May 1, 2015
File No. 89220
Via Email and U.S. Mail

Town of Carlisle, Zoning Board of Appeals
ATTN. Steve Hinton
66 Westford Street
Carlisle, MA 01741
shinton@mindspring.com

Re: Phase 2 Report
Independent Hydrogeologic Study
100 Long Ridge Road, Carlisle, MA

Dear Mr. Hinton:

Nobis Engineering, Inc. (Nobis) is pleased to present this Report to the Town of Carlisle Zoning Board of Appeals (”Town”) as part of Phase 2 of an independent hydrogeologic study of potential impacts related to a proposed 40B housing development on the Brem property located at 100 Long Ridge Road (a.k.a. “The Birches”) in Carlisle, Massachusetts (“Site”). (See Figure 1.) The Site is Carlisle tax lot 1-72-33K, with the recent subtraction of a lot for a new home at 90 Long Ridge Road. This Report is the primary deliverable item for Phase 2 of the project under Nobis’ contract with the Town, dated January 13, 2015 and signed by the Town on January 14, 2015 (Town of Carlisle ZBA document # BREM_151 01.14.2015), with an amendment dated March 19, 2015 and signed by the Town on March 23, 2015. The Scope of Work for Phase 1 is included in the contract, and the Scope of Work for Phase 2 is included in the amendment.

Nobis understands that the Site is proposed for development by a private owner, Jeffrey Brem of Lifetime Green Homes, LLC (LGH), and that the Town’s concerns include potential impacts of proposed on-Site wastewater disposal systems on proposed on-Site and existing off-Site drinking water wells, and potential yield and water level interference effects between the new wells and the existing nearby wells. Also, potential interference effects between the proposed new wells are a concern.

1.0 BACKGROUND

LGH submitted a 40B housing approval application (“LGH application”; BREM_001 07.03.2014) to the Town on July 2, 2014. Nobis has received updated Site plans from the Town (prepared by LGH, dated November 14, 2014; BREM_127 12.08.2014 and dated March 27, 2015; BREM_197 03.31.2015) which depict 19 proposed residences and one existing residence to be served by a septic system consisting of three proposed septic disposal areas, the existing house septic system, and eleven new drinking water wells (Figure 1). Nobis understands that the existing well that serves the present house will be used for irrigation. Neighboring homes, including abutters, are served by existing bedrock domestic drinking water wells (Figure 3).

Nobis understands that the Town has retained Nitsch Engineering of Boston, MA and GeoHydroCycle, Inc. (GHC) of Newton, MA as peer reviewers for the LGH application and related

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submittals. Nobis further understands that Site property abutters retained Hill Law of Cambridge, MA and the Horsley Witten Group of Sandwich, MA to review the LGH application and related materials. Nobis has reviewed key submittals by these attorneys and consultants and by LGH and its hydrogeologic consultant, Northeast Geoscience Inc (NGI). The submittals are available on the Town's website for the project.

Nobis understands that the objectives of the independent hydrogeologic study include assessing potential impacts of the following:

- Proposed Site septic system systems on proposed Site and existing drinking water wells at abutting Site properties
- Proposed drinking water wells on existing drinking wells at abutting properties;
- Proposed drinking water wells and on each other; and,
- Evaluation of actions intended to reduce the risks of impacts to acceptable levels.

Additional technical objectives have been developed during the course of the project and include developing a hydrogeologic conceptual site model as a framework for understanding groundwater in the overburden and bedrock in the vicinity, conducting groundwater mounding analyses, and conducting nitrate mass balance and dispersion analyses.

Nobis completed Phase 1 of an independent hydrogeologic study and submitted a Phase 1 Report, dated February 20, 2015. In the Phase 1 study, Nobis reviewed background hydrogeologic information and submittals to the Town regarding the project between July 2014 and February 2015. Nobis also conducted a Site walk, accompanied by the Town and LGH, on the LGH property on January 23, 2015 and in the neighborhood, accompanied by the Town and Thornton Ash, owner of a nearby property, on the same date. During the Site walk, general Site conditions and topography, bedrock outcrops, and well locations were observed, and the bedrock outcrops were photographed. No borings, test pits or sample collections were performed during Phase 1. Nobis also conducted a photolineament analysis, which indicated that several weak but persistent photolineaments are present in and around the Site; the most prevalent photolineament orientation is northwest-southeast.

Nobis' Phase 1 Report also listed several data requirements to achieve the hydrogeologic objectives described above. The data requirements were grouped into four categories:

- Overburden Hydrogeologic Characterization,
- Bedrock Hydrogeologic Characterization,
- Groundwater Mounding Analysis, and
- Nitrate Loading Analysis.

As presented in Nobis' Phase 2 proposal dated March 19, 2015, the objective of the Phase 2 study is to conduct investigations to meet data requirements and analyses that either have not been proposed or agreed to by LGH in communications with the Town. Additional analyses may be conducted by Nobis to reconfirm to the results of analyses that may be performed by LGH.

Subsequent to Nobis' Phase 2 proposal, NGI submitted a hydrogeologic report entitled "Groundwater Impact Analysis, Brem Property, 100 Long Ridge Road, Carlisle, MA", dated March

25, 2015 (NGI Report). Evaluating or critiquing the NGI report is not one of Nobis' Phase 2 objectives; rather, Nobis' Phase 2 work is intended to:

- Use information provided by NGI where appropriate;
- In some cases, conduct independent analyses of elements also addressed by NGI, using different methodologies or assumptions where appropriate; and
- In some cases, conduct investigations not undertaken by others.

Recommendations regarding approval or denial of LGH's permit application(s) is outside the scope of this Phase 2 report.

2.0 PHASE 2 RESULTS

2.1 Hydrogeologic Conceptual Site Model

2.1.1 Bedrock Geology

Mass GIS bedrock geologic mapping was included in Nobis' Phase 1 Report, Attachment C4 and shows two primary lithologies in the area. Nearly all of the Site (and all of the area for the proposed development) is underlain by mafic rocks. The mafic rocks are dark rocks rich in magnesium and iron that may have had a volcanic origin, although they have since been metamorphosed (see more description below and in Attachment 1). Granite, a quartz-rich igneous rock, is mapped beneath the extreme southeastern corner of the Site. The bedrock lithology is important because lithology can influence fracturing, and thus transport pathways in the rock, and groundwater quality in the bedrock and in drilled bedrock wells.

Bedrock mapping by Nobis during Phase 2 indicates that the Site is underlain by metamorphic bedrock consisting of biotite schist with orange feldspar and hornblende (typical of mafic rock); iron oxide (rusty) weathering is present in portions of the outcrops observed by Nobis. Schistose foliations are prominent in one of three outcrops observed by Nobis; in the other two outcrops, the rock is more massive and has a blocky appearance, with some dark, amphibolitic (mafic) rock observed. Granitic rock outcrops were not observed. Outcrop descriptions from Nobis' bedrock fracture mapping are provided in Attachment 1, and the outcrop locations are shown on Figure 3. Outcrop 1, located on Site, east of the brook, is defined by steep metamorphic foliations that trend northeastward and are sometimes coincident with open fractures. The outcrop is segmented into three different sections, aligned southwest to northeast. Outcrops 2 and 3 are road cuts located south of the Site.

On April 3, 2015, Nobis measured bedrock fractures at the three outcrops described above. The purpose of the investigation was to characterize the bedrock and assess the orientation of bedrock fractures that appear capable of transporting groundwater flow. This information is intended to enhance the hydrogeologic conceptual site model for the LGH site and surrounding area and is important because all of the existing and proposed water wells in the area are completed in bedrock and draw their water from bedrock fractures.

Nobis measured the orientation (strike and dip) of all observed fractures in these outcrops that appeared potentially capable of conducting groundwater flow. A description of these measurements and the terminology is found in Attachment 1. In summary, steep, northeast-striking fractures are the most commonly observed fractures in these outcrops, whether the rock is prominently foliated or massive. Steep, northwest-striking fractures are somewhat common in the non-foliated rocks, but only rarely observed in the foliated rocks. Scattered fractures with shallower dips and other strike directions are also present. These results differ somewhat from the results of photolineament analysis, documented in Nobis' Phase 1 report (see also Figure 3). The most common photolineament trend is northwest. Based on the observation that some steep, northwest-striking fractures are present in the outcrops, the photolineaments may represent fracture traces, and the differences in predominant orientations between photolineaments and outcrops may be scale related.

Based on the photolineament and outcrop data, northeast and northwest appear to be the most likely directions for groundwater flow in fractured bedrock in the area. This means that sensitive receptors that occur to the northwest, southeast, northeast, or southwest of one of the Proposed Septic Disposal Areas are more likely to be impacted by discharges to the septic system than those that lie in other directions. Similarly, proposed and existing wells that are aligned in these predominant fracture or lineament directions are more likely to interfere with each other when pumping. However, actual effects are highly dependent on the characteristics of particular fractures that supply water to the wells, and specific predictions regarding impacts cannot be made with the present level of information.

2.1.2 Overburden Geology

Based on Mass GIS surficial geologic mapping, included in Nobis' Phase 1 Report, Attachment C2, much of the site is characterized by "abundant outcrop and shallow bedrock", with swamp deposits along the northern boundary. Mass GIS wetland mapping (Phase 1 Report, Attachment C3) shows wetlands along the northern edge of the site (similar location as the mapped swamp deposits) and in the far southeastern corner of the site. On-Site mapping by LGH (BREM_127 12.08.2014) shows that these two wetlands are connected by a brook that flows southward and then southeastward and is flanked by a narrow strip of wetlands.

Based on test pit and boring logs for the Site (BREM_147 01.08.2015; BREM_146 01.08.2015; and the NGI Report (BREM_193 03.25.2015)), soils at the Site include loamy sand, loamy gravel, sandy loam, fine sand and silt, medium sand, and traces of gravel and clay. Soil colors noted in boring logs include gray, dark gray, and tan. Soil samples collected by the Town and Nobis on April 3, 2015 (see below) include dark brown and yellow brown stony fine sand, silt, and clay with organic material. The soils are likely formed from glacial till, deposited directly by glacial ice. Two of seven borings installed by NGI (NGI Report, Appendix A) encountered weathered rock beneath glacial till but above competent bedrock. The weathered rock deposits range in thickness from 7 to 14 feet. The overall thickness of soil/overburden (depth to bedrock) at the Site varies from zero at Outcrop 1 to 24 feet at MW-4-15, including weathered rock where it was noted. Test pits range in depth from 4.75 feet to 11 feet, indicating that bedrock is at these depths or deeper at these locations.

Nobis has also reviewed existing records on abutters' wells and other wells in the area, provided by the Carlisle Board of Health and in a letter report by NGI entitled "Groundwater Impact Analysis" and dated September 15, 2014 (BREM_080 10.14.2014). All of the wells known in the area are drilled into bedrock, with casing extending through the overburden and several feet into bedrock. The total depths of the wells are between 110 feet and 710 feet, with depth to bedrock (overburden thickness) ranging between 4 and 25 feet in the area. Outcrops are present; therefore, overburden thickness is zero in some places, so the overall range is 0 to 25 or more feet. Information compiled on wells in the area is shown on Figure 3.

On April 3, 2015, the Town and Nobis collected soil samples at two locations, near Proposed Septic Area 1 and MW-1-15 (sample SS-1) and east of Proposed Septic Area 2 and MW-3-15 (sample SS-2). Sieve analyses of these samples were performed to obtain hydraulic conductivity estimates, and the samples were also analyzed for total organic carbon. The samples were collected from depths of 21.5 and 23 inches by hand digging and were sent to GeoTesting Express in Acton, Massachusetts for analysis. Results (Attachment 2) show that the soils consist of "moist, dark yellowish brown silty sand" (SS-1, near Proposed Septic Area 1) and "moist, light olive brown clayey sand with gravel" (SS-2, east of Proposed Septic Area 2). The range of grain sizes, including a high fines (silt and clay) content and significant gravel content made an accurate estimate of permeability impossible. GeoTesting Express estimated permeability (hydraulic conductivity) at between 5.0 x 10E-3 cm/sec and 1.0 x 10E-5 cm/sec (14.2 ft/day and 2.8 x 10E-2 ft/day respectively). The upper end of this range is consistent with hydraulic conductivity values obtained by NGI from slug tests (see below). The two samples also contained 1.7% and 3.1% organic matter.

