suggests that a transmissivity of about $2 \times 10^{-2.}$ m^2/s. should be used to initially estimate the dimensions of the capture zone (same as the zone of influence) for the well field.

Capture Zone Analysis and Water Balance

Based on the local area gradient (11m per Km) and the best fit aquifer transmissivity value 2 x 10-2 m2/s) the width of the capture zone around the well field when pumping at a combined rate of 8.2 L/s (the ADD) was calculated using a conventional hydraulic analysis. As indicated on Table V this indicated that the width would be 37.3m. However, as indicated previously, the area around the well field has mostly silt free sand and the area in the south has mostly lower permeability silty sand, a different type of analysis was necessary to determine the width of the capture zone in this area. This involved calculating the recharge area required to sustain the well field flow. Using recharge rates provided in Harris and Usher 2017 the required recharge area was determined, and from this, an average of recharge zone width of

279m was calculated (see Table VI). As a check on the validity of this width, an estimate of the aquifer transmissivity and hydraulic conductivity was performed using the Darcy equation. As indicated on the bottom of Table VI, this indicated a transmissivity of 1.6 x 10^{-3} m^2/s and a hydraulic conductivity of 3.6 x 10^{-5} m/s. This transmissivity value was similar to the median transmissivity of the 197 aquifer and similar to those obtained from pump tests run on Braithwaite and Cobble Hill wells located in the south (EBA Engineering Ltd., 2006) and hence the 279m wide capture zone appeared reasonable. The capture zone will follow the flow lines with the steepest gradient, and hence this 279m wide zone in the southern area is centered along this line, as indicated on Fig. 3. The transition from the 279m wide zone to the 37.3m wide zone around the wells is difficult to determine as there no current water level elevations in the two wells located to the south (40670 and 85452) as indicated on Fig. 3. There is no record of these well being decommissioned, however a search in the field was not able to locate these wells and make depth to water measurements. These wells did not penetrate to the same depth as the two Production wells (see profile on Fig. 5), but it is relevant to note that Well 85452 has a relatively high yield (7.5 L/s) and prior to 2013 it was used as a backup CBWD water supply well.

As indicated on Fig. 3, the current well field capture zone does not pass under the existing sewage effluent dispersal field. The hydraulic analysis for the well field capture zone when the ADD increased to 11.3 L/s in 2041 indicates it will have a 51.4m width (see Table V) and the average width of the southern part of the capture zone will increase to 283m (see Table VI). These are relatively small changes and so even when the discharge to the proposed new seepage beds is increased to 86 m^3/d it is concluded that the infiltrating effluent from the future fields and beds will still not reach the two production wells.

Travel Times

Even though it is concluded that the discharged effluent does not reach the well field, the MWR requires an assessment of the potential travel time from the effluent dispersal facility to the nearest well. As set out on Table VII the travel time from the proposed new infiltration basin and Well 1 is presented for both current (2021) and future (2041) pumping rates. These analyses indicated that the dispersed effluent could take about a day to percolate through the 19m thick unsaturated zone to reach the water table. Once in the water table it will take about 234 days to reach Well 1, for a total travel time of 235 days. When the well field ADD is increased to 11.3 L/s, the gradient will increase and the total travel time will decrease to 208 days. These travel times are much greater that the minimum 6 days as set out in the MWR for Class B effluent dispersal.

Effluent Dispersal into the Ground.

As set out in Payne Engineering and Geology (1995 and 1999) (PEG) and MSR Solutions Inc (2022) the existing effluent dispersal field consist of two dispersal fields, each with 100m long trenches. As indicated earlier, this was constructed in 1997 under the May 1996 BC Environment issued a “bypass” permit. Each field comprised two 50m long trenches. As the near surface soil was not very permeable a series of about 4m deep and 1m wide trenches with sand backfill were constructed to enable percolation into a deep, more permeable, sand unit. The effluent is evenly distributed using a pressurized system into each trench via 0.86m wide (34-inch) wide infiltrator chambers. It is understood that the operator has been alternating the discharge between the two 100m long dispersal fields. When Mr. Payne visited the site in 1999 he determined that the discharge was about 25 m^3/day and that the hydraulic loading rate in the dispersal field was 83 mm/day. However, as indicated in PEG (1999) the bypass permit was for 87 m^3/day and if it had all been pumped into one 100m length of field the HLR at the bottom of the trench would be 1,450 mm/day. When he visited the site in 1999 he inspected the sand interface under some of the infiltrators and did not observe any build up of organic matter and there was no evidence of sewage surfacing on the slope below the dispersal field.

According to PEG (1995) the installers installed a 5.7m deep monitoring tube (piezometer) near the dispersal field and after the discharge commenced no water was detected.

MSR Solutions Inc (2022) conducted an investigation in the area in and east of the existing dispersal field. This involved digging six 2m to 3.2m deep test pits, conducting three permeameter tests and one 16 day infiltration test at a rate of 11.4 m^3/day. The locations of the test pits are indicated on Fig. 3. The logs of these pits indicated that they encountered mostly silty clay and only Test Pit 3 (TP-3) encountered permeable sand and gravel. The estimated Kfs values for the permeameter tests ranged from 3×10^{-7} m/s in a silt clay loam material in TP 2 to 3.8×10^{-4} m/s in TP-3. The infiltration test was conducted in a 100mm diameter perforated pipe installed to a depth of 3m in a test pit near TP-3 that had penetrated into a sand and gravel unit. The water level in the discharge casing stabilized when an 11.4 m^3/day discharge was applied for 16 days. The calculated Kfs based on this test data was about 13,000mm/day (1.4×10^{-4} m/s), which was slightly lower than the permeameter test result.

Based on the results of these tests and the hydraulic performance of the existing dispersal fields, it was decided to design a large seepage bed or Rapid Infiltration Basin (RBI) that would be located in the general area of TP-3. Plans and profiles of this facility are presented on Figs. 9 and 10 and hydraulic analyses are presented on Table VIII. The plan is to have one 14 by 10m wide sand bed that would be dug down into the underlying sand and gravel unit and to fill the bed with clean sand. The bed would be divided into three sections using lock blocks leaving three surface areas each measuring 6 by 10m. It is understood that MSR Solutions Inc. plan to have effluent flood the surface of the sand beds and essentially create three rapid infiltration basins. They plan to apply effluent to only one bed at a time and cycle from one to another, which will provide time for resting the surface and for conducting maintenance when required. As indicated on Table VIII, the maximum hydraulic loading rates (HLR) on each individual bed would be 1,433 mm/day and when averaged over three beds it would be 478 mm/day. These parameters are similar to the HLR values based on MWR and for the bypass approval for the existing dispersal field (see calculations set out on Table VIII).

As an infiltration bed exposes the effluent to the atmospheric dust and plant materials can land in the beds it can lead to lead maintenance issues. Also, there will be a need to fence off the area and there may be an odour problem especially during hot weather. For these reasons consideration should be given to install infiltrators and covering them with soil. As the infiltrator panels will reduce the infiltration area slightly, it will be necessary to increase the width of the beds from 10 to 11.5m (see calculations on Table VIII).

Water Quality

MSR Solutions Inc (2022) have provided extensive information on water supply well water quality as well as effluent water quality and long term local area monitoring well data. As they and PEG (1999) have indicated there is no evidence of the effluent dispersal having an impact on well water quality. As indicated, nitrate is often used as a tracer as it does not absorb to sediments and typically does not change form as it flows in the sub surface. The nitrate concentrations in the effluent are typically in the range 40 to 55 mg/L range and the nitrate concentrations in the two CBWD production wells are in the

0.07 to 0.34 mg/L range. The current effluent discharge rate is 23.4 m^3/d (0.27 L/s) and the well field discharge is about 8.2 L/s which could provide a dilution of about 30 and hence effluent with a nitrate concentration of 45 mg/L would result in a 1.5 mg/L nitrate concentration in well water. The lack of nitrate in the well water supports the conclusion that the plume from the existing dispersal field does not enter the well field capture zone.

Conclusions

1) All CBWD well field is located in a relatively permeable portion of the aquifer, which does not extend into the less permeable area towards the south, where most of the well field the recharge comes from.
2) The width of the capture zone is relatively narrow in the more permeable zone and gradually expands towards the south, with an average width of about 279m.
3) The capture zone does not pass under the existing dispersal field, which is consistent with the lack of any evidence of sewage effluent having an impact on the well field water quality.
4) When the well field pumping rate is increased to an anticipated 11.3 L/s in the year 2041, the expanded well field capture zone will not pass under the proposed RIBs and hence effluent discharged will not have an impact on well water quality.
5) The distance between the RIBs and the nearest well (Well 1) is 150m and the estimated travel time between these two is 235 days, which is much in greater than the 6 days set out in the MWR for a Class B effluent dispersal system. When the well field pumpage increases to the anticipated 11.3 L/s in year 2041, the travel time will be reduced to about 208 days and will still meet the MWR criteria..
6) The proposed RIBs were designed on the basis of a 1,433 mm/day hydraulic loading rate, which is similar to that in the previously approved existing dispersal field, when a design maximum daily discharge is applied. For this reason the proposed RIBs meet the criteria set out in the MWR.

Recommendations

1) Consideration should be given to installing infiltrator panels over the sand bed to ensure an even distribution of effluent, as well as minimize the impact of fugitive dust and organic matter accumulating on the infiltration surface, to minimize odour issues and to maximize site security.
2) The upper 0.5 m of the sand bed should meet C33 sand criteria, to ensure maximum infiltration.
3) The four regional monitoring wells stipulated by the MWR for water quality testing should include: Wells 108356 (Well 1), 85452 (Well 2), 63623 and 65065. In addition, depths to water level in Well 63623 should periodically be monitored, preferably with an automatic water level recorder, such as a datalogger.

Limitations.

This investigation has been conducted using a standard of care consistent with that expected of scientific and engineering professionals undertaking similar work under similar conditions in B.C. No warranty is expressed or implied.

I trust that this is sufficient for you present purposes.

Yours truly,

Elanco Enterprises Ltd.

R. Allan Dakin, FEC, P. Eng.
Senior Groundwater Engineer

Att.