2.1.3 Hydrogeology

Groundwater occurs in both fractured bedrock and in overburden beneath the Site and vicinity; however, overburden is absent where the bedrock outcrops at the surface, and overburden is presumably thin and unsaturated in the immediate vicinity of bedrock outcrops. Groundwater in the overburden is unconfined; no areally extensive clay or other confining deposit is known to be present above the saturated zone in the overburden. The potentiometric surface (unconfined water table) in the overburden ranged in depth from zero at the brook to 5 to 9 feet below ground surface (ft bgs) in monitoring wells at the Site on January 23, 2015, when measured by NGI (NGI Report, Table 1). Depth to water can be expected to vary with season and weather. Logs of test pits installed at various times (BREM_147 01.08.2015; BREM_146 01.08.2015) show depths to water ranging from 5.5 to 10 feet. Assuming that the brook is an expression of the groundwater potentiometric surface, depth to the water table can be considered to be zero at the brook.

Metamorphic bedrock beneath the site is crystalline and can be expected to have near-zero porosity and permeability (hydraulic conductivity) in the rock matrix. Groundwater flow can only occur in open fractures in the bedrock. Predicting the exact groundwater flow paths in fractured rock and the flow of water to wells completed in fractured bedrock is difficult and highly site specific. It is generally safe to assume that water flowing through bedrock or to a well completed in bedrock is ultimately recharged from overburden above the bedrock, except where bedrock is exposed at the surface. The degree of hydraulic connection between overburden and bedrock at the Site is not characterized as of this writing. Logs for eight monitoring wells drilled at 5 locations by NGI (NGI Report, Appendix A) indicate "refusal" at the bottom of each boring, but it is not always clear whether such refusal occurred because bedrock has been reached. "Competent bedrock" is noted at the bottom of MW-4-15 and MW-5-15, but the geologic reason for refusal at the other sites is less clear. Generally, refusal can be caused by bedrock, boulders or cobbles, or dense glacial till (hardpan) and may be subject to the drilling method. A dense glacial till, where present, can provide a hydraulic barrier between overburden and bedrock.

Reported well yields in the area are relatively high for most of the wells, and the yields range from 0.75 gallon per minute (gpm) (35 Suffolk Lane in a report at the BOH) to 100 gpm (148 Stony Gate, per Table 1 in the 9/15/14 NGI report (BREM_080 10.14.2014)). Reported yields of 10 – 20 gpm are common, suggesting that the bedrock in the area is generally sufficiently fractured to transmit significant quantities of water, leading to drilling success. One exception appears to be 35 Suffolk Lane, located east of the site (Figure 3); two drilling attempts were apparently needed to obtain a well with sufficient water. The first well was drilled to 710 feet and produced only 0.75 gpm. A second well was drilled to 200 feet and obtained 10 gpm. This also illustrates the high degree of local variability in the bedrock and its ability to transmit and produce water. Nobis notes that yields reported in domestic well drilling records (Figure 3) are typically airlift yields conducted by the well driller for up to an hour immediately following drilling. Pumping tests to determine sustainable yield are not always performed on domestic bedrock wells. The sustainable yield for such a well is nearly always significantly less than the reported airlift yield. Also, it is currently unknown whether any of the existing wells interfere with each other.

The existing well on the site is rated at 20 gpm and is 150 feet deep, with the depth to bedrock 8 feet. This well currently serves the existing home on the site, but this well has been proposed to be used for irrigation, and 11 new wells are shown on the November 14, 2014 map (BREM_127 12.08.2014), to serve the existing home and the 19 proposed homes. Most wells will serve two homes each, with two of the wells only serving one home each.

Water quality information is available for some of the wells in the area, and elevated concentrations of dissolved iron in the water are fairly common. Elevated iron levels are generally considered an esthetic concern, not a health concern. Elevated radon, which has health implications but no maximum contaminant level set by the U.S. Environmental Protection Agency, is noted in two of the wells. Nobis has compiled well information obtained from the September 2014 NGI Report (BREM_080 10.14.2014) and the Carlisle Board of Health; the information is presented on Figure 3.

Nobis has plotted groundwater levels in overburden monitoring wells and two staff gauges in the brook measured by NGI on January 23, 2015 (NGI Report, Table 1). Nobis notes that where a pair of monitoring wells exists at the same location (MW-1-15 and MW-1A-15; MW-2-15 and MW-2A-15; and MW-3-15 and MW-3A-15), water elevations, referenced to a local benchmark, not sea level, calculated by NGI based on the water level measurements varied between 0.03 and 1.04 feet between members of the well pairs. Because the NGI report does not offer an explanation for the differences nor an opinion as to which water level in each pair is more accurate, Nobis averaged the water elevations to obtain a data point at each location. The significant discrepancy in water levels between MW-1-15 and MW-1A-15 lend uncertainty to the groundwater contours and gradients in the vicinity of Proposed Septic Disposal Area 1. This in turn could affect the predicted depth to the top of the groundwater mound at this Area and could also affect the dispersion calculations in this area (see Section 2.4).

In addition to the five monitoring well locations, water levels at staff gauges SG-1 and SG-2, located in the brook are assumed to represent the water table (potentiometric surface) at these locations. The NGI Report (Table 1) does not present a water elevation at wetland piezometer PZ-1, located in the wetland that is drained by the brook and that is north of the Site in the adjacent property (NGI Report, Figure 2). Therefore, seven data points are available for contouring the overburden groundwater potentiometric surface on January 23, 2015.

Nobis' interpreted overburden groundwater potentiometric surface contours, based on NGI's measurements, as described above, are shown in Figure 1. Between and beyond the seven locations with water elevation data, Nobis assumed that the potentiometric surface parallels topography; this is a standard assumption in unconfined conditions for a shallow water table in areas where there are no direct water level measurements. The contours are dashed where they are estimated in areas beyond data points.

In a porous medium such as the soil and sandy glacial till that underlies the Site, groundwater flows from areas with higher head to areas of lower head. Flow lines are perpendicular to potentiometric surface contours. In the vicinity of proposed Septic Area 1, groundwater flow can be expected in the easterly or southeasterly directions, based on the groundwater contours shown on Figure 1. However, based on topography, a component of groundwater flow may occur to the south and southwest as well. These flow arrows are shown with question marks on Figure 1. Installation of a new monitoring well or wells in this area would be needed in order to characterize shallow groundwater flow in these directions more accurately. In the vicinity of proposed Septic Disposal Areas 2 and 3, the inferred groundwater flow direction is east-northeast or northeast. Because no water elevation data point (monitoring well) is located in this direction, this groundwater flow direction is somewhat uncertain, but is the most likely, based on the available measurements combined with topography. Installation of a new monitoring well or wells in this area would be needed in order to verify or discount shallow groundwater flow in these directions.

Nobis notes that proposed Septic Disposal Areas 3 and 2 (with monitoring wells MW-2/2A and 3/3A) are located in an area that is currently used for horseback riding and has been leveled by excavation in the western portion (proposed Septic Disposal Area 3) and by emplacement of fill east of proposed Septic Disposal Area 2. It is unknown whether these alterations of the natural grade affect the groundwater potentiometric surface in this area, but it is possible that these changes may influence local groundwater flow and the potentiometric surface (including groundwater mounds after the septic system is in operation). During the April 3, 2015 field trip, Nobis noted a green plastic drain pipe discharging water from the base of the fill near the northeastern corner of the horseback riding area (near SS-2 on Figure 1). If this drainage infrastructure remains in place, it may significantly influence the direction of transport of wastewater that may be discharged at the proposed Septic Disposal Areas.

The groundwater potentiometric surface, as interpreted by the contours and groundwater flow arrows in Figure 1, represents current conditions. After construction and when the septic system is in operation, groundwater mounding is expected at the proposed septic disposal areas, locally changing the groundwater contours, groundwater flow gradient, and possibly local flow directions. Anticipated groundwater mounding is described in Section 2.2, and resulting potentiometric surface contours are shown in Figure 2. Neither Figure 1 nor Figure 2 shows the effects that well pumping may have on the overburden potentiometric surface.

Hydraulic conductivity is a measure of the capacity of a soil or other geologic deposit to transmit water, although a hydraulic gradient (e.g. sloping water table, pumping stress, groundwater mound) must be present in order to drive flow. Hydraulic conductivity (or permeability) is expressed in units such as feet per day (ft/d) or centimeters per second (cm/sec), but is not the same as velocity. Several methods are used for estimating hydraulic conductivity, which is notoriously difficult to do with accuracy. In practice, agreement between various estimates is considered good if the estimates are within an order of magnitude of each other.

For the LGH Site, hydraulic conductivity has been estimated by NGI using slug tests on the monitoring wells. (A slug test is a short-term method by which the water level in a well is quickly raised or lowered on a one-time basis; the resulting drawdown or recovery of well water levels is measured for several minutes following this event. The water level data can then be analyzed by one of several methods to obtain an estimate of hydraulic conductivity. See the NGI Report text and Appendix B for more explanation of the methods and results.) Nobis also collected soil samples for the purpose of obtaining additional estimates of hydraulic conductivity by a different method. As described in Section 2.1.2 of this report, the results of the soil sample test are not suitable for estimating hydraulic conductivity because of the high proportion of fine-grained particles and also gravels in the samples. However, the high end of the hydraulic conductivity estimates obtained by this method is consistent with NGI's slug test results. For these reasons, Nobis has used the hydraulic conductivity estimates obtained by NGI in our calculations.

For eight wells at five locations, the NGI slug test results suggest that hydraulic conductivity is between 2.08 ft/d and 23.75 ft/d, with a geometric mean of 9.0 ft/d. NGI also recommended additional testing for hydraulic conductivity, by means of pumping tests and/or hydraulic loading tests.

Assessing the water budget for a site or an aquifer can be a useful exercise for understanding the approximate quantities of groundwater that recharge and discharge from the site or the aquifer for a given period of time (e.g. a year) under typical conditions. As with many hydrogeologic analyses, most water budget assessments are very approximate, depend on simplifying assumptions, and provide order-of-magnitude results. For the LGH Site, after the proposed project is constructed and in operation, the primary inputs to groundwater beneath the Site are:

- Recharge from precipitation (rain and snowmelt);
- Existing and proposed septic systems; and
- Groundwater inflow (in the subsurface) from outside the Site; based on topography; some land areas outside of the Site are upgradient of the Site, and some groundwater may flow into the Site from these areas; this input is not included in the present estimate, because it is assumed to be balanced by groundwater flow out of the site to the brook and in the southeastern portion of the Site.
- Other inputs are assumed to be minor.

Groundwater outputs from the Site include:

- Withdrawals from the existing and proposed wells;
- Groundwater outflow (in the subsurface) to the brook or to downgradient areas (see above).

Water that is already stored beneath the surface in pores spaces in the overburden and in fractures in the bedrock can be significant and can dominate the water budget inputs and outputs for an initial period of time after a project is built and begins operation. The amount of groundwater stored beneath a Site can be very difficult to estimate, however.

Water budget inputs from precipitation have been estimated by NGI at 20% of the average annual precipitation; for the Site, this amounts to 8.2 inches per year distributed over the 9.84 acres of the Site, for a total of about 7.8 million liters per year (NGI Report, Table 2), or about 2 million gallons per year. Nobis agrees that 20% is a reasonable assumption for the sandy glacial till deposits at the Site. Nobis also notes that this recharge will be restricted in some portions of the Site by impervious surfaces (driveways and buildings), but that stormwater infrastructure is designed to recharge the stormwater on Site, so this does not reduce the total recharge for the Site.