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Tables

Table I
Information on Wells - Sorted by Well Tag Number

Well Tag Number	Construct	Well Diam	Intended Water Use	Address		Depth to (m)			Elevation (m-asl)			Yield
	Date	mm		No.	Street	SWL TOC	SWL	Bottom	Ground	SWL	Bottom	L/s
8982	Jan-50		Unknown	1500	Freeman Rd.	na	na	24.4	69	na	44.6	na
8985	Jan-50		Unknown	1400	Cowichan Bay Rd	14.3	13.7	17.1	64	50.3	46.9	na
13830	Jan-52		Unknown	3965	Highway 1	na	na	9.4	72	na	62.6	na
21297	Feb-68		Unknown	3965	Highway 1	13.7	13.1	49.1	74	60.9	24.9	0.2
24760	May-71		Unknown	3905	Highway 1	32.0	31.4	68.3	93	61.6	24.7	3.1
26018	Mar-72	150	WS System	1451	Freeman Rd.	18.9	18.3	43.3	75	56.7	31.7	5.8
30370	22-May-74		Unknown	1425	Cowichan Bay Rd	24.4	23.8	41.1	76	52.2	34.9	0.5
39909	19-Jun-78		Unknown	1451	Freeman Rd.	20.7	20.1	36.3	75	54.9	38.7	1.9
40670	24-Sep-78		Unknown	1400	Cowichan Bay Rd	13.7	13.1	29.3	62	48.9	32.7	1.1
51240	28-Oct-82		Unknown	1415	Freeman Rd.	15.2	14.6	85.3	74	59.4	-11.3	na
53919	20-Aug-84		Irrigation	1360	Freeman Rd.	6.4	5.8	32.6	70	64.2	37.4	4.7
56007	16-May-86		Domestic	1441	Cowichan Bay Rd	30.5	29.9	48.8	85	55.1	36.2	0.2
60500	09-Dec-93	150	WS System	1451	Freeman Rd.	22.9	22.3	42.1	75	52.7	32.9	1.9
63024	Oct-82		Domestic	1500	Freeman Rd.	15.2	14.6	35.5	71	56.4	35.5	1.3
63623	25-Apr-90	150	Irrigation	1420	Cowichan Bay Rd	27.4	26.8	46.0	77	50.2	31.0	2.5
64015	14-Mar-89		Domestic	1614	Tommy Rd.	6.1	5.5	11.9	52	46.5	40.1	0.4
65065	08-Jul-81		Domestic	1440	Cowichan Bay Rd	22.9	22.3	50.0	76	53.7	26.0	0.6
67022	07-May-90		Irrigation	3925	Highway 1	30.2	29.6	72.5	90	60.4	17.5	4.7
68621	07-May-90		Irrigation	3925	Highway 1	30.2	29.6	72.5	86	56.4	13.5	4.7
68623	13-Dec-93		Irrigation	1500	Freeman Rd.	22.6	22.0	39.3	72	50.0	32.7	3.8
68628	26-May-93	150	Domestic	4090	Joseph Rd.	41.1	40.5	68.6	89	48.5	20.4	1.3
77081	08-Jul-81	150	Domestic	1440	Cowichan Bay Rd	22.9	22.3	50.0	76	53.7	26.0	0.6
77087	26-May-93		Domestic	4020	Joseph Rd.	41.1	40.5	68.6	89	48.5	20.4	1.3
77097	30-Sep-98		Domestic	3966	Cobble Hill Road	4.9	4.3	47.9	65	60.7	17.1	1.6

Table I

Information on Wells - Sorted by Well Tag Number

Well Tag Number	Construct	Well Diam	Intended Water Use	Address		Depth to (m)			Elevation (m-asl)			Yield
	Date	mm		No.	Street	SWL TOC	SWL	Bottom	Ground	SWL	Bottom	L/s
84512	08-Dec-95	150	Domestic	1445	Cowichan Bay Rd	na	na	58.8	100	na	41.2	0.9
84861	25-Feb-92	150	Domestic	1400	Freeman Rd.	na	na	32.6	74	na	41.4	1.3
85452	**18-Oct-96**	150	**VV Well #2**	**1420**	**Cowichan Bay Rd**	**11.6**	**10.74**	**61.0**	**59.845**	49.1	**-1.1**	**18.9**
85453			WS System	1400	Cowichan Bay Rd	6.1	5.5	36.6	62	56.5	25.4	7.5
108356	**02-Jul-13**	**250**	**VV Well #1**	**1420**	**Cowichan Bay Rd**	**12.2**	**11.69**	**60.4**	**60.173**	48.5	**-0.2**	**37.7**
108440	08-Aug-13	150	Domestic	4101	Judge Drive	19.5	18.9	34.7	76	57.1	41.3	2.5
124772	06-Oct-21	150	Domestic	4154	St Catherines Rd.	24.1	23.5	50.0	80	56.5	30.0	0.9
Number of Wells =		31										
Minimum						4.9	4.3	9.4	52.0	47.7	-11.3	0.2
Median						20.7	20.1	46.0	75.0	54.9	31.7	1.6
Average						20.4	19.8	45.9	74.8	55.0	28.9	4.2
Maximum						41.1	40.5	85.3	100.0	59.5	62.6	37.7

Notes:
1) See locations of wells on Fig 3
2) SWL = static water level at time of construction. TOC = top of well screen.

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Table II
Information on Wells - Sorted by Address

Well Tag Number	Construct	Well Diam	Intended Water Use	Address		Depth to (m)			Elevation (m-asl)			Yield
	Date	mm		No.	Street	SWL TOC	SWL	Bottom	Ground	SWL	Bottom	L/s
77097	30-Sep-98		Domestic	3966	Cobble Hill Road	4.9	4.3	47.9	65	60.7	17.1	1.6
8985	Jan-50		Unknown	1400	Cowichan Bay Rd	14.3	13.7	17.1	64	50.3	46.9	na
40670	24-Sep-78		Unknown	1400	Cowichan Bay Rd	13.7	13.1	29.3	62	48.9	32.7	1.1
85453			WS System	1400	Cowichan Bay Rd	6.1	5.5	36.6	62	56.5	25.4	7.5
63623	25-Apr-90	150	Irrigation	1420	Cowichan Bay Rd	27.4	26.8	46.0	77	50.2	31.0	2.5
85452	**18-Oct-96**	150	**VV Well #2**	**1420**	**Cowichan Bay Rd**	**11.6**	**10.74**	**61.0**	**59.845**	49.1	**-1.1**	**18.9**
108356	**02-Jul-13**	**250**	**VV Well #1**	**1420**	**Cowichan Bay Rd**	**12.2**	**11.69**	**60.4**	**60.173**	48.5	**-0.2**	**37.7**
30370	22-May-74		Unknown	1425	Cowichan Bay Rd	24.4	23.8	41.1	76	52.2	34.9	0.5
65065	08-Jul-81		Domestic	1440	Cowichan Bay Rd	22.9	22.3	50.0	76	53.7	26.0	0.6
77081	08-Jul-81	150	Domestic	1440	Cowichan Bay Rd	22.9	22.3	50.0	76	53.7	26.0	0.6
56007	16-May-86		Domestic	1441	Cowichan Bay Rd	30.5	29.9	48.8	85	55.1	36.2	0.2
84512	08-Dec-95	150	Domestic	1445	Cowichan Bay Rd	na	na	58.8	100	na	41.2	0.9
53919	20-Aug-84		Irrigation	1360	Freeman Rd.	6.4	5.8	32.6	70	64.2	37.4	4.7
84861	25-Feb-92	150	Domestic	1400	Freeman Rd.	na	na	32.6	74	na	41.4	1.3
51240	28-Oct-82		Unknown	1415	Freeman Rd.	15.2	14.6	85.3	74	59.4	-11.3	na
26018	Mar-72	150	WS System	1451	Freeman Rd.	18.9	18.3	43.3	75	56.7	31.7	5.8
39909	19-Jun-78		Unknown	1451	Freeman Rd.	20.7	20.1	36.3	75	54.9	38.7	1.9
60500	09-Dec-93	150	WS System	1451	Freeman Rd.	22.9	22.3	42.1	75	52.7	32.9	1.9
8982	Jan-50		Unknown	1500	Freeman Rd.	na	na	24.4	69	na	44.6	na
63024	Oct-82		Domestic	1500	Freeman Rd.	15.2	14.6	35.5	71	56.4	35.5	1.3
68623	13-Dec-93		Irrigation	1500	Freeman Rd.	22.6	22.0	39.3	72	50.0	32.7	3.8
24760	May-71		Unknown	3905	Highway 1	32.0	31.4	68.3	93	61.6	24.7	3.1
67022	07-May-90		Irrigation	3925	Highway 1	30.2	29.6	72.5	90	60.4	17.5	4.7
68621	07-May-90		Irrigation	3925	Highway 1	30.2	29.6	72.5	86	56.4	13.5	4.7

Table II

Information on Wells - Sorted by Address

Well Tag Number	Construct	Well Diam	Intended Water Use	Address		Depth to (m)			Elevation (m-asl)			Yield
	Date	mm		No.	Street	SWL TOC	SWL	Bottom	Ground	SWL	Bottom	L/s
13830	Jan-52		Unknown	3965	Highway 1	na	na	9.4	72	na	62.6	na
21297	Feb-68		Unknown	3965	Highway 1	13.7	13.1	49.1	74	60.9	24.9	0.2
77087	26-May-93		Domestic	4020	Joseph Rd.	41.1	40.5	68.6	89	48.5	20.4	1.3
68628	26-May-93	150	Domestic	4090	Joseph Rd.	41.1	40.5	68.6	89	48.5	20.4	1.3
108440	08-Aug-13	150	Domestic	4101	Judge Drive	19.5	18.9	34.7	76	57.1	41.3	2.5
124772	06-Oct-21	150	Domestic	4154	St Catherines Rd.	24.1	23.5	50.0	80	56.5	30.0	0.9
64015	14-Mar-89		Domestic	1614	Tommy Rd.	6.1	5.5	11.9	52	46.5	40.1	0.4
Number of Wells =		31										
Minimum						4.9	4.3	9.4	52.0	47.7	-11.3	0.2
Median						20.7	20.1	46.0	75.0	54.9	31.7	1.6
Average						20.4	19.8	45.9	74.8	55.0	28.9	4.2
Maximum						41.1	40.5	85.3	100.0	59.5	62.6	37.7

Notes:
1) See locations of wells on Fig 3
2) SWL = static water level at time of construction. TOC = top of well screen.