Estimated design flows for the proposed septic disposal areas (LGH "Computation of Sewage Flows" document dated January 5, 2015 (BREM_145 01.05.2015)) are estimated by LGH at 1,980 gallons per day (gpd) each for the three proposed septic disposal areas (Attachment C). If the Town stipulates that the design flow for the septic system should be 165 gpd per bedroom, as opposed to the design flow of 110 gpd per bedroom, this increases the design flow for each proposed septic disposal area to 2,970 gpd. The existing home is assumed by Nobis to remain on its existing septic system, whose design flow is unknown, but is probably 660 gpd, based on 4 bedrooms at 165 gpd per bedroom. Nobis notes that because neither BREM_145 01.05.2015 (Attachment C) nor the "Plan & Profile and Utility Plan (BREM_128 12.08.2014) show the existing home (#20) as connected to the new septic system, LGH apparently intends to keep the existing home on its existing septic system. Not including the septic system for the existing home, proposed septic disposal areas 1 – 3, designed to serve the 19 new homes, will have a combined design flow of 5,940 gpd (8,910 gpd using Town design flow). If 660 gpd is added for the existing home, the total design flow for the new and existing septic systems, combined, could be as high as 9,570 gpd. This is equivalent to about 3.5 million gallons per year. The yearly estimate using LGH's design numbers would be 5,940 gpd plus discharge from the existing home; if 660 gpd is used for the latter discharge, the total yearly input to groundwater would be 2.4 million gallons per year. Nobis notes that this is greater than the 2 million gallons per year assumed to recharge the Site from precipitation (see above).

Presumably, LGH anticipates that the combined withdrawals from the 11 proposed wells, in aggregate and not including the existing well, proposed to be used for irrigation and therefore not discharging to a septic system and not including a well to supply the firefighting cistern, should be capable of meeting the design flow discussed above. It is assumed that most of the water pumped from the wells (except water used for watering lawns or gardens, the intended use for the well that serves the existing house) will be discharged to the septic systems. Clearly, the actual available yields of the individual new wells cannot be known until the wells are drilled. Also, actual water use by future residents should be less than the wastewater design flows described above, because design flows are intended to cover the maximum water usage that is reasonably expected.

2.2 Groundwater Mounding Analysis

The objective of groundwater mounding analysis is to estimate the height of a water table mound that is expected to form beneath the proposed new septic disposal area designed by LGH for the 19 new homes. Nobis conducted separate mound height analyses for each of the three proposed septic disposal areas, but because Areas 2 and 3 are adjacent, it is more appropriate to treat these as a single, combined rectangular septic disposal area. Nobis did so by simulating the two areas as a single leachfield with the same total area (5,566 square feet); the resulting rectangle is shown in purple on Figure 2 and has a length of 110 feet and a width of 50.6 feet.

The mound height was estimated using a web-based mounding program by Aqtesolve: http://www.aqtesolve.com/forum/rmound.asp. This program is intended for a rectangular loading area (leachfield) and is based on an equation developed by Hantush (1967). The program assumes that the aquifer is a porous medium, is infinite, is unconfined, is homogeneous and isotropic, and has a flat potentiometric surface. The program gives a single result for the maximum mound height but does not produce a contour map of the predicted mound. The resulting predicted mound height can be checked against depth to seasonal high water table to avoid potential breakout and to predict whether compliance with vertical separation requirements will be achieved. The program produces a series of approximations intended to converge to a predicted mound height. The program is recommended by the Massachusetts Department of Environmental Protection.

As with any groundwater model or simulation, the actual groundwater mound that will form over a leachfield may differ from the estimate provided by the program, depending on the degree to which the assumptions are violated and the accuracy of the input parameters to the program. Nobis points out that for the combined Area 2/3 simulation, the water table slopes down to the east-northeast (Figure 1), and the aquifer is not homogeneous in this area, because the saturated thickness is considerably different between Proposed Septic Area 3 (MW-2) and Proposed Septic Area 2 (MW-3). This represents an unavoidable departure from the Hantush (1967) model assumptions and lends a degree of uncertainty to the result. Saturated thickness values are presented with the slug test results for February 13, 2015 by NGI (NGI Report, Appendix B).

Nobis conducted two sets of mound height calculations: the first set assumed the LGH design flows of 1,980 gpd for each Proposed Septic Disposal Area; the second set assumed the Town's recommended design flows of 2,970 gpd for each disposal area. Under each scenario, Nobis performed calculations for each of the three Proposed Septic Disposal Areas and for Areas 2 and 3 combined. For each of these discharge rate scenarios and Disposal Areas, Nobis calculated mound heights for durations of 30 days (the duration used by NGI), 90 days, and 180 days (the latter two durations are recommended by Mass DEP). For each calculation, Nobis used a saturated thickness that is the arithmetic average of values provided by NGI for the two monitoring wells at each location on February 13, 2015 (NGI Report, Appendix B). For the combined Areas 2/3, Nobis averaged the saturated thicknesses reported for MW-3-15, MW-3A-15, MW-2-15, and MW-2A-15. Finally, Nobis calculated the mound heights using two different hydraulic conductivities – the Site wide geometric mean of 9 ft/d and the Area-specific conductivity obtained by averaging (arithmetically) the slug test results for wells at the specific areas. The mound calculation inputs used by Nobis are summarized below and presented in Table 1.

Inputs:

The following input parameters were used in the calculation:

- Length of loading area: measured by Nobis for each specific Proposed Septic Disposal Area shown in Plate 5 of 11 in LGH's Residential Site Plan Set dated March 27, 2015 (BREM_197 03.31.2015) or as simulated for combined Areas 2 and 3
- Width of loading area: measured by Nobis for each specific Proposed Septic Disposal Area
- Loading volume: 1,980 or 2,970 gallons per day for each Area
- Loading rate: calculated by converting Loading Volume in gallons per day to cubic feet per day and dividing by the area of the leachfield for each Area
- Saturated thickness: averaged for each Area from slug test results for that specific Area (see above)

- Hydraulic conductivity: averaged for each Area from slug test results for that specific Area (see above)
- Specific yield: 0.195 (average of values for silt and for fine sand (Fetter, 1988, page 74)
- Duration: 30 days (used by NGI); 90 days (low end of Mass DEP recommended range; value requested by Town); and 180 days (high end of range recommended by Mass DEP)

Results:

Monitoring simulation results for different durations are shown in Tables 2 and 3 for the 110 gpd per bedroom (LGH) and 165 gpd per bedroom (Town) inputs, respectively. Depending on the assumptions used, for Area 1, the estimated mound height (above starting water level) under the LGH discharge rate varies from 0.58 feet to 1.43 feet. For the Town discharge rate, the estimated mound height varies from 0.86 feet to 2.08 feet.

For Area 2, the estimated mound height with LGH discharge rates varies from 1.85 to 2.82 feet. For the Town discharge rate, the estimated mound height varies from 2.6 to 3.88 feet.

For Area 3, the estimated mound height with LGH discharge rates varies from 1.24 to 1.85 feet. For the Town discharge rate, the estimated mound height varies from 1.81 to 2.66 feet.

For Areas 2 and 3 combined, the estimated mound height with LGH discharge rates varies from 1.18 to 1.93 feet. For the Town discharge rate, the estimated mound height varies from 1.74 to 2.72 feet. All of the results, for each duration and each value of hydraulic conductivity are shown on Table 2 (LGH discharge rate) and Table 3 (Town discharge rate).

In order to convert mound height to the predicted depth to the top of the groundwater mound (for Title 5 compliance), it is standard practice to use estimated seasonal high water table (ESHWT) as a starting point for a "worst case" assessment. However, no indication of the ESHWT was observed in any of the six monitoring wells drilled at the three Proposed Septic Disposal Areas (NGI Report, p. 2). In this case, NGI calculated ESHWT from test pit observations (NGI Report, p. 2-3); however, such an estimate can be unreliable because the ESHWT was extrapolated over significant distances. This resulted in ESHWT levels that were lower than actual measured water levels in some of the monitoring wells (NGI Report, Table 1), a contraction in terms. Therefore, Nobis used the water levels that were directly measured on January 23, 2015 (NGI Report, Table 1) as the starting water levels to which mound heights were added.

The following are the results for depth to groundwater after estimating groundwater mounding potential and adding it to the water levels on January 23, 2015:

- Potential depth to the top of the groundwater mound located in Area 1 ranges from 4.55 to 6.05 feet below ground surface.
- Potential depth to the top of the groundwater mound located in Area 2 ranges from 5.02 to 7.05 feet below ground surface.
- Potential depth to the top of the groundwater mound located in Area 3 ranges from 2.42 to 3.84 feet below ground surface.

- Potential depth to the top of the groundwater mound located in Areas 2&3, combined ranges from 4.25 to 5.81 feet below ground surface.

In order to convert the predicted depths to the top of the mound to relative mound elevations, Nobis added the predicted mound height to the average January 23, 2015 water elevations for the monitoring wells located at the particular proposed Area (Area 1 or Areas 2 and 3, combined). For illustrative purposes on Figure 2, Nobis selected the estimates based on LGH loading rate, Area-specific hydraulic conductivity, and 90 days duration. For Area 1, a mound height of 0.70 feet was added to the groundwater elevation of 103.01 feet to obtain 103.71 feet. For Areas 2 and 3 combined, the predicted mound height of 1.53 feet was added to the average groundwater elevation for the four MW-2 and MW-3 monitoring wells to obtain 113.21 feet, as shown on Figure 2. Clearly, because of the sloping water table in this vicinity, the post-mounding elevation may actually be higher than 113.21 feet in the western portion of the combined 2/3 area.

At the Site scale and with a groundwater contour interval of 5 feet, the predicted mound height of 0.70 feet is not enough to alter the contouring around Proposed Septic Disposal Area 1. However, the predicted mound increases the groundwater gradient and therefore the groundwater flow velocity. Also, the groundwater mound may encourage radial flow away from Proposed Septic Area 1, at least in the immediate vicinity.

At Proposed Septic Disposal Areas 2 and 3 the predicted mound height of 1.53 feet may cause a noticeable change, at map scale, in groundwater contouring in the vicinity, as shown by red contour lines in Figure 2. These lines are dashed, because they are conceptual and not supported by direct groundwater modeling that extends outside of the Septic Disposal Areas. As with Proposed Septic Disposal Area 1, increased groundwater gradients and groundwater flow velocities can be expected to result from the mound over Areas 2/3. Also, localized flow to the southwest appears likely, in addition to flow to the east-northeast and northeast. Additional monitoring wells would be needed in these areas to verify (or refute) these potential groundwater flow directions.

2.3 Nitrate Loading and Mass Balance Calculations

Nobis conducted a site-specific mass balance analysis for nitrate, based on Mass DEP's "Guidelines for Title 5 Aggregation of Flows and Nitrogen Loading, 310 CMR 15.216", revised 2/11/15 (Guidance). This Guidance follows Title 5, which "imposes a nitrogen loading limitation of 440 gallons per day per acre design flow for systems serving new construction … where the use of both on-site systems and on-site drinking water supply wells are proposed." According to the Guidance, if the mass balance analysis indicates that a nitrate concentration of 10 milligrams per liter (mg/L) or lower cannot be achieved at the downgradient property line or at a sensitive receptor, steps must be taken to reduce the volume or nitrogen concentration of the discharge or to obtain more land for nitrate dilution (Guidance, p. 11). Nobis notes that the mass balance approach is a regulatory yardstick that estimates nitrate concentrations for a site or portion of a site due to dilution and is not a not method for modeling the fate and transport of nitrate contamination in groundwater.

Calculations performed by Nobis for the LGH Site used information provided by LGH, followed the Guidance, and are based on the following assumptions:

- The design flow is 110 gpd per bedroom, as in LGH's design (BREM_145 01.05.2015). This results in a discharge of 1,980 gpd for each Proposed Septic Disposal Area.
- Nobis also performed mass balance calculations assuming the Town's recommended design flow of 165 gpd per bedroom, or 2,970 gpd for each Proposed Septic Disposal Area.
- Discharge to the existing septic system for the existing house will continue at a rate of 165 gpd (55 gpd per person X 3 people, for LGH's estimate) or 660 gpd (165 gpd per bedroom X 4 bedrooms, for Town's estimate)
- Wastewater discharged to the three Proposed Septic Disposal Areas has a nitrate concentration of 19 mg/L.
- Wastewater discharged to the existing septic system will have a nitrate concentration of 35 mg/L.
- The total nitrate load due to fertilizer will be about 34 kg per year (value obtained from NGI Report, Figure 2).
- 20% of precipitation that falls on the Site in an average year will recharge groundwater (value obtained from NGI Report, Figure 2).
- The effect of impervious surfaces is discounted, because the stormwater systems are designed to discharge stormwater back to the ground, on Site.
- Nitrates that are discharged from the Proposed Septic Disposal Areas, existing septic system, and fertilizer application fully mix with overburden groundwater beneath the Site.
- Interaction with bedrock groundwater is not included in the analysis.
- According to the Guidance, the resulting nitrate concentration is obtained by dividing the sum of the nitrate loads due to wastewater and fertilizer by the sum of the volume of wastewater discharge and recharge from precipitation.