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Table III

Well Field Pumpage

Year	Well 1		Well 2		Total		
	m^3/yr	L/s	m^3/yr	L/s	m^3/yr	m^3/day	L/s
2018 [4]	54,390	5.3	22,234	2.2	76,624	628.1	7.3
2019	186,004	6.1	90,745	3.0	276,749	758.2	8.8
2020	184,515	6.0	87,163	2.8	271,678	744.3	8.6
2021	209,034	6.8	90,626	3.0	299,660	821.0	9.5
Average		6.1		2.7		737.9	8.5
Proportion		70%		30%			100%

Month	$m^3/month$	L/s	$m^3/month$	L/s	$m^3/month$	m^3/day	L/s
Jul-21	26,334	9.8	12,653	4.7	38,987	1,258	14.6
Feb-21	12,991	5.4	5,807	2.4	18,798	606.4	7.0

Instantaneous [5]

09-Jan-23		4.6		2.3			6.9
Proportion		67%		33%			100%

Long term prediction [6]

2021		5.7		2.5		ADD	8.2
2026		6.6		2.8		ADD	9.4
2041		7.9		3.4		ADD	11.3

Notes
1) See locations of wells on Fig. 3.
2) Volumes of water pumped for 2018-2020 from Associated Engineering (BC) Ltd., 2022 and 2021 data from Western Water Associated Ltd 2022.
3) ADD = Average daily demand for CBWD and not just Wells 1 and 2.
4) Data for 2018 was for September to December only.
5) Provided by MSR Solutions Inc staff.
6) From Table 3-10 in Associated Engineering (BC) Ltd. 2022

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Table IV

Depth To Water in Wells and Inter Well Gradients on the Property

Station	Date	DTW	Elevation (m-asl)		Pumping Rate	Gradient	See Note	Comments
		m	Datum	WL	L/s	m/Km		
Well 1	02-Jul-13	12.2	60.675	48.48	0.0		1	At construction
	10-Sep-13	12.100	60.675	48.58	0.0		2	Start of 48 hour test
	12-Sep-13	15.969	60.675	44.71	37.9		2	End of 48 hour test
	09-Jan-23	13.6	60.675	47.08	4.6		3	MSRSI measurement
Well 2	18-Oct-96	11.6	60.686	49.09	0.0		1	At construction
	01-Oct-99	9.23	59.845	50.62	0.0		4	M. Payne visit
	10-Sep-13	12.200	60.686	48.49	0.0		2	Start of 48 hour test on Well 1
	12-Sep-13	15.239	60.686	45.45	0.0		2	End of 48 hour test on Well 1
	09-Jan-23	13.6	60.686	47.09	2.3		3	MSRSI measurement
63623	25-Apr-90	27.4	77.6	50.20	0.0		1	At construction
	09-Jan-23	27.3	77.6	50.30	0.0		3	MSRSI measurement
Gradient	Regional				0.0	15.0	5	
	At construction				0.0	5.4	1, 6	Calculated gradients between Well 63623 and Well 2 - a distance of 205m (see Fig. 3).
	10-Sep-13	Start of pump test			0.0	7.9	2, 6	
	12-Sep-13	End of 48 hour pump test			37.9	23.2	2, 6	
	09-Jan-23				6.9	15.7	3, 6	
	Best fit calculated values		Well 2	63623				
	09-Jan-23		47.08	50.3	6.9	15.7		
	2021 ADD		46.78	50.21	8.2	16.7	6, 7	
	2041 ADD		46.10	49.98	11.3	18.9		

Notes:

1) See locations of wells on Fig. 3 and elevations of water levels at time of construction - as dates vary this only an approximate value
2) Based on depth to water measured when Well 1 was pump tested in 2013 (see Thurber Engineering Ltd., 2013)
3) Based on water levels and pumping rates observed by MSR Solutions Inc staff on January 2023.
4) A depth to water level made in Well 2 during a site visit (Payne Engineering Geology. Nov. 1999)
5) See water table contours on Fig. 2 and interpreted gradient indicated on Fig. 5.
6) Distance between Well 63623 = 205 m
7) Gradients calculated using the Theis formula and best fit values for transmissivity (2 x 10^{-2} m^2/s) and storativity (0.0001).

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Table V

Capture Zone Analysis and Travel Time

Example of a Capture Zone Analysis

Formula	Symbol	Parameter	Note No	Value	Unit
	Q	2021 Pumping Wells 1 and 2	1	708	m^3/d
				8.2	L/s
				130	gpm
				187,747	gpd
	T	Transmissivity	2	2.0E-02	m^2/s
				1,728	m^2/d
	i	Gradient	3	0.011	
Y = Q/2T/i	Y	half width	4	18.6	m
	2Y	Width		37.3	m
X = Y/pi	x	down distance	4	5.9	m

Summary of results

Year	Well	Rate L/s	2Y m	x m
2021	Well 1 and 2	8.2	37.3	5.9
2041	Well 1 and 2	11.3	51.4	8.2

Notes:

1) Well pumping rates from Table III
2) Transmissivity value based on an interpretation of the results of a 48 hour pump test run on Well 1 (see Figs. 6, 7 and 8 and information provided in the "Pump Testing of Well" section of this report)
3) Estimated regional gradient (see Fig 5 and Table III)
4) See dimension illustration below

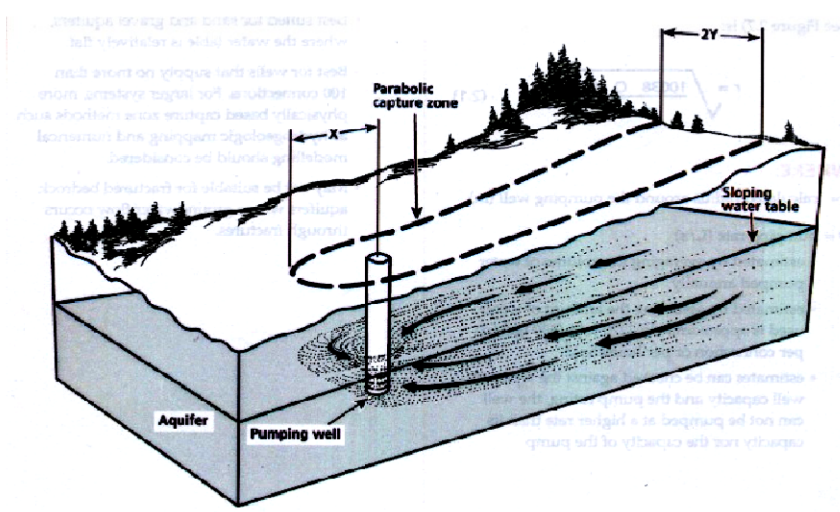

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Table VI

Well Field Water Balance

Item	Units	2021	2041	Note No.
Pumpage from Wells 1 and 2	L/s	8.2	11.3	1
Average effluent discharge	m^3/day	23.4	86	2
	L/s	0.27	1.00	
Percent of well field pumpage if included in capture zone		3.3%	8.8%	
Effluent contribution to well field	L/s	0.0	0.0	
Pumpage minus effluent discharge	L/s	8.2	11.3	
Annual precipitation	mm	1361	1429	3
Groundwater recharge				
Beddis Soils recharge factor		50%	45%	4
Beddis Soils annual recharge rate	mm/yr	681	643.0725	
Fairbridge & Hill bank (Other soils) soils factor		25%	25%	4
Other soils rate	mm/yr	340	357.2625	
Portion of capture zone under Beddis soil		1%	1%	5
Area of Beddis soils	m^2	7,533	7,641	
Recharge through Beddis soils	m^3/yr	5,126	4,914	
	L/s	0.16	0.2	
Portion of capture zone under other soils		99%	99%	
Area of Other Soils (Fairbridge and Hill Bank)	m^2	745,767	756,459	
Recharge through Other Soils	m^3/yr	253,747	270,254	
	L/s	8.05	8.57	
	m^3/yr/m^2	0.340	0.357	
Capture zone				
Length	m	2,700	2,700	6
Width	m	279	283	7
Area	m^2	753,300	764,100	
Capture Zone Recharge	m^3/yr	258,873	275,168	
	L/s	8.2	11.3	

Estimate hydraulic conductivity K = Q/(i *D*W)	Units	2021	2041	Note No.
Thickness of aquifer (D)	m	45	45	8
Width of capture zone (W)	m	279	283	See above
Gradient (i)	m/Km	18	18	9
Flow rate (Q) from Other Soil recharge area	L/s	8.05	8.57	See above
Transmissivity	m^2/s	1.6E-03	1.7E-03	Calculated [10]
Hydraulic conductivity	m/s	3.6E-05	3.7E-05	Calculated

Notes
1) See location of wells on Fig 3 and water usage quantities on Table III.
2) Design average daily sewage effluent discharges into the RIB (MSR Solutions Inc 2022)
3) 30 year annual average precipitation from Duncan Kelvin Creek Stn. (located about 62 Km NW)
Assumes a 5% increase in annual precipitation by 2041
4) Factors selected based on soil type, experience and data presented on Fig. 6 in Harris and Usher (2017).
The infiltration rate in Beddis soils is predicted to decrease and in the other soils to remain the same.
5) Estimate of Beddis sols located in capture zone (see soils on Fig. 1)
6) Distance to groundwater divide located near Fisher Road (south of the Property)
7) Width required to infiltrate the recharge rate through the Other Soils.
8) Interpreted from profile presented on Fig. 5
9) Gradient calculated from contours presented on Fig. 3.
10) Median transmissivity for Aquifer 197 is 2.2 x 10^{-3} m^2/s

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Table VII

Hydraulic Loading on Sand Bed

MWR Requirements

Parameter	Quantity	Units
Length of pipe per field	50	m/10m^3/d
Planned discharge	86	m^3/d
Required pipe (prior to reduction)	430	m
Section 79(1) reduction factor	60%	
Reduced pipe length	258	m
Depth of trench (H)	2	m
Factor 1/H$^{0.5}$	0.71	
Max allowed for depth reduction	0.8	
Required pipe (after all reductions)	206.4	m
Infiltrator width	0.86	m
Required area under infiltrators	177.5	m^2
Average HLR for discharge to three beds	484	mm/d
HLR applied to one of three beds	1453	mm/d

MRS Solutions Ltd. Approach

Parameter	Quantity	Units
Length of bed	10	m
Width of bed	6	m
Area of bed	60	m^2
Number of beds	3	
Total area	180	m^2
Horizontal HLR from site infiltration test	13,000	mm/d
	1.5E-04	m/s
Design discharge	86	m^3/d
Gross HLR on sand bed	478	mm/d
Intermittent HLR on individual beds	1,433	mm/d

Proposed method of application to surface of sand bed

Parameter	Quantity	Units
Bed width	11.5	m
Chambers lines per bed	2	
Number of beds	3	
Total length of chamber lines	69	m
Area under infiltrators	59.34	m^2
HLR	1,449	mm/day

Notes:

1) The above calculations are for average daily discharge to a sand bed similar to that indicated on Figs 9 and 10.
2) The MWR requires two fields (or beds) be constructed and a reserve area for a third.
3) Allowed HLR under existing bypass permit is 1,450 mm/day.

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Table VIII

Travel Time Between Proposed Sand Bed and Well 1

Travel time to Well 1 for 8.2 L/s well field ADD in 2021					
Formula	Symbol	**Parameter**	**Value**	**Units**	**See Note No.**
	D	Distance from bottom of RIB to water table	19	m	1
	K_v	Estimated vertical conductivity	1.5E-04	m/s	2
	n	Porosity	0.3		
	$\mathbf{i_v}$	Vertical gradient	0.5		3
$V= K_v i/n$	V_v	Vertical migration rate	21.7	m/day	
D/V_v	Tv	Time to reach water table	0.9	days	
	L	Horizontal distance (SB to Well 1)	150	m	1
		Aquifer transmissivity	6.E-03	m^2/s	4
		Aquifer thickness	45	m	5
	K	Kh	1.3E-04	m/s	6
	n	Aquifer porosity	0.3		7
	$\mathbf{i_h}$	Horizontal gradient for 2021 ADD	0.0167		8
V= Ki/n	V_h	Horizontal velocity	0.6	m/day	
L/V_h	T_h	Horizontal travel time	234	days	
Th + Tv		Travel time to Well 1	235	days	

Travel time to Well 1 for 11.3 L/s well field ADD in 2041			
Time to reach water table	0.9	days	see above
Horizontal Gradient for 2041 ADD	0.0189		8
Horizontal travel time	207	days	
Travel time to Well 1	208	days	

Notes:
1) See dimensions on Fig 3.
2) Hydraulic conductivity measured in Test Pit 3 is assumed to indicate the horizontal conductivity K_h and an assumed anisotropy of 10 was used to calculate vertical conductivity (K_v).
3) Approximate maximum vertical gradient accounting for unsaturated flow in inhomogeneous anisotropic media.
4) Best fit transmissivity calculated from back analysis of 48 hour pump test bottom of Table IV
5) See profile on Fig. 5
6) This assume value is based on the 1.5 x 10^{-4} m/s value calculated from an inflow test in the Sand Basin area (see table on Fig. 10).
7) Typical value for sand.
8) Estimated gradients for 8.2 L/s ADD and 11.3 L/s ADD flows (see Table IV)

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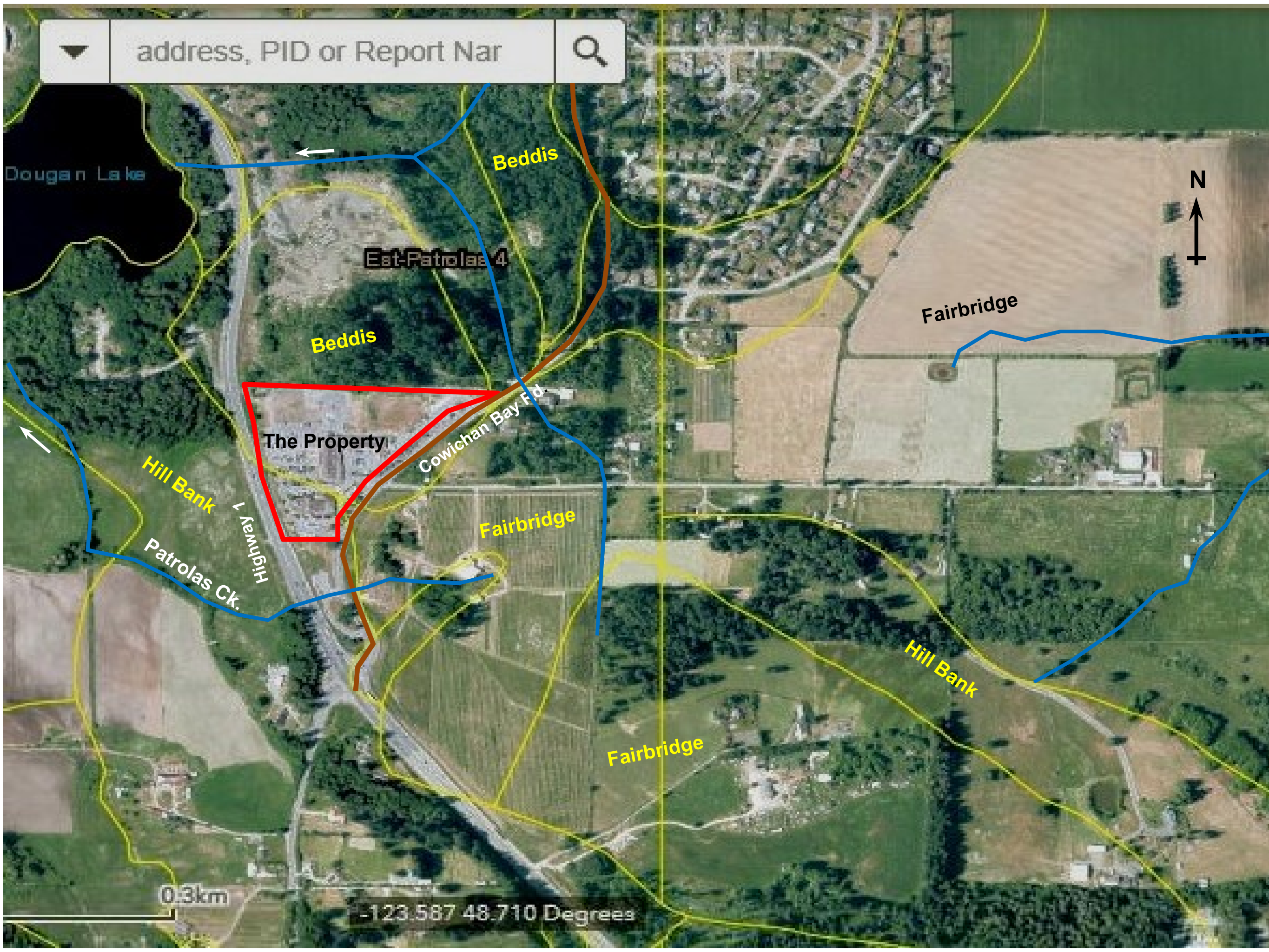
Figures

JOB NUMBER: 294



Scale (1:10,000)

0 150 300m

Soils	Description	Depositional Environment	Drainage
Beddis	Coarse textured (sandy) brunisol	Fluvial marine and/or eolian	Rapid
Fairbridge	Silt loam and clay silt loam. Gleyed dystric brunisol.	Marine	Imperfect
Hillbank	Silt loam & organic silt loam. Orthic dystric brunisol.	Morainal overlying fluvial	Moderate

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NOTES
1) See detailed map of the northern part of the Property on Fig. 3.

Countryview Centre Ltd.

ELANCO ENTERPRISES LTD.
Victoria, B.C. (250 744-1357)
Permit to Practice No: 1001505

Assessment of Impact of Dispersal Field on Local Aquifer. Valleyview Shopping Centre, 1400 Cowichan Bay Road, Cobble Hill, B.C.

Property Location Map.

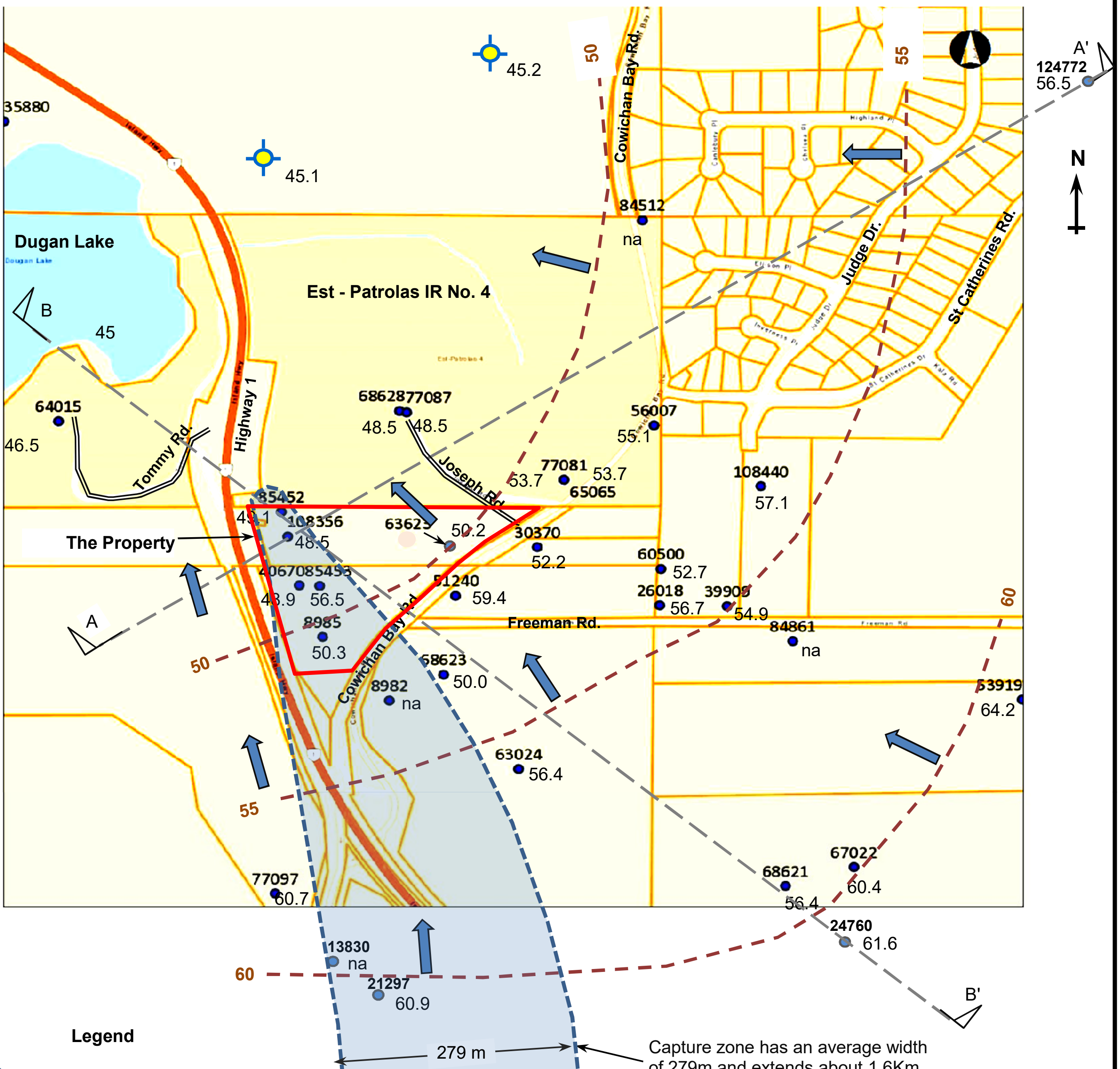
Drawn:

Approved:

Date
Feb. 2023

Fig.
1

JOB NUMBER: 294



Legend

Approximate northern portion of existing well field capture zone.