Nobis estimated the nitrate concentration under four scenarios:

1. Scenario 1 – LGH design discharge rate described above for entire 9.84 acre Site;
2. Scenario 2 – Town design discharge rate described above for entire 9.84 acre Site;
3. Scenario 3 – LGH design discharge rate for Site west of the brook only (groundwater recharge from east of the brook will likely flow to the brook and not be available to dilute nitrate discharged to the Site west of the brook); and
4. Scenario 4 – Town design discharge rate for Site west of the brook only (same reason as for Scenario 3).

Although Scenarios 3 and 4 may be more realistic than Scenarios 1 and 2, Nobis notes that groundwater that is recharged by precipitation that falls to the northwest of the Site may recharge the site in the subsurface and provide additional dilution. Nobis also considered delineating "Areas of Impact" (AOI) for each of the Proposed Septic Disposal Areas and conducting mass balance analyses for these AOIs. The results would show nitrate concentrations even higher than those for Scenarios 1 – 4 (see below), so these AOI analyses were not necessary. The reason the nitrate concentrations would be higher if a smaller area is considered is that a smaller amount of recharge is available to dilute the nitrates in the groundwater.

Results of the mass balance nitrate analyses are summarized on Table 4, and more detailed information on the analyses is presented in Attachment 4. Scenario 1 provides the most favorable resulting nitrate concentration, 11.9 mg/L. The other scenarios indicate that nitrate concentrations of 13.6 to 15.5 mg/L can be expected under the assumptions described.

2.4 Nitrate Dispersion Analyses

Nobis performed nitrate dispersion analyses using an analytical method by Domenico (1987) in order to predict nitrate concentrations in overburden groundwater from Proposed Septic Disposal Area 1 and Proposed Septic Disposal Areas 2 and 3, combined. The concentrations are calculated along flow lines that lead from the Septic Areas to down-gradient property lines and sensitive receptors. (Analyses of flow lines that might emanate from the existing house's septic system were not performed.) The calculations were performed for three property line locations, two brook locations, three proposed well locations, and four existing neighbors' well locations (Figure 4). Each dispersion calculation starts from a point source, assuming a constant nitrate concentration of 19 mg/L. The calculation then simulates the diffusion of nitrate away from the flow line in the horizontal and vertical dimensions, in addition to dispersion and advection along the flow line. Because a point source and not a leachfield with given dimensions and volumetric inputs is modeled by this method, the approach and results are not strictly comparable to those obtained by the mass balance approach. The results of the different dispersion analyses presented below are best used as relative estimates of nitrate concentrations at the selected locations.

Nobis' analytical modeling was based on dispersion (diffusion and advection only) and was conservative (i.e. assumed no breakdown or attenuation of nitrates by other processes such as adsorption or biodegradation). As described in Attachment 5, Nobis accepted many of the inputs and assumptions used by NGI (NGI Report, Appendix D). However, Nobis and NGI used two different analytical model methods. (See NGI Report, p. 5 for a description of their method.) Nobis developed a spreadsheet to use the Domenico (1987) method. Nobis also used site specific hydraulic conductivity values for the calculations associated with Area 1 vs. the calculations associated with Area 2/3. Also, Nobis modified the Domenico (1987) method to use non-linear dispersivity coefficients (Attachment 5).

The results (Table 5) show predicted nitrate concentrations of 3.8 mg/L, 17.8 mg/L, and 8.4 mg/L at three property line locations (Flow Lines 1, 7, and 12). The results show predicted nitrate concentrations of 1.4 mg/L and 0.8 mg/L at two staff gauge locations on the brook (Flow Lines 4 and 10), but the calculation does not account for dilution by surface water flow during times of high runoff. The results show predicted nitrate concentrations of 1.7 to 4.7 mg/L **in the overburden** at three new proposed on-Site well locations, A11, A4, and A5. The results show predicted nitrate concentrations of 1.4 to 3.4 mg/L **in the overburden** at four existing neighbors' well locations, 90 Long Ridge Road, 55 Suffolk Lane Extension, 68 Garnet Rock Lane, and 200 Long Ridge Road. Because the existing wells and the proposed wells obtain (or will obtain) their water from fractures in bedrock, actual nitrate concentrations in these wells are not predictable with present information. Fractures that supply water to these wells likely obtain their recharge from overlying soils and overburden, with the water quality dependent on where the fracture reaches the top of bedrock. If the glacial till deposits at the site have a dense, low-permeability layer at the bottom this may provide hydraulic isolation of the bedrock from overlying overburden at the Site. On the other hand, if a fracture that supplies water to a well receives recharge from an area where the overburden groundwater has an elevated nitrate concentration, this may result in high nitrate levels in the well.

As with any groundwater model, the method and input assumptions necessarily involve simplifying assumptions that impact the accuracy of the results. One of the most critical of the inputs is the groundwater flow directions and gradients from predicted groundwater mounds that are expected over the Proposed Septic Disposal Areas. Because water level data are not available in some key locations, groundwater contours and flow directions are inferred based on topography (see Section 2.1.3). Additional monitoring wells would add confidence to this key aspect. For example, if additional monitoring wells east and south of Proposed Septic Disposal Areas 2 and 3 (Figure 4) show groundwater flow components to the southeast and south, proposed wells A3 and A8 and the existing well at 132 Long Ridge Road may be vulnerable and should be calculation points for future dispersion analyses. Similarly, if additional monitoring wells south or southwest of Proposed Septic Area 1 confirm that southward and southwestward components of groundwater flow occur in this area, additional dispersion analyses should be performed in this region, and elevated nitrate concentrations would likely be indicated along the property line south and southwest of Area 1.

Based on these considerations and the fact that key receptors include bedrock wells (see previous paragraph), the nitrate dispersion analysis results should call attention to key issues and locations for further study or monitoring and should not be taken as absolute predictors of nitrate concentrations at a given location. Further, the quantitative results shown by the dispersion analysis are not directly comparable to the mass balance results. Not only are both sets of results subject to their inherent assumptions and simplifications, but also the two methods model different mechanisms by which nitrate concentrations enter the groundwater and different mechanisms by which the nitrate concentrations may be reduced with time or location, as described above. Therefore, the results for the nitrate dispersion estimates are relative and should be used to identify locations of particular concern.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Conclusions

Lifetime Green Homes (LGH) has proposed a 40B development consisting of 19 new homes, to be served by 11 new bedrock wells and a shared septic system with three Septic Disposal Areas, on a 9.84 acre lot at 100 Long Ridge Road in Carlisle, Massachusetts (Site). Concerns have been raised regarding the potential impacts of proposed on-Site wastewater disposal systems on proposed on-Site and existing off-Site drinking water wells, and potential interference (yield and water level) effects between the new wells and the existing nearby wells. Also, potential interference effects between the proposed new wells are a concern.

A hydrogeologic conceptual site model has been developed to provide a framework for addressing these concerns; the model should be updated as more information is gained. According to the model, the Site is underlain by sandy glacial till. These overburden deposits are underlain by fractured and variably foliated metamorphic bedrock. Predominant bedrock fracture orientations are northeast and northwest, with steep dips. Because there is essentially no porosity or permeability in the rock matrix, these fractures represent the primary avenues for flow of groundwater within the bedrock. In the overburden, groundwater occurs in the pore spaces within the unconsolidated deposits and flows from areas of higher head to areas of lower head. The degree of hydraulic connection between overburden groundwater and bedrock groundwater at the Site has not been characterized. If dense, low permeability glacial till deposits are present on top of the bedrock in some locations, these deposits may inhibit flow between the overburden and the bedrock. Proposed wastewater discharge will be to the overburden; all new and proposed wells obtain their water from the bedrock.

Overburden thickness varies from zero to 24 feet at the Site, and the unconfined groundwater potentiometric surface (water table) ranges in depth from zero to 11 or more feet (expected to vary with seasons and weather). Based on nearby well records, overburden has a similar thickness range (zero to 25 feet) in the neighborhood. Based on reported well yields (0.75 to 100 gallons per minute), the fractured bedrock appears to have the capacity to transport groundwater in most locations in and near the Site.

Water level measurements taken by NGI in overburden monitoring wells and piezometers allow contouring the groundwater potentiometric surface (water table), although the spacing of the data allows varying interpretations. Nobis has assumed that the potentiometric surface follows topographic slope in areas between and beyond direct water level measurements. Uncertainty could be reduced by installing additional monitoring wells or piezometers and conducting a new round of water level measurements. In the vicinity of Proposed Septic Disposal Area 1, the predicted groundwater flow direction is eastward, with components of flow to the south and southwest also possible, based on topography. In the vicinity of Proposed Septic Disposal Areas 2 and 3, the predicted groundwater flow direction is east-northeastward and northeastward, with components of flow to the east and south also possible, based on topography.

Discharge of wastewater to the Proposed Septic Disposal Areas will create groundwater mounds whose predicted height depends on model assumptions and inputs. After 90 days of discharge and using site-specific hydraulic conductivity, LGH-designed discharge of 1,980 gpd for each Area, and other input parameters, maximum mound heights of 0.70 feet and 1.53 feet above the existing water table are predicted for Area 1 and Areas 2 and 3 combined, respectively. When Proposed Septic Area 3 is modeled separately, the predicted groundwater mound is less than 4 feet below the present ground surface. These groundwater mounds will increase groundwater gradients and flow velocities and may create radial flow in the immediate vicinities of the Proposed Septic Disposal Areas, but do not significantly alter the groundwater flow directions at the Site scale.

Nitrate mass balance analyses and nitrate dispersion analyses along selected flow lines to property boundaries and sensitive receptors predict overall average nitrate concentrations and location-specific concentrations. As with any modelling, the accuracy of these predicted concentrations depends on the simplifying assumptions inherent to the model and on the accuracy of the input values. With these cautions in mind, the mass balance results indicate that for either LGH's or the Town's proposed wastewater design flows, there may not be sufficient dilution to reduce nitrate concentrations below 10 milligrams per liter on the average. Dispersivity analyses also predict that at the site boundary to the northeast of Proposed Septic Disposal Areas 2 and 3, nitrate concentrations will be greater than 10 milligrams per liter. Nitrate concentrations above background levels but below 10 milligrams per liter are predicted at proposed and existing well locations. These predictions apply to nitrate concentrations in overburden groundwater, not in bedrock groundwater, nor are they directly comparable to the concentrations predicted by the mass balance approach. Present information is not sufficient to predict nitrate concentrations in any well. However, proposed on-Site wells A11, A4, A5, and possibly others appear vulnerable to nitrate levels significantly above background values or current levels.

3.2 Recommendations for Additional Study

The work documented in this report and presented in the NGI Report provides valuable information but does not fully resolve the key questions posed in Section 1.0. Some aspects of the project objectives can be met with additional studies recommended below, but other questions cannot be answered until some or all of the proposed new bedrock drinking water wells are drilled and tested. Nobis recommends the following additional studies:

1. ***Characterize the hydraulic connection between overburden and groundwater –*** If groundwater beneath the Site flows freely between overburden and bedrock, several potential well locations are threatened with possibly elevated nitrate levels. However, if dense, low permeability glacial till mantles the top of the bedrock, properly constructed bedrock wells might be relatively isolated from this threat. Nobis proposes two investigations, one or both of which will help characterize the hydraulic connection (or lack thereof) between overburden and bedrock, but Nobis also points out that this characterization will not completely eliminate risk.
    a) ***Install borings to confirmed bedrock –*** Using a larger drilling rig or different drilling method, drill past “refusal” to determine whether “refusal” was caused by dense glacial till, boulder, or bedrock. The borings could also be used to construct additional monitoring wells, suggested in item 2.
    b) ***Install pressure transducers in existing monitoring wells –*** Install pressure transducers in one member of each of the well pairs, MW-1, MW-2, and MW-3, and collect water level measurements at a pre-set interval, say once a minute for one week. When the existing bedrock well at the Site (or possibly a neighboring well) cycles on or off, the transducers in the monitoring wells should show a water level response if there is a hydraulic connection.