Water well from MOE wells database with indicated well tag number (See Note 2)

60.7 Water table elevation (m-asl)

Approximate water table contour (m-asl)

Groundwater flow direction

A Hydrogeological Section (see Figs 3 and 4)

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Scale (1:8,000)

0 100 200m

NOTES
1) Base map from BCMOE G-Wells web site.

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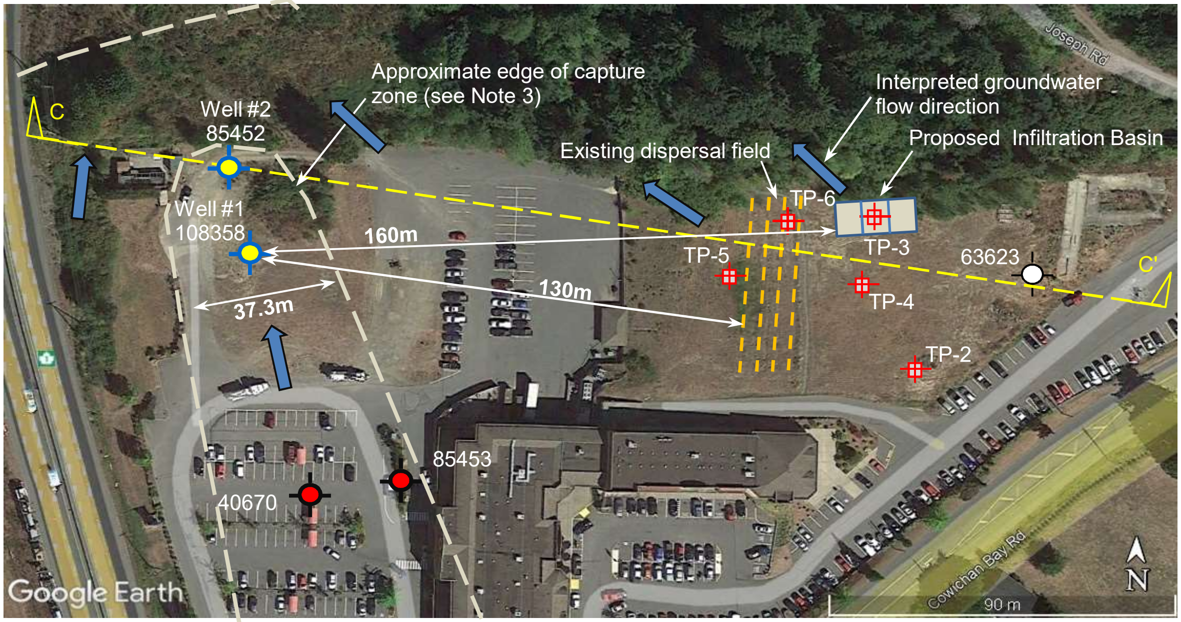
Assessment of Impact of Dispersal Field on Local Aquifer. Valleyview Shopping Centre, 1400 Cowichan Bay Road, Cobble Hill, B.C.	**Local Area Wells and Interpreted Water Table Contours**	Drawn: Approved:	Date Feb. 2023 Fig. **2**

File:2600 \ 294 Figures 2 Well Map 2/13/2023 5:37 PM

JOB NUMBER: 294



N

TP-1

Map Legend

Cowichan Bay Water District Production well with indicated well tag number.

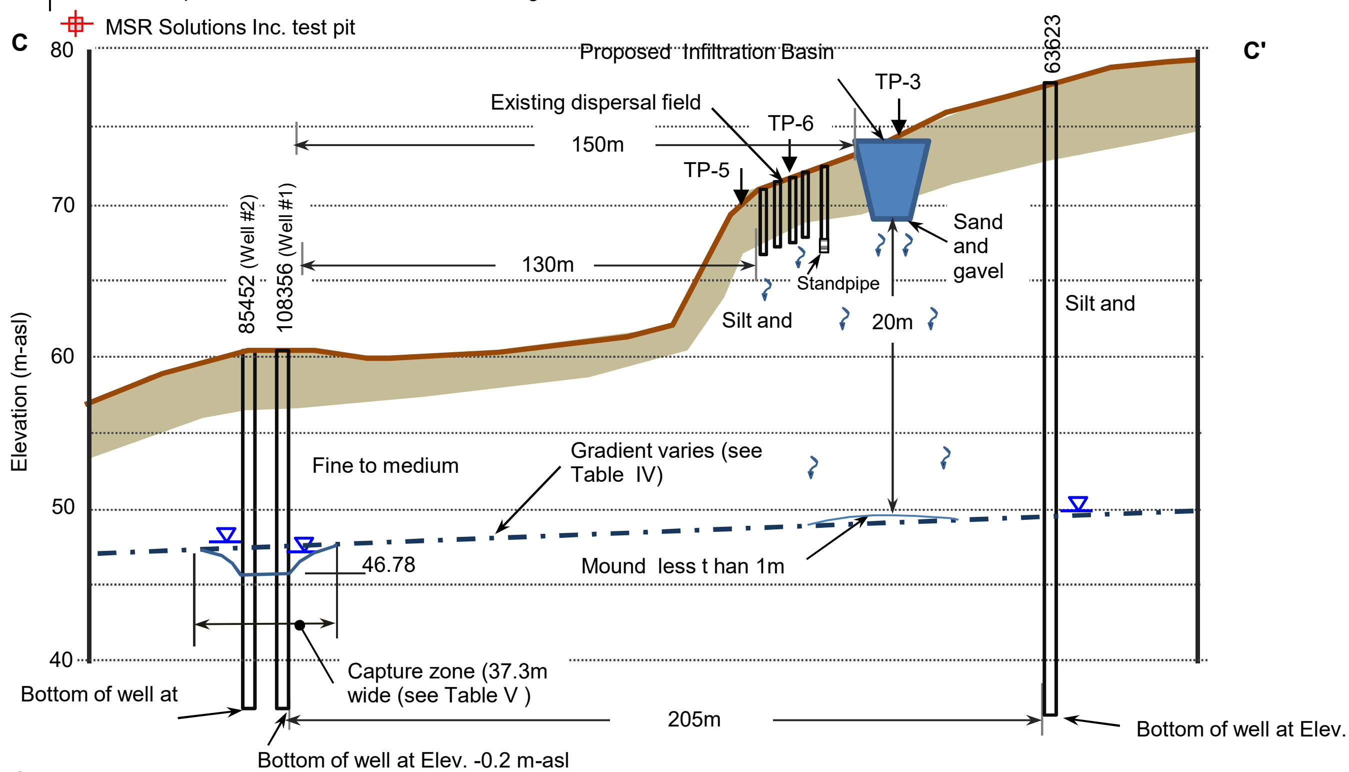
Former irrigation well with indicated well tag number.

Former production well with indicated well tag number.

MSR Solutions Inc. test pit

Scale (1:2,000)

0 30 60m



NOTE
1) Base map based on Google Earth and see location on Figs 1 and 2.
2) See details of the Infiltration basin on Figs 6 and 7.

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Map of Property and Section C - C'

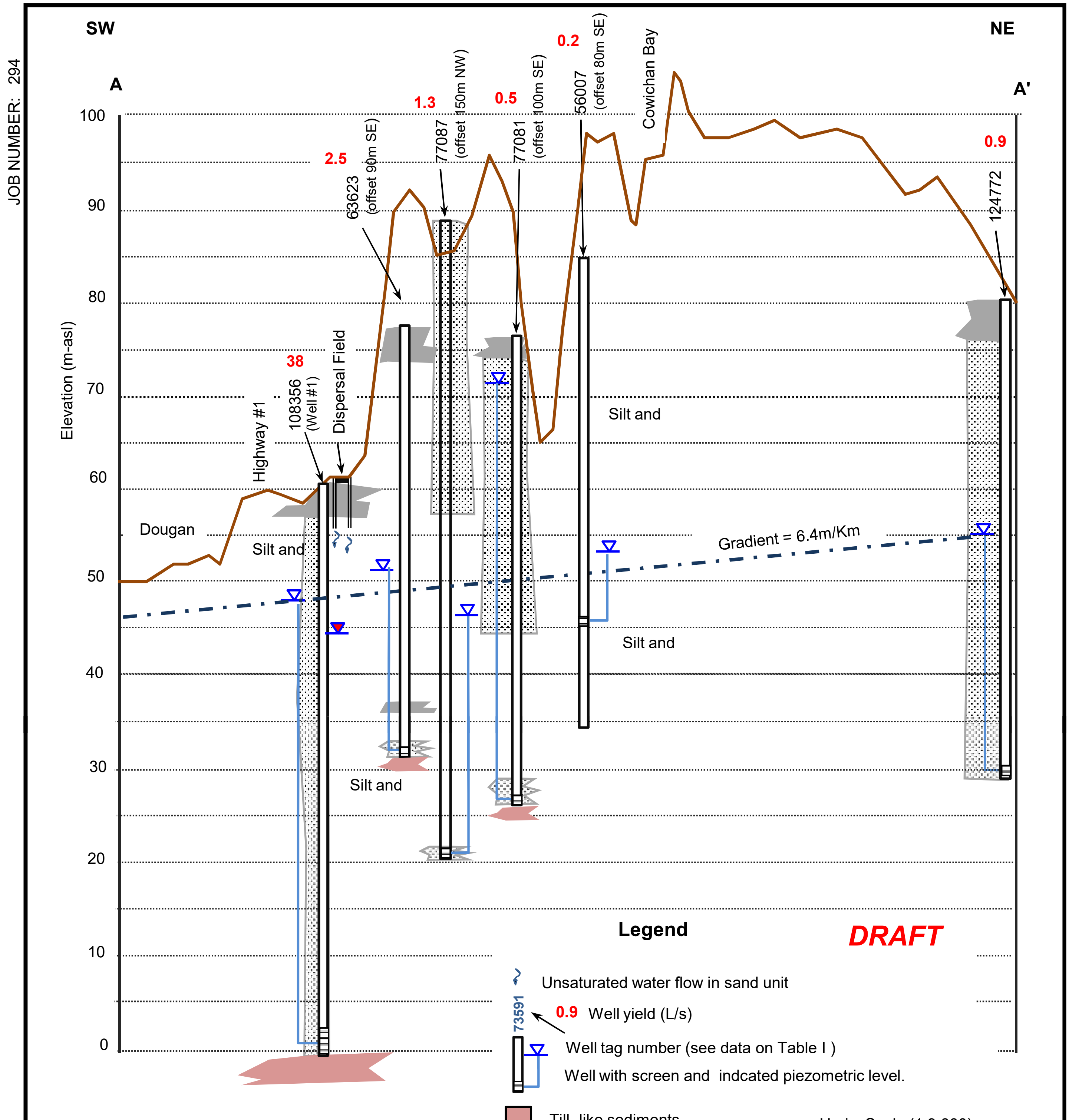
Drawn:

Date Feb. 2023

Approved:

Fig. 3

File:2600 \ 294 Figures 3 site map 2/13/2023 5:05 PM



NOTES
1) See location of section on Fig.3 and data on wells on Tables I and II.

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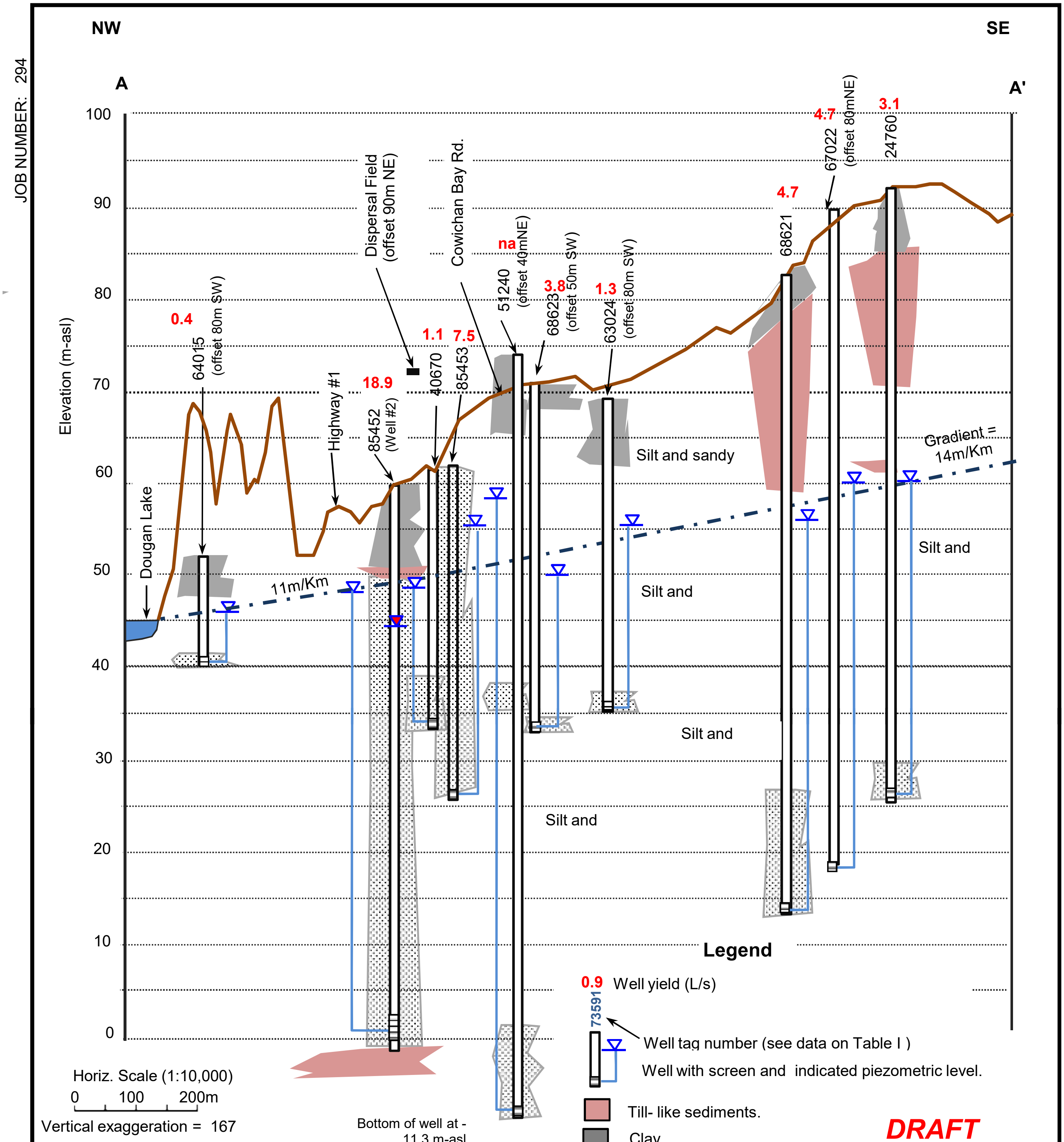
Assessment of Impact of Dispersal Field on Local Aquifer. Valleyview Shopping Centre, 1400 Cowichan Bay Road, Cobble Hill, B.C.	**HYDROGEOLOGICAL SECTION A - A'**	Drawn:	Date Feb. 2023
		Approved:	Fig. **4**

File:2600 \ 294 Figures 4 Section A-A 2/13/2023 5:06 PM



NOTES
1) See location of section on Fig. 2 and data on wells on Tables I and II.)
2) Groundwater divide is located 1.25Km to southeast.

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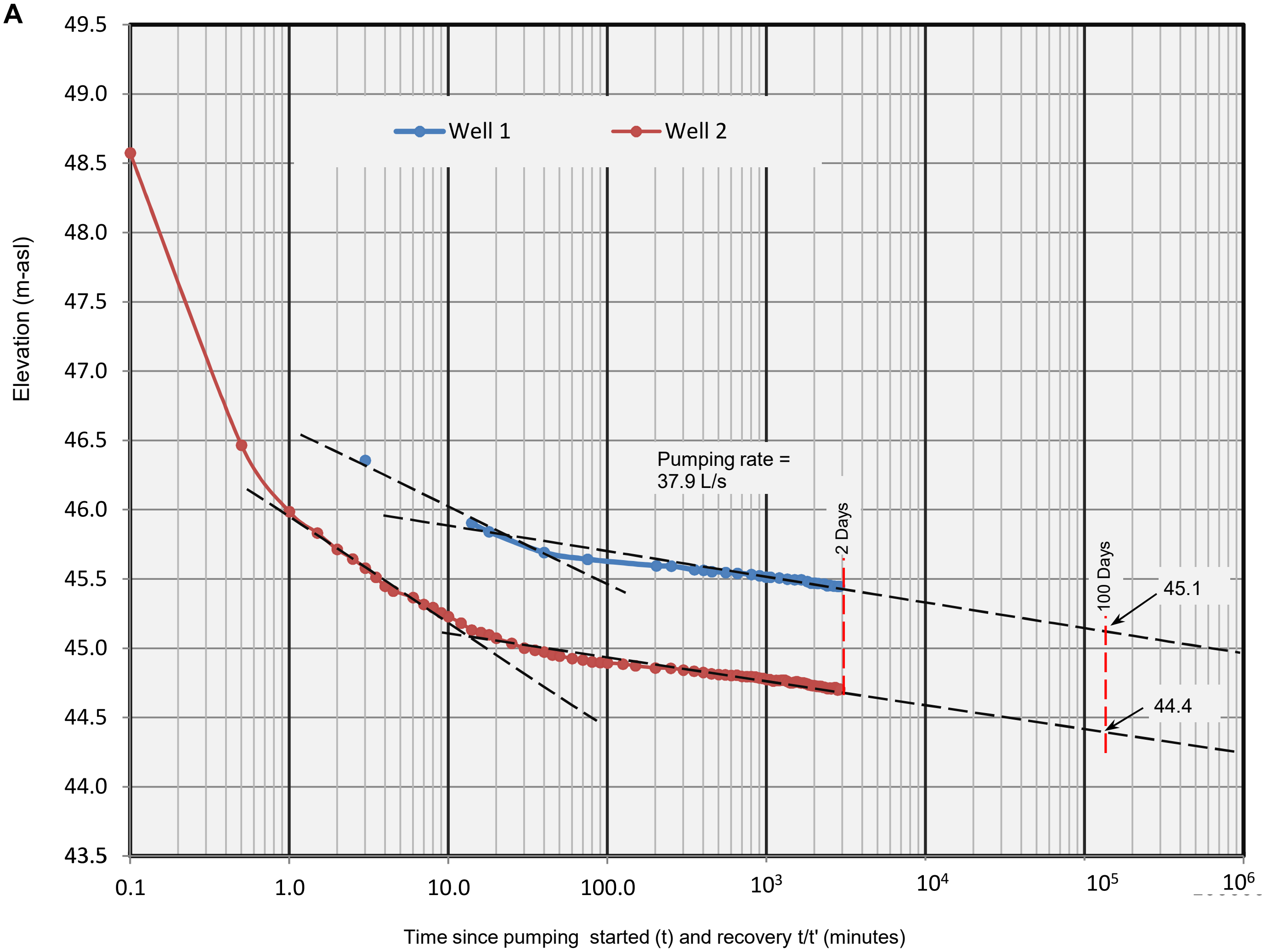
		Approved:	Fig. **5**

File:2600 \ 294 Figures 5 Section B-B 2/13/2023 5:08 PM

JOB NUMBER : 279

A



	Drawdown	Sp Cap L/s/m	L/s
Tested	4.09	9.267	37.9
Current	0.5	9.267	4.60
Future	1.2	9.267	11.3

Transmissivity = 1.83 * 10^{-4} * Q / ΔS

Leg 1	Q (L/s)	ΔS (m)	T (m^2/s)
Well 1	37.9	0.75	9.2E-03
Well 2	37.9	0.55	1.3E-02
	3,275	m3/d	

Static water level = 48.49

Elevation at 100 days = 44.40

Drawdown = 4.09

Leg 2	Q (L/s)	ΔS (m)	T (m^2/s)
Well 1	37.9	0.17	4.1E-02
Well 2	37.9	0.17	4.1E-02

NOTES
1) See location of well on Fig s 2 & 3 and profile on Fig. 5.
2) Pump tests data from Thurber Engineering Ltd., 2013..