2. ***Install additional shallow monitoring wells for more complete mapping of overburden groundwater contours and flow directions –*** Accurate groundwater flow directions are crucial for several analyses. In order to map the overburden groundwater potentiometric surface near the Proposed Septic Disposal Areas using observed water levels and not just topography, Nobis recommends single new monitoring wells in the following locations:
    a) ***Southeast of Proposed Septic Area 2,*** *but closer than MW-4-15*
    b) ***South or southwest of Proposed Septic Area 3,*** *near the property line*
    c) ***Northeast of Proposed Septic Area 2 or 3,*** *preferably across the property line if neighbor will allow*
    d) ***East of Proposed Septic Area 1,*** *between proposed well sites A10 and A11*
    e) ***Southwest of Proposed Septic Area 1,*** *just off the edge of the road*

    Following installation of new wells, a new synoptic round of groundwater levels from the new and existing monitoring wells and the two staff gauges (and PZ-1 if accessible) should be taken and a new groundwater contour map should be constructed.

3. ***Depending on the results of the revised groundwater mapping, conduct additional nitrate dispersivity analyses –*** If item 2 is conducted, the new groundwater contour map will likely suggest additional sensitive receptors and down-gradient property line locations for dispersivity analyses.

4.0 REFERENCES

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We look forward to continuing to work with you and the Town on this project. Thank you for the opportunity to be of service. If you require additional information, please do not hesitate to contact us at (603) 224–4182.

Very truly yours,

Nobis Engineering, Inc.

James H. Vernon, Ph.D., P.G.
Senior Hydrogeologist

Tables:

Figures:

Attachments:

TABLES

Table 1
Input Parameters for Mounding Analysis
100 Long Ridge Road, Carlisle, MA

Parameter	Values Used	Notes
Length (E-W) of Recharge Area (ft)	51.73	from NGI report, 3/25/15; same dimensions for all 3 areas; not used by Nobis
	55.95	Area 1, measured from Sheet 5 of 11 from 3/27/15 LGH Plan Set
	50.17	Area 2, measured from Sheet 5 of 11 from 3/27/15 LGH Plan Set
	59.64	Area 3, measured from Sheet 5 of 11 from 3/27/15 LGH Plan Set
	110	Areas 2&3 combined, with same total area
Width (N-S) of Recharge Area (ft)	51.73	from NGI report, 3/25/15; same dimensions for all 3 areas; not used by Nobis
	52.15	Area 1, measured from Sheet 5 of 11 from 3/27/15 LGH Plan Set
	53.15	Area 2, measured from Sheet 5 of 11 from 3/27/15 LGH Plan Set
	47.76	Area 3, measured from Sheet 5 of 11 from 3/27/15 LGH Plan Set
	50.6	Areas 2&3 combined, with same total area
Recharge Volume (gpd)	1980	LGH Estimate, per each recharge area
	2970	Town Estimate, per each recharge area
Recharge Rate (ft/day)	0.0906	Recharge volume divided by leachfield Area 1 (LGH Est.)
	0.1359	Recharge volume divided by leachfield Area 1 (Town Est.)
	0.0991	Recharge volume divided by leachfield Area 2 (LGH Est.)
	0.1487	Recharge volume divided by leachfield Area 2 (Town Est.)
	0.0928	Recharge volume divided by leachfield Area 3 (LGH Est.)
	0.1392	Recharge volume divided by leachfield Area 3 (Town Est.)
	0.0475	Recharge volume divided by leachfield Area 2&3 (LGH Est.)
	0.0712	Recharge volume divided by leachfield Area 2&3 (Town Est.)
Saturated Thickness Area #1(ft)	12.49	appears to use Est. Seasonal High Water Table; in NGI text, p.4; not used by Nobis
	9.65	average of MW1 and 1A from slug tests by NGI
Saturated Thickness Area #2 and 3(ft)	7.13	NGI text, p.4; not used by Nobis
	5.07	average of MW2, 2A, 3, 3A from slug tests by NGI
Hydraulic Conductvity (ft/day)	2.8x10e-2 to 14.2	sieve analysis range; not used because of high percentages of fines and gravel in soil samples
	9	geometric mean (range from 2.08-23.75 ft/day) of slug test results
	19.1	Area #1 (average of MW1 and 1A slug tests)
	9.98	Area #3 (average of MW2 and 2A slug tests)
	10.61	Area #2 (average of MW3 and 3A slug tests)
	10.3	Area #2 and 3 (average of MW2, 2A, 3, 3A slug tests)
Specific Yield	0.07	Based on literature value for sandy clay (Fetter, 1988); used by NGI
	0.195	Based on literature value for fine sand/silt (Fetter 1988); used by Nobis
Duration (days)	30	used by NGI
	90	low end of DEP range for septic systems >2000 gpd; requested by Town
	180	high end of DEP range for septic systems >2000 gpd

Table 2
Mounding Analysis Results
100 Long Ridge Road, Carlisle, MA
LGH Loading Rates: 1980 gpd for each Proposed Septic Disposal Area

Trial	Average Depth to Water for Area, measured by NGI 1/23/15	Estimated Groundwater Mounding Potential(ft) 30 days	Estimated Groundwater Mounding Potential(ft) 90 days	Estimated Groundwater Mounding Potential(ft) 180 days	Depth to Top of Mound (ft bgs) 30 days	Depth to Top of Mound (ft bgs) 90 days	Depth to Top of Mound (ft bgs) 180 days	Notes
1	6.63	1.04	1.28	1.43	5.59	5.35	5.20	Area 1; starting saturated thickness 9.65; k=9
2	6.63	0.58	0.70	0.77	6.05	5.93	5.86	Area 1; starting saturated thickness 9.65; k=19.1
3	8.9	2.05	2.54	2.82	6.85	6.36	6.08	Area 2; starting saturated thickness 3.23; k=9
4	8.9	1.85	2.28	2.53	7.05	6.62	6.37	Area 2; starting saturated thickness 3.23; k=10.61
5	5.08	1.34	1.66	1.85	3.74	3.42	3.23	Area 3; starting saturated thickness 6.66; k=9
6	5.08	1.24	1.54	1.71	3.84	3.54	3.37	Area 3; starting saturated thickness 6.66; k=9.98
7	6.99	1.29	1.68	1.93	5.70	5.31	5.06	Areas 2&3 combined; starting saturated thickness 5.07; k=9
8	6.99	1.18	1.53	1.75	5.81	5.46	5.24	Areas 2&3 combined; starting saturated thickness 5.07; k=10.3

Notes:

1. All trials used an on-line version of Aqtesolve's mound calculator for a rectangular area, based on Hantush (1967).
2. All trials assumed 1980 gpd discharged to each septic disposal area for 30 days.
3. Saturated thickness for Area 1 is average for MW1 & MW1A on 2/13/15 (date of slug test).
4. Saturated thickness for Area 2 is average for MW3 & MW3A on 2/13/15 (date of slug test).
5. Saturated thickness for Area 3 is average for MW2 & MW2A on 2/13/15 (date of slug test).
6. Hydraulic conductivity for Trials 1,3,5,&7 is the geometric mean hydraulic conductivity from NGI 3/25/15 report.
7. Hydraulic conductivity for Trial 2 is the average of slug tests for MW1 & MW1A by NGI on 2/13/15.
8. Hydraulic conductivity for Trial 4 is the average of slug tests for MW3 & MW3A by NGI on 2/13/15.
9. Hydraulic conductivity for Trial 6 is the average of slug tests for MW2 & MW2A by NGI on 2/13/15.
10. Hydraulic conductivity for Trial 8 is the average of slug tests for MW3, MW3A, MW2, & MW2A by NGI on 2/13/15.
11. For each trial, Depth to Top of Mound is calculated by subtracting the calculated maximum mound height from the average depth to water (in MWs for that area) measured by NGI on 1/23/15.
12. All trials used specific yield of 0.195, published value for fine sand/silt (Fetter, 1988).
13. Highlighted cells used for groundwater contouring in Figure 2.

Table 3
Mounding Analysis Results
100 Long Ridge Road, Carlisle, MA
Town Loading Rates: 2970 gpd for each Proposed Septic Disposal Area

Trial	Average Depth to Water for Area, measured by NGI 1/23/15	Estimated Groundwater Mounding Potential(ft) 30 days	Estimated Groundwater Mounding Potential(ft) 90 days	Estimated Groundwater Mounding Potential(ft) 180 days	Depth to Top of Mound (ft bgs) 30 days	Depth to Top of Mound (ft bgs) 90 days	Depth to Top of Mound (ft bgs) 180 days	Notes
1	6.63	1.53	1.87	2.08	5.10	4.76	4.55	Area 1; starting saturated thickness 9.65; k=9
2	6.63	0.86	1.03	1.14	5.77	5.60	5.49	Area 1; starting saturated thickness 9.65; k=19.1
3	8.9	2.88	3.51	3.88	6.02	5.39	5.02	Area 2; starting saturated thickness 3.23; k=9
4	8.9	2.6	3.17	3.49	6.30	5.73	5.41	Area 2; starting saturated thickness 3.23; k=10.61
5	5.08	1.95	2.39	2.66	3.13	2.69	2.42	Area 3; starting saturated thickness 6.66; k=9
6	5.08	1.81	2.22	2.47	3.27	2.86	2.61	Area 3; starting saturated thickness 6.66; k=9.98
7	6.99	1.87	2.41	2.74	5.12	4.58	4.25	Areas 2&3 combined; starting saturated thickness 5.07; k=9
8	6.99	1.72	2.2	2.50	5.27	4.79	4.49	Areas 2&3 combined; starting saturated thickness 5.07; k=10.3

Notes:
1. All trials used an on-line version of Aqtesolve's mound calculator for a rectangular area, based on Hantush (1967).
2. All trials assumed 2970 gpd discharged to each septic disposal area for 30 days.
3. Saturated thickness for Area 1 is average for MW1 & MW1A on 2/13/15 (date of slug test).
4. Saturated thickness for Area 2 is average for MW3 & MW3A on 2/13/15 (date of slug test).
5. Saturated thickness for Area 3 is average for MW2 & MW2A on 2/13/15 (date of slug test).
6. Hydraulic conductivity for Trials 1,3,5,&7 is the geometric mean hydraulic conductivity from NGI 3/25/15 report.
7. Hydraulic conductivity for Trial 2 is the average of slug tests for MW1 & MW1A by NGI on 2/13/15.
8. Hydraulic conductivity for Trial 4 is the average of slug tests for MW3 & MW3A by NGI on 2/13/15.
9. Hydraulic conductivity for Trial 6 is the average of slug tests for MW2 & MW2A by NGI on 2/13/15.
10. Hydraulic conductivity for Trial 8 is the average of slug tests for MW3, MW3A, MW2, & MW2A by NGI on 2/13/15.
11. For each trial, Depth to Top of Mound is calculated by subtracting the calculated maximum mound height from the average depth to water (in MWs for that area) measured by NGI on 1/23/15.
12. All trials used specific yield of 0.195, published value for fine sand/silt (Fetter, 1988).