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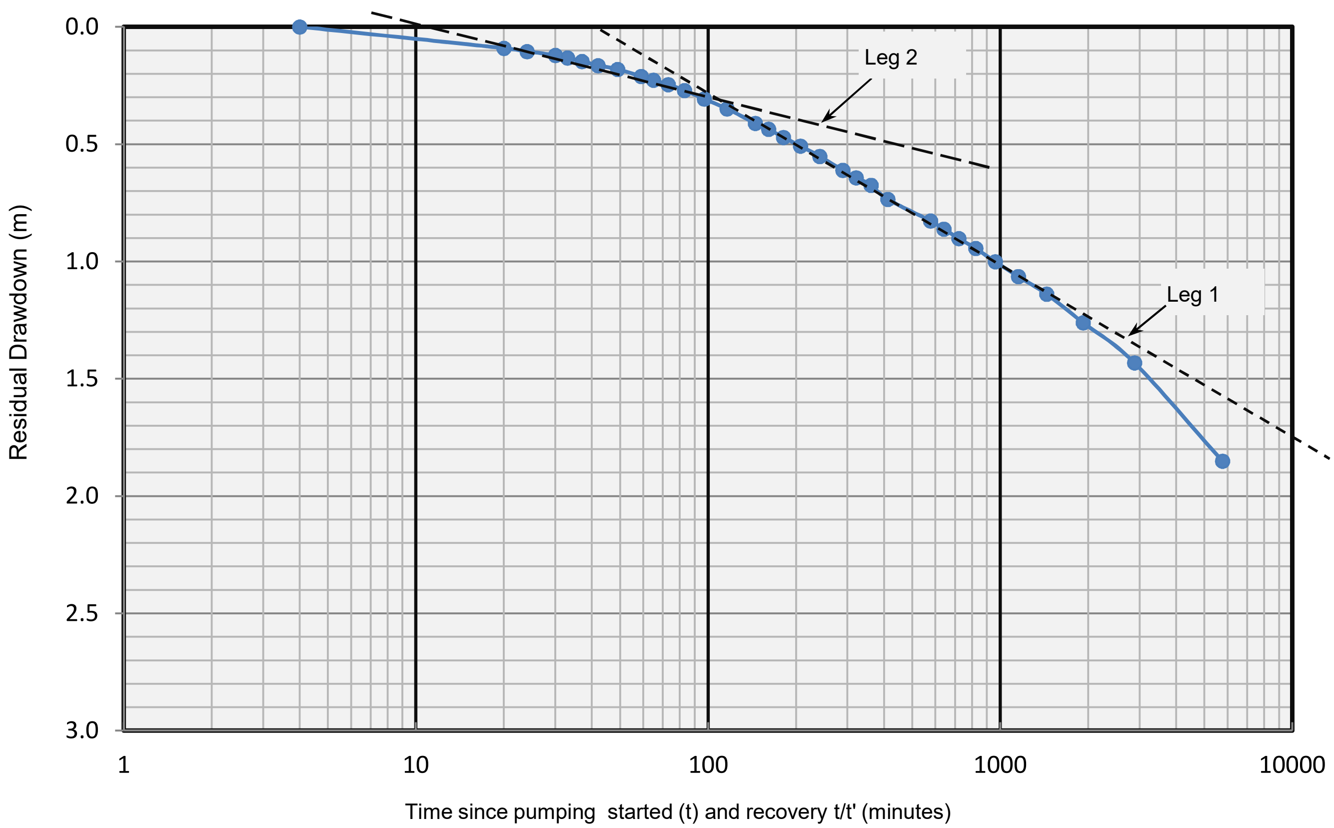
Assessment of Impact of Dispersal Field on Local Aquifer. Valleyview Shopping Centre, 1400 Cowichan Bay Road, Cobble Hill, B.C.

Water Levels in Pump Tested Well 1

Drawn:
Approved:
Date Feb. 2023
Fig. **6**

File:2600 \ 294 Figures 6 Well 2 Test 2/13/2023 5:11 PM

JOB NUMBER : 279



Transmissivity = 1.83 * 10^{-4} * Q / ΔS

	Q (L/s)	ΔS (m)	T (m^2/s)
Leg 1	37.9	0.73	9.5E-03
Leg 2	37.9	0.3	2.3E-02
	3274.6	m3/d	

Aquifer thickness 45 m

Leg 2 hydraulic conductivity 5.1E-04 m/s

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NOTES
1) See location of well on Fig 2 & 3 and profile on Fig. 5.
2) Pump tests data from Thurber Engineering Ltd., 2013..

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Water Level Recovery in Pump Tested Well 1

Drawn: RO

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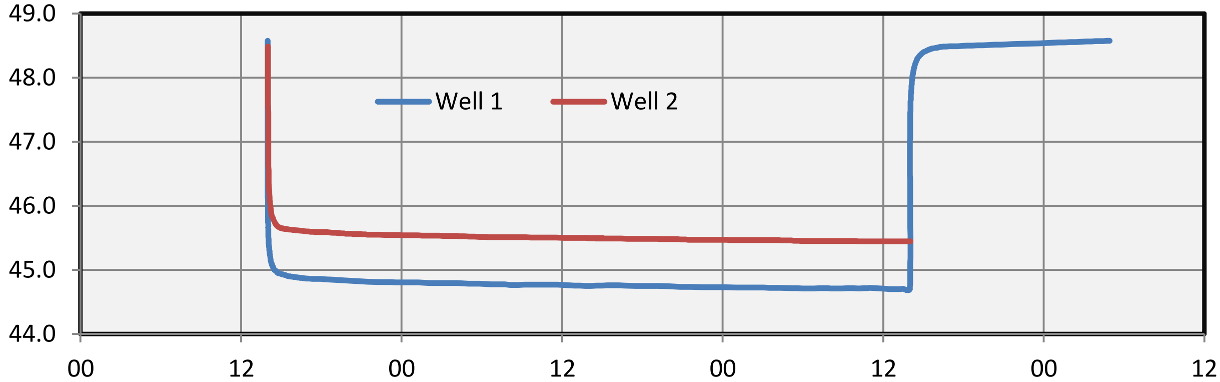
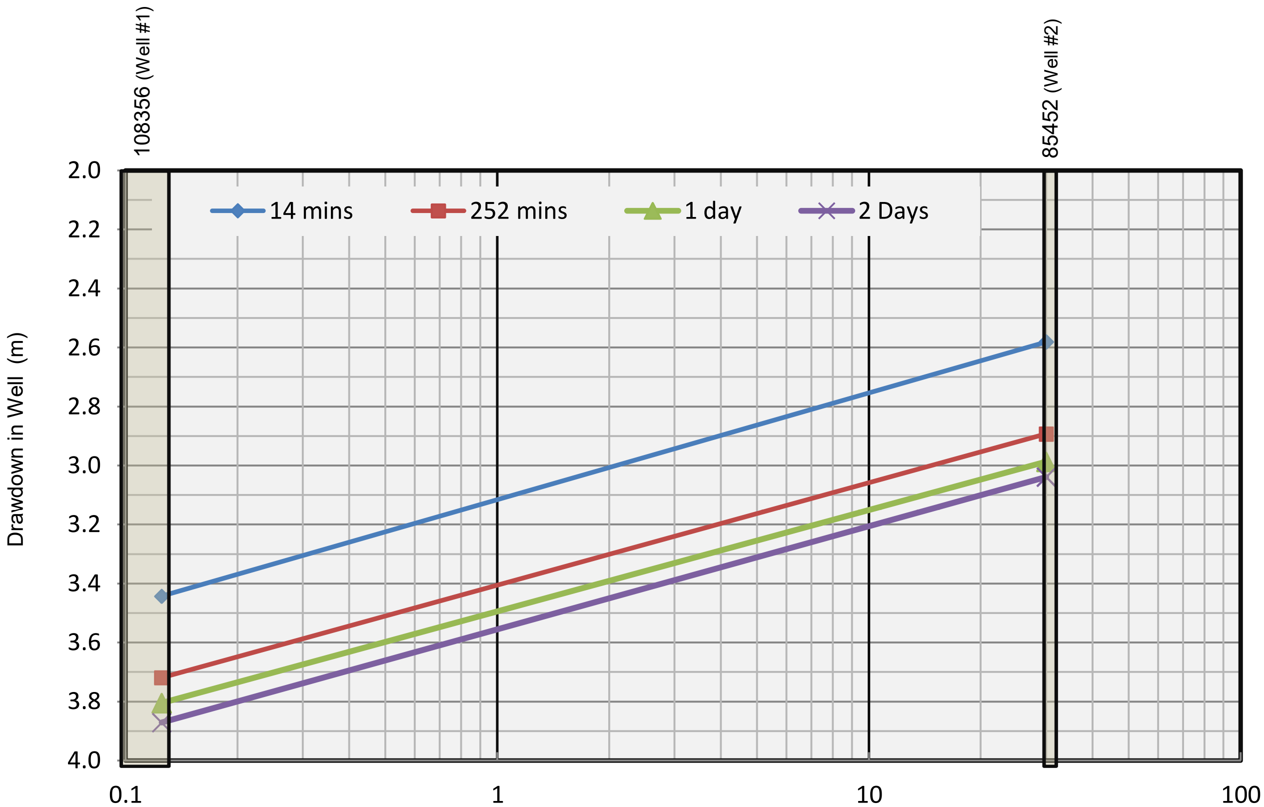
Date Feb. 2023

Fig. **7**

JOB NUMBER: 294





NOTES
1) See location of well on Fig 2 & 3 and profile on Fig. 5.
2) Pump tests data from Thurber Engineering Ltd., 2013..