TABLE 4

MASS-BALANCE NITRATE LOADING ANALYSES - MULTIPLE SCENARIOS
100 Long Ridge Road
Carlisle, Massachusetts

<u>Scenario 1</u> **Variables - Applicant Assumptions**

Wastewater Yearly Volume Area 1 (L/yr)	=	W_{A1}	=	2735708.58
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	$W_{A1/2}$	=	5471417.16
Wastewater Yearly Volume Existing (L/yr)	=	W_E	=	227975.715
Entire Site Area (ft^2)		A_S	=	428630.4

	(W_{A1} x N_{A1}	+	$W_{A1/2}$ x $N_{A1/2}$	+	W_E x N_E)	+	N_F			=	Load	
Load	51978463.02		103956926		7979150		34019428			=	197933967.1	mg
Volume	(W_{A1}	+	$W_{A1/2}$	+	W_E)	+	(R_P x A_S	x	7.48 gal / ft^3 x 3.79 L / gal)	=	Volume	
	2735708.58		5471417.16		227975.7		291468.7	x	28.31	=	16687976.28	L

= **11.9 mg/L Scenario 1**

<u>Scenario 2</u> **Variables - Town Assumptions**

Wastewater Yearly Volume Area 1 (L/yr)	=	W_{A1}	=	4103562.87
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	$W_{A1/2}$	=	8207125.74
Wastewater Yearly Volume Existing (L/yr)	=	W_E	=	911902.86
Entire Site Area (ft^2)	=	A_S	=	428630.4

	(W_{A1} x N_{A1}	+	$W_{A1/2}$ x $N_{A1/2}$	+	W_E x N_E)	+	N_F			=	Load	
Load	77967694.53		155935389.1		31916600		34019428			=	299839111.7	mg
Volume	(W_{A1}	+	$W_{A1/2}$	+	W_E)	+	(R_P x A_S	x	7.48 gal / ft^3 x 3.79 L / gal)	=	Volume	
	4103562.87		8207125.74		911902.9		291468.7	x	28.31	=	21475466.29	L

= **14.0 mg/L Scenario 2**

<u>Scenario 3</u> **Variables - Applicant Assumptions**

Wastewater Yearly Volume Area 1 (L/yr)	=	W_{A1}	=	2735708.58
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	$W_{A1/2}$	=	5471417.16
Wastewater Yearly Volume Existing (L/yr)	=	W_E	=	227975.715
Site West of the Brook Only Area (ft^2)	=	A_W	=	317447

	(W_{A1} x N_{A1}	+	$W_{A1/2}$ x $N_{A1/2}$	+	W_E x N_E)	+	N_F			=	Load	
Load	51978463.02		103956926		7979150		34019428			=	197933967.1	mg
Volume	(W_{A1}	+	$W_{A1/2}$	+	W_E)	+	(R_P x A_W	x	7.48 gal / ft^3 x 3.79 L / gal)	=	Volume	
	2735708.58		5471417.16		227975.7		215864	x	28.31	=	14547244.58	L

= **13.6 mg/L Scenario 3**

<u>Scenario 4</u> **Variables - Town Assumptions**

Wastewater Yearly Volume Area 1 (L/yr)	=	W_{A1}	=	4103562.87
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	$W_{A1/2}$	=	8207125.74
Wastewater Yearly Volume Existing (L/yr)	=	W_E	=	911902.86
Site West of the Brook Only Area (ft^2)	=	A_W	=	317447

	(W_{A1} x N_{A1}	+	$W_{A1/2}$ x $N_{A1/2}$	+	W_E x N_E)	+	N_F			=	Load	
Load	77967694.53		155935389.1		31916600		34019428			=	299839111.7	mg
Volume	(W_{A1}	+	$W_{A1/2}$	+	W_E)	+	(R_P x A_W	x	7.48 gal / ft^3 x 3.79 L / gal)	=	Volume	
	4103562.87		8207125.74		911902.9		215864	x	28.31	=	19334734.6	L

= **15.5 mg/L Scenario 4**

Scenario Constants

Wastewater Nitrate Concentration Area 1 (mg/L)	=	N_{A1}	=	19
Wastewater Nitrate Concentration Areas 2/3 (mg/L)	=	$N_{A2/3}$	=	19
Wastewater Nitrate Concentration Existing (mg/L)	=	N_E	=	35
Nitrate Load from Fertilizer (mg/yr)	=	N_F	=	34019428
Recharge from Precipitation (ft/yr)	=	R_P	=	0.68

Table 5
Nitrate Dispersion Analysis
100 Long Ridge Road
Stratford, Connecticut

Line No.	Start	End	K (m/yr)	X (m)	Y (m)	Z (m)	ΔH	v_X (m/yr)	α_X (m)	α_Y (m)	α_Z (m)	C_0 (mg/L)	T (yr)	C (mg/L)	Target
1	Septic 1	Property line southeast of A11	22830	54.55	15.9	1.8	0.02	2646	3.1464	0.3146	0.315	19	30	3.783944	5
2	Septic 1	A11	22830	44.97	15.9	1.8	0.03	3209.5	2.7921	0.2792	0.279	19	30	4.7301565	5
3	Septic 1	90 Long Ridge Road well	22830	91.85	15.9	1.8	0.03	3689.2	4.2288	0.4229	0.423	19	30	1.9461301	5
4	Septic 1	SG-1	22830	118.1	15.9	1.8	0.04	4515.3	4.8196	0.482	0.482	19	30	1.3839848	5
5	Septic 1	Ringheiser #68 well	22830	61.67	17.1	1.8	0.01	1709.5	3.3846	0.3385	0.338	19	30	3.3949038	5
6	Septic 1	Hanauer #200 well	22830	80.87	17.1	1.8	0.02	1784.7	3.9471	0.3947	0.395	19	30	2.4228643	5
7	Septic 2/3	Property line to east	12330	7.675	33.5	1.8	0.14	8787.6	0.6181	0.0618	0.062	19	30	17.772342	5
8	Septic 2/3	Well A4	10363	70.58	15.4	1.8	0.09	4555.8	3.6584	0.3658	0.366	19	30	2.6907234	5
9	Septic 2/3	Well A5	10363	97.81	15.4	1.8	0.07	3829.3	4.3723	0.4372	0.437	19	30	1.7443379	5
10	Septic 2/3	SG-2	11315	175.9	15.4	1.8	0.04	2544.6	5.8478	0.5848	0.585	19	30	0.7755147	5
11	Septic 2/3	Higgins #55 well	12330	183.3	33.5	1.8	0.04	2660	5.962	0.5962	0.596	19	30	1.3674463	5
12	Septic 2/3	south property line	12330	24.88	15.4	1.8	0.04	2710.9	1.8565	0.1857	0.186	19	30	8.3841112	5

Aquifer Properties

n = 0.35

n_e = 0.2

ρ_d (g/cm^3)= 1.59

	K (ft/day)	K (m/yr)
MW-1	23.8	28441
MW-1A	14.4	17220
MW-2	12.1	14502
MW-2A	7.86	9406
MW-3	8.84	10585
MW-3A	12.4	14825
MW-4	2.09	2498
MW-5	6.06	7254

	source H (ft)	end H (ft)	length (ft)	ΔH
Line 1	103.71	100	160.1	0.02318
Line 2	103.71	100	132.0	0.02812
Line 3	103.71	95	269.5	0.03232
Line 4	103.71	90	346.6	0.03956
Line 5	103.71	101	181.0	0.01498
Line 6	103.71	100	237.3	0.01563
Line 7	113.21	110	22.5	0.14254
Line 8	113.21	95	207.1	0.08792
Line 9	113.21	92	287.0	0.0739
Line 10	113.21	90	516.0	0.04498
Line 11	113.21	90	537.9	0.04315
Line 12	113.21	110	73.0	0.04397

Notes:

Kd = solute-specific distribution coefficient, T = arrival time

C_0 = source concentration, C = predicted concentration

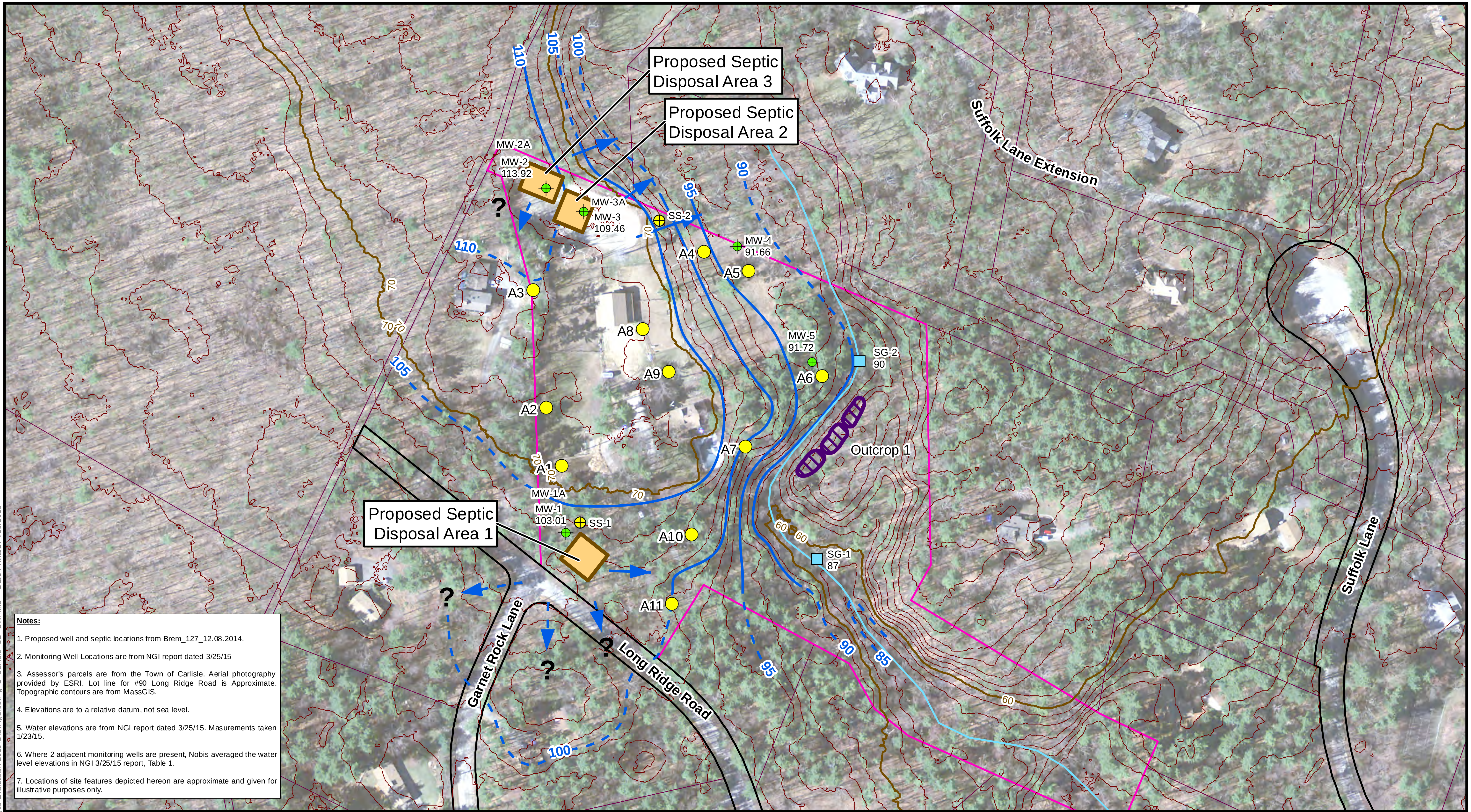
SWPC = CTDEEP RSR Surface Water Protection Criteria, June 27, 2013

V_C substituted for V_X in equation below to incorporate retardation

$V_C = V_X/(1+[(1-n)/n*K_d*\rho_d])$

$v_X = \Delta H * K/n_e$

FIGURES



Notes:

1. Proposed well and septic locations from Brem_127_12.08.2014.

2. Monitoring Well Locations are from NGI report dated 3/25/15

3. Assessor's parcels are from the Town of Carlisle. Aerial photography provided by ESRI. Lot line for #90 Long Ridge Road is Approximate. Topographic contours are from MassGIS.

4. Elevations are to a relative datum, not sea level.

5. Water elevations are from NGI report dated 3/25/15. Masurements taken 1/23/15.

6. Where 2 adjacent monitoring wells are present, Nobis averaged the water level elevations in NGI 3/25/15 report, Table 1.

7. Locations of site features depicted hereon are approximate and given for illustrative purposes only.

Legend

A1 Proposed Well with LGH # (Location Approximate)

Groundwater Flow Estimate (Dashed Where Inferred From Topography)

Monitoring Well with Water Level Elevation (To Relative Datum) on 1/23/15

Nobis Soil Sample 4/3/15

Staff Gauge Installed by NGI

Proposed Septic Disposal Area

LGH Project Site

10 Foot Topographic Contour

1 Foot Topographic Contour

5 Foot Overburden Groundwater Level Contour (Dashed Where Inferred; see text for explanation)

0 75 150 300
Feet
1 inch = 150 feet

N

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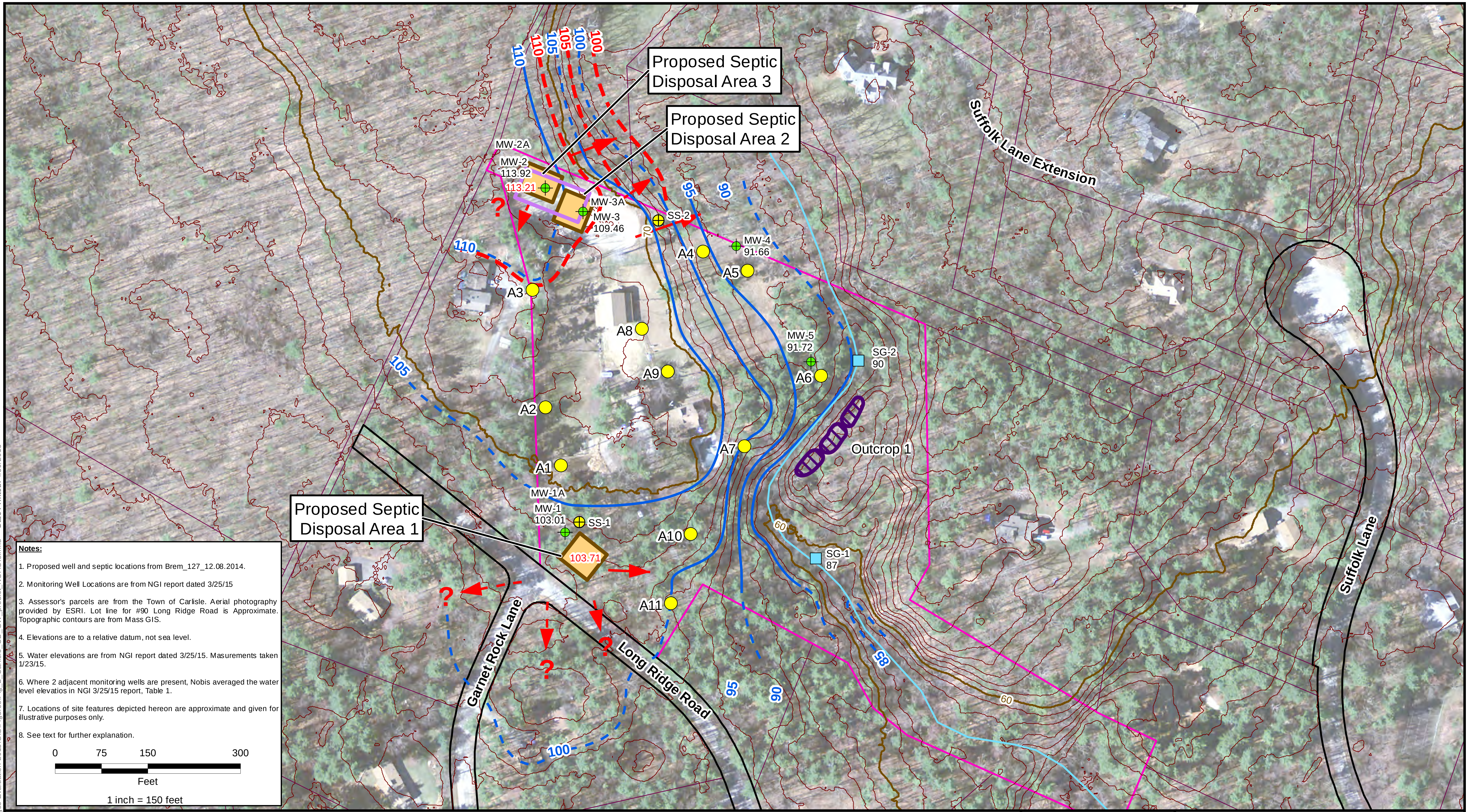
Client-Focused, Employee-Owned

FIGURE 1

OVERBURDEN GROUNDWATER POTENTIOMETRIC SURFACE MAP
100 LONG RIDGE ROAD
CARLISLE, MASSACHUSETTS

PREPARED BY: JH	CHECKED BY: JV
PROJECT NO. 89220.00	DATE: APRIL 2015

Path: J:\89220.00 - Carlisle Hydrogeo Evaluation 2015\GIS\Figures\Fig_1_Carlisle_OB_GW.mxd **Date Printed:** 4/29/2015



Notes:

1. Proposed well and septic locations from Brem_127_12.08.2014.
2. Monitoring Well Locations are from NGI report dated 3/25/15
3. Assessor's parcels are from the Town of Carlisle. Aerial photography provided by ESRI. Lot line for #90 Long Ridge Road is Approximate. Topographic contours are from Mass GIS.
4. Elevations are to a relative datum, not sea level.
5. Water elevations are from NGI report dated 3/25/15. Masurements taken 1/23/15.
6. Where 2 adjacent monitoring wells are present, Nobis averaged the water level elevatios in NGI 3/25/15 report, Table 1.
7. Locations of site features depicted hereon are approximate and given for illustrative purposes only.
8. See text for further explanation.

0 75 150 300
Feet
1 inch = 150 feet

Legend

- A1 — Proposed Well with LGH # (Location Approximate)
- Groundwater Flow Estimate (With Mounding) (Dashed Where Inferred From Topography)
- MW-2 113.92 — Monitoring Well with Water Level Elevation (To Relative Datum) on 1/23/15
- 113.21 — Predicted Mound Height
- SS-1 — Nobis Soil Sample 4/3/15
- SG-2 — Staff Gauge Installed by NGI
- Proposed Septic Disposal Area
- Composite Conceptual Septic Disposal Area
- LGH Project Site
- Bedrock Outcrops
- 5 Foot Overburden Groundwater Level Contour (Dashed Where Inferred; see text for explanation)
- Predicted Overburden Groundwater Level Contour with Mounding (see text for explanation)
- 10 Foot Topographic Contour
- 1 Foot Topographic Contour

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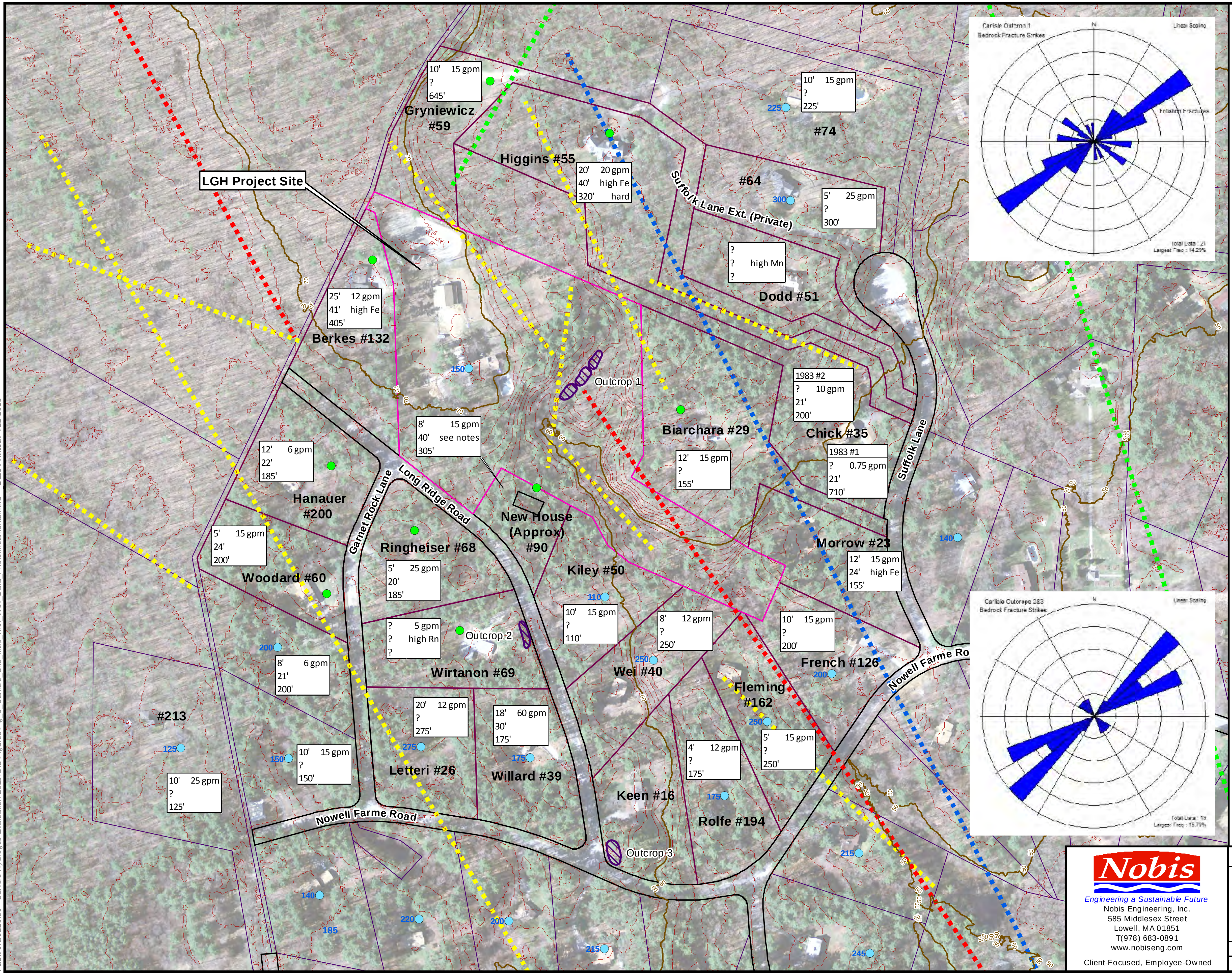
FIGURE 2

OVERBURDEN GROUNDWATER POTENTIOMETRIC SURFACE MAP WITH PREDICTED MOUNDING
100 LONG RIDGE ROAD
CARLISLE, MASSACHUSETTS

PREPARED BY: JH	CHECKED BY: JV
PROJECT NO. 89220.00	DATE: APRIL 2015

Path: J:\89220.00 - Carlisle Hydrogeo Evaluation 2015\GIS\Figures\Fig_2_Carlisle_OB_GW_predict_NOAOIs.mxd Date Printed: 4/30/2015



Notes:

1. Well at 90 Long Ridge Road overflowed following drilling (overflow stopped by raising casing); high Fe, Mn, Rn; flowed at 75 gpm; pumped 4 hours at 15 gpm.
2. If no well symbol is shown for a lot, location is not known.
3. Non DEP locations from LGH map updated 1/8/15 and from Nobis site visit 1/23/15.
4. Assessor's parcels are from the Town of Carlisle. Contours are from MassGIS. Aerial photography provided by ESRI. Lot line for #90 Long Ridge Road is approximate.
5. Abutters identified by Board of Assessors (10/2/14 letter).
6. Elevations based on a relative datum, not sea level.
7. Fracture stikes relative to true north (see text).
8. Locations of site features depicted hereon are approximate and given for illustrative purposes only.