Estimates for fracture zone transmissivity

Transmissivity = 3.66 * 10^{-4} * Q / ΔS

Aquifer thickness (m) 45

	Q (L/s)	ΔS (m)	T (m^2/s)	K (m/s)
14mins	37.9	0.37	3.7E-02	8.3E-04
252 mins	37.9	0.34	4.1E-02	9.1E-04
1 Day	37.9	0.33	4.2E-02	9.3E-04
2 Days	37.9	0.35	4.0E-02	8.8E-04

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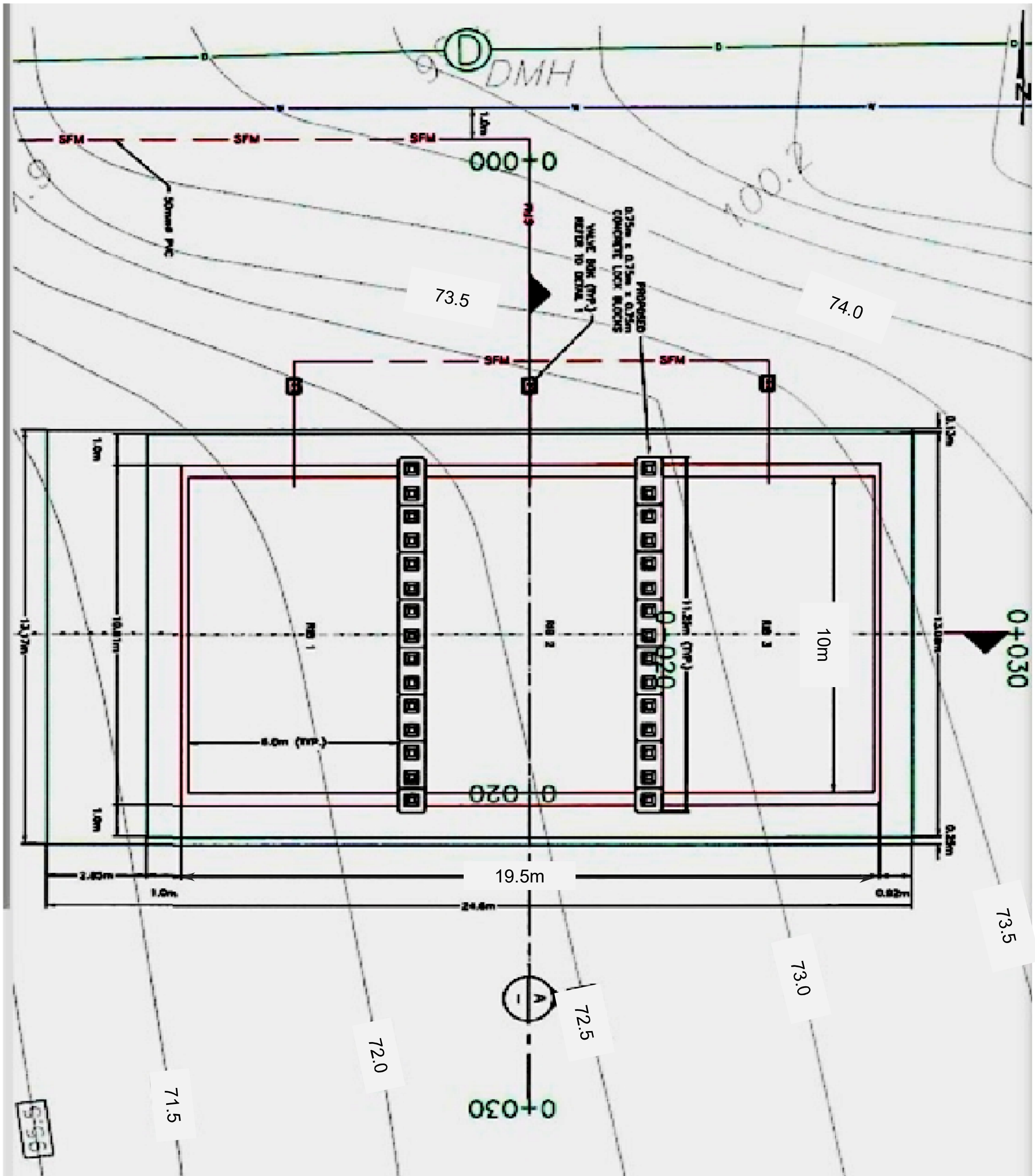
Assessment of Impact of Dispersal Field on Local Aquifer. Valleyview Shopping Centre, 1400 Cowichan Bay Road, Cobble Hill, B.C.

Inter-well Distance Drawdown

Drawn: | Date Feb. 2023
Approved: | Fig. **8**

JOB NUMBER: 294



Ground elevations (m-asl)

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NOTES
1) See location of basin on Fig. 3 and profiles on Fig. 10.

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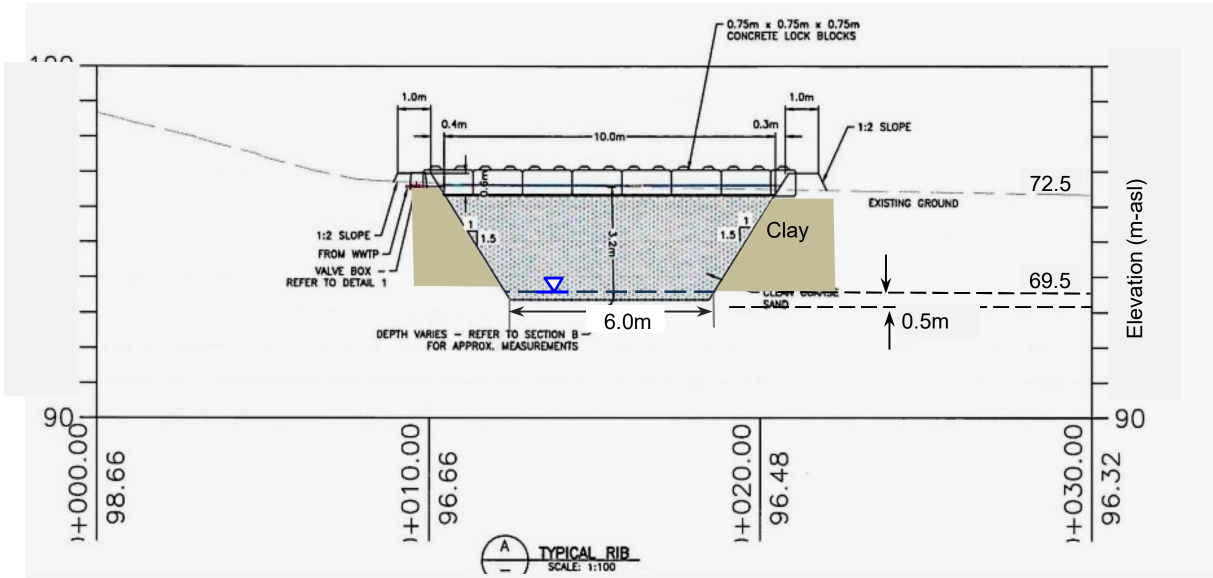
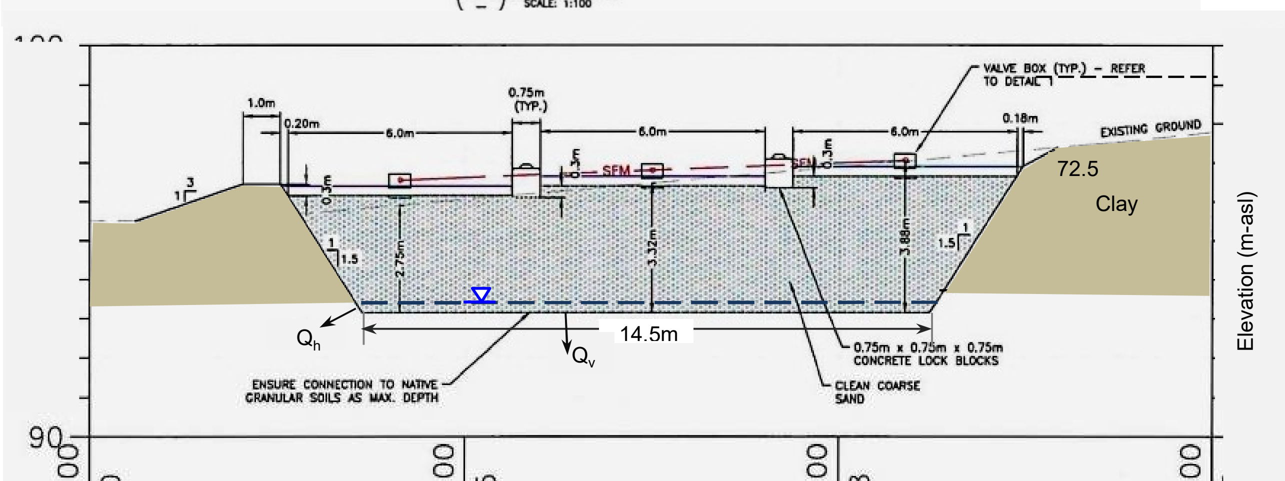
Assessment of Impact of Dispersal Field on Local Aquifer. Valleyview Shopping Center, 1400 Cowichan Bay Road, Cobble Hill, B.C.	**Rapid Infiltration Basin**	Drawn:	Date Feb. 2023
		Approved:	Fig. **9**

JOB NUMBER: 294





Hydraulic loading at base of sand bed

Horizontal seepage Q_h

Parameter	Quantity	Units
Perimeter	41	m
Width	1	m
Area	41	m^2
Porosity	30%	
Kh	1.5E-04	m/s
Gradient	0.379	
Flow (Q_h)	7.0E-04	m^3/s
	61	m^3/day
HLR	1,478	mm/day

Vertical seepage Q_v

Parameter	Quantity	Units
Bottom area	87	m^2
Porosity	30%	
Kfs	13.0	m/day
	1.5E-04	m/s
Kv	3.0E-05	m/s
Gradient	0.379	
Flow (Q_v)	3.0E-04	m^3/s
	25.7	m^3/day
HLR	296	mm/day

Gross loading at bed bottom (mm/d) = 989

Estimated discharge = 86 m^3/day

Design maximum discharge = 86 m^3/day

NOTES
1) See location of basin on Fig. 3 and plan on Fig. 9
2) Drawings from MSR Solutions Inc . 2022.

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Rapid Infiltration Basin Profiles

Drawn:

Date Feb. 2023

Approved:

Fig. **10**

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