Legend

- Existing Well Observed by Nobis, 1/23/15
- DEP Well Locations with Depth
- LGH Project Site
- Abutters with Owner Name and Address
- Other Properties
- Bedrock Outcrops
- 10 Foot Contour
- 1 Foot Contour

Photolineament Source

- NAPP 1995
- NHAP 1995 CIR
- NHAP 1985 B&W
- VESC 1981

Photolineament Strength

- Intermediate
- Subtle

Depth to Bedrock — 20' — 20 gpm — Estimated Yield
Depth of Casing — 40' — Other
Total Depth — 320'

N

0 125 250 500
Feet
1 inch = 250 feet

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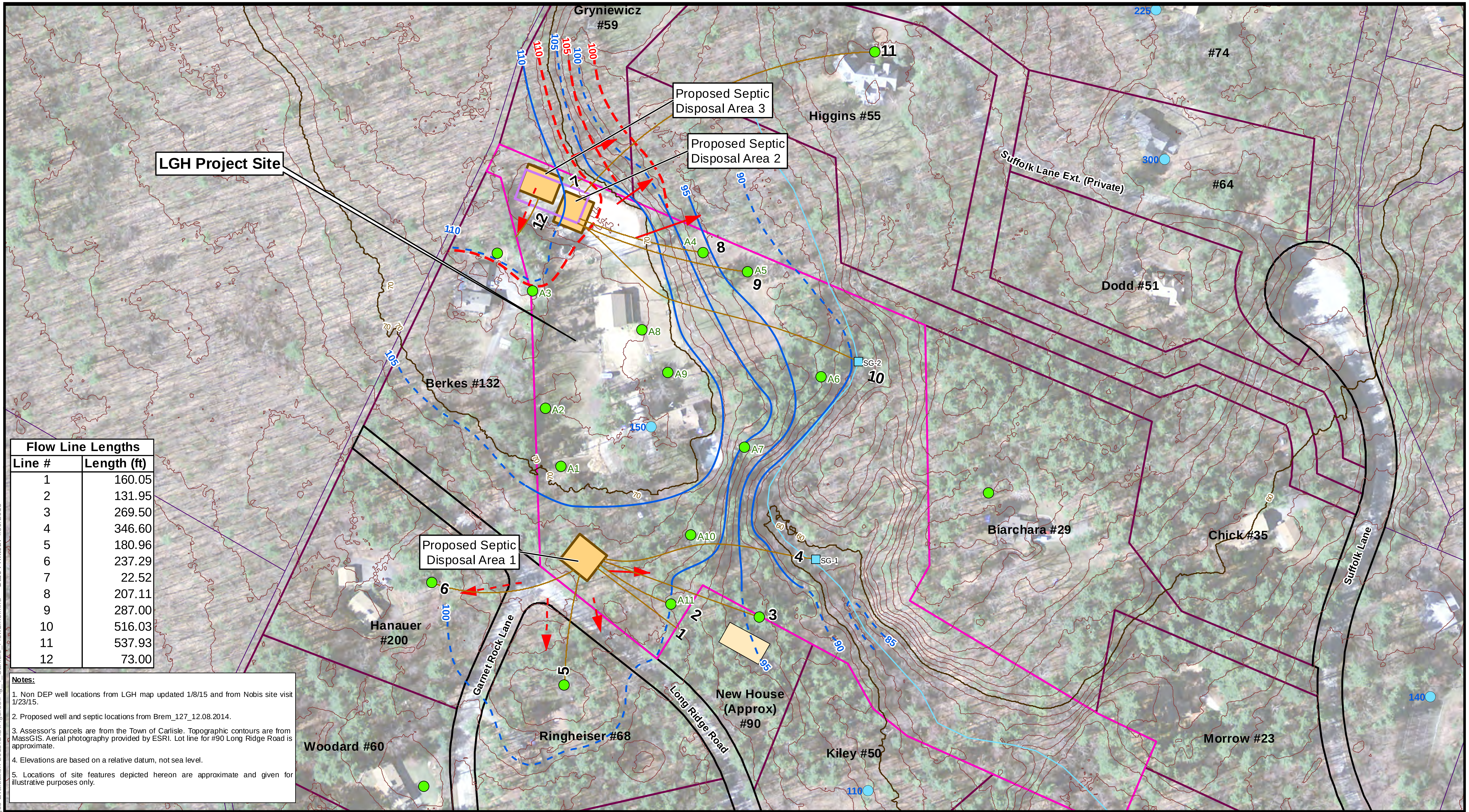
FIGURE 3

SITE MAP WITH EXISTING WELLS AND BEDROCK FRACTURE DATA
100 LONG RIDGE ROAD
CARLISLE, MASSACHUSETTS

PREPARED BY: NZ	CHECKED BY: JV
PROJECT NO. 89220.00	DATE: APRIL 2015

Path: J:\89220.00 - Carlisle Hydrogeo Evaluation 2015\GIS\Figures\Fig_3_Carlisle_Site_Map_with_Well_Data_Photolineaments.mxd Date Printed: 4/22/2015



Flow Line Lengths	
Line #	Length (ft)
1	160.05
2	131.95
3	269.50
4	346.60
5	180.96
6	237.29
7	22.52
8	207.11
9	287.00
10	516.03
11	537.93
12	73.00

Notes:

1. Non DEP well locations from LGH map updated 1/8/15 and from Nobis site visit 1/23/15.

2. Proposed well and septic locations from Brem_127_12.08.2014.

3. Assessor's parcels are from the Town of Carlisle. Topographic contours are from MassGIS. Aerial photography provided by ESRI. Lot line for #90 Long Ridge Road is approximate.

4. Elevations are based on a relative datum, not sea level.

5. Locations of site features depicted hereon are approximate and given for illustrative purposes only.

Legend

- A1 Non-DEP Well Locations
- 185 DEP Well Locations with Depth
- SG1 Staff Gauge Installed by NGI
- 1 Dispersivity Calculation Points (see text for details)
- Proposed Septic Disposal Area
- Composite Conceptual Septic Disposal Area
- LGH Project Site
- Abutters
- Other Properties
- 10 Foot Contour
- 1 Foot Contour
- Flow Line for Nitrate Dispersion Analysis
- Groundwater Contour (Dashed where inferred)
- Predicted Overburden Groundwater Level Contour with Mounding (see text for explanation)
- Groundwater Flow Estimate (With Mounding) (Dashed Where Inferred From Topography)

0 75 150
Feet
1 inch = 150 feet

N

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FIGURE 4

APPROXIMATE FLOW LINES AND SENSITIVE RECEPTORS FOR NITRATE DISPERSION ANALYSIS
100 LONG RIDGE ROAD
CARLISLE, MASSACHUSETTS

PREPARED BY: JH	CHECKED BY: JV
PROJECT NO. 89220.00	DATE: APRIL 2015

Path: J:\89220.00 - Carlisle Hydrogeo Evaluation 2015\GIS\Figures\Fig_4_Carlisle_FlowLine.mxd Date Printed: 4/30/2015

ATTACHMENTS

ATTACHMENT 1 – BEDROCK OUTCROP FRACTURE TECHNICAL MEMORANDUM

Bedrock Outcrop Fracture Measurements

On April 3, 2015, Nobis measured bedrock fractures at three outcrops in or near the study area (Figure 3):

- Outcrop 1 – located on the LGH property, 100 Long Ridge Road, east of the brook; Nobis thanks Jeffrey Brem for permission to access the property;
- Outcrop 2 – located on the west side of Long Ridge Road, across from 50 Long Ridge Road; and
- Outcrop 3 – located at the intersection (north side) of Long Ridge Road and Nowell Farme Road.

The purpose of the investigation was to characterize the bedrock and assess the orientation of bedrock fractures that appear capable of transporting groundwater flow. This information is intended to enhance the hydrogeologic conceptual site model for the LGH site and surrounding area and is important because all of the existing and proposed water wells in the area are (or will be) completed in bedrock and draw their water from bedrock fractures.

Nobis measured the orientation (strike and dip) of all observed fractures in these outcrops that appeared capable of conducting groundwater flow. Fractures that were measured in portions of the outcrop that appear to have slumped or not be "in place" were excluded. All measurements were taken with a geologist's field transit and compass and were made relative to magnetic north. The strike orientations were then converted to true north by subtracting 14.75 degrees, the current magnetic declination for Carlisle. (*Strike* is defined as the orientation of a horizontal line in a fracture plane; for a vertical or steep fracture, this is simply the trend of the fracture in map view. *Dip* is the angle or slope of the fracture, down from the horizontal; 80 degrees is steep, and 15 degrees is shallow.) For convenient data transfer to fracture plotting software, the orientation data were recorded according to the "right-hand rule". (According to the right hand rule, a fracture's orientation can be uniquely described by two numbers, the strike relative to 360 degrees, and the dip angle below horizontal. If one faces in the strike direction, one's right arm would point down the dip of the fracture. For example, for a fracture with a right-hand-rule orientation of "60, 35", the strike is 60 degrees east of north, and the dip is 35 degrees down to the southeast.)

A description of each outcrop follows:

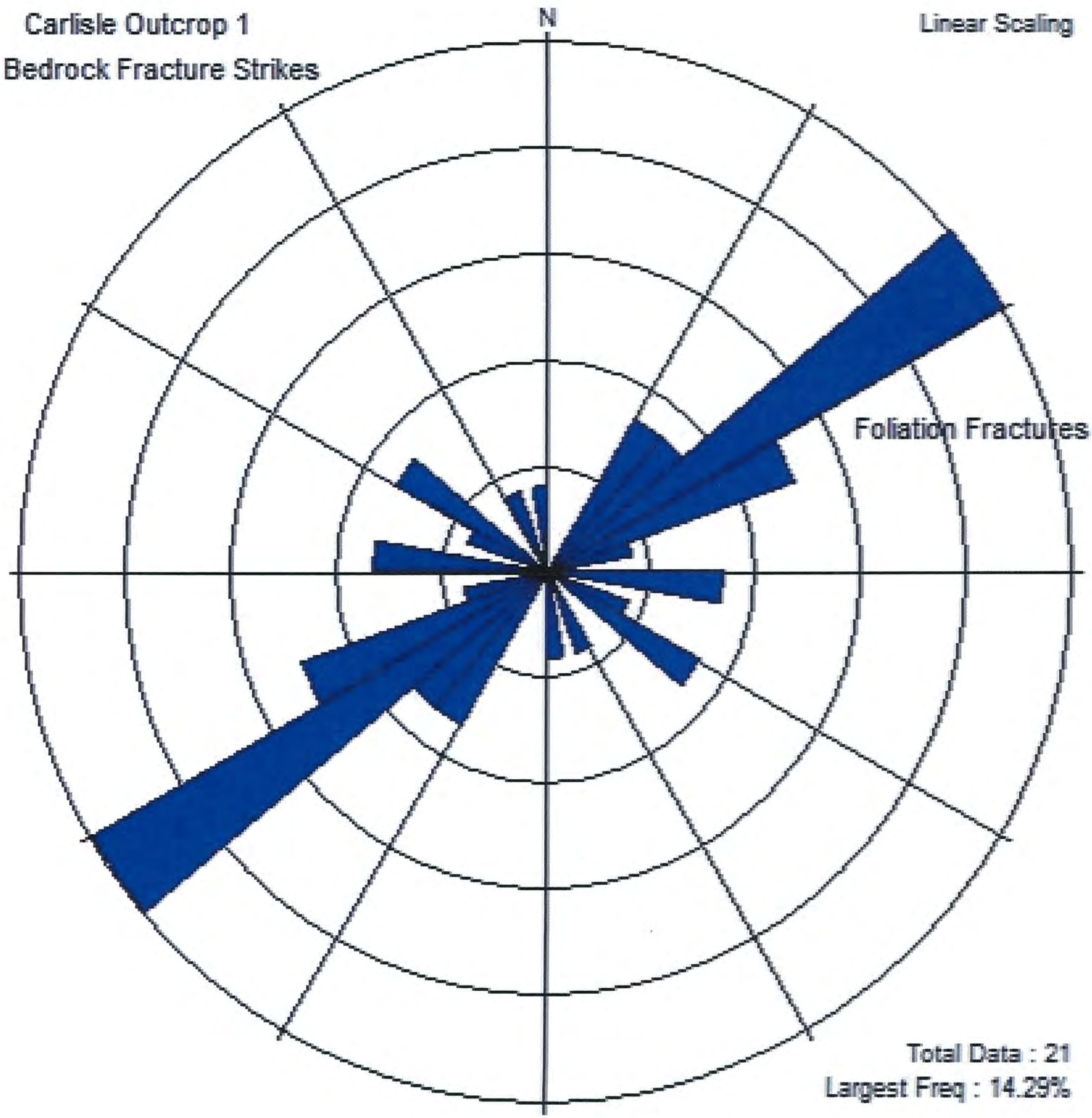
- Outcrop 1 – biotite schist with orange feldspar and hornblende; iron oxide (rusty) weathering is present. The outcrop is a ridge-type outcrop, extending northeastward along a northeast-trending ridge on the east side of a small brook (see Figure 3). The outcrop is defined by steep metamorphic foliations (schistosity) that trend northeastward and are sometimes coincident with open fractures. The outcrop is segmented into three different sections, aligned southwest to northeast.
- Outcrop 2 – more massive and blocky than outcrop 1. The outcrop is a road cut and was partially snow-covered. Metamorphic foliations are present but less prominent and generally are not fractured, except at the south end of the outcrop, which is more schistose and less blocky.
- Outcrop 3 – darker, more massive, and blockier than outcrops 1 and 2; amphibolitic (hornblende) and weakly foliated. The outcrop is a road cut. Foliation is more evident at the southeast end of the outcrop.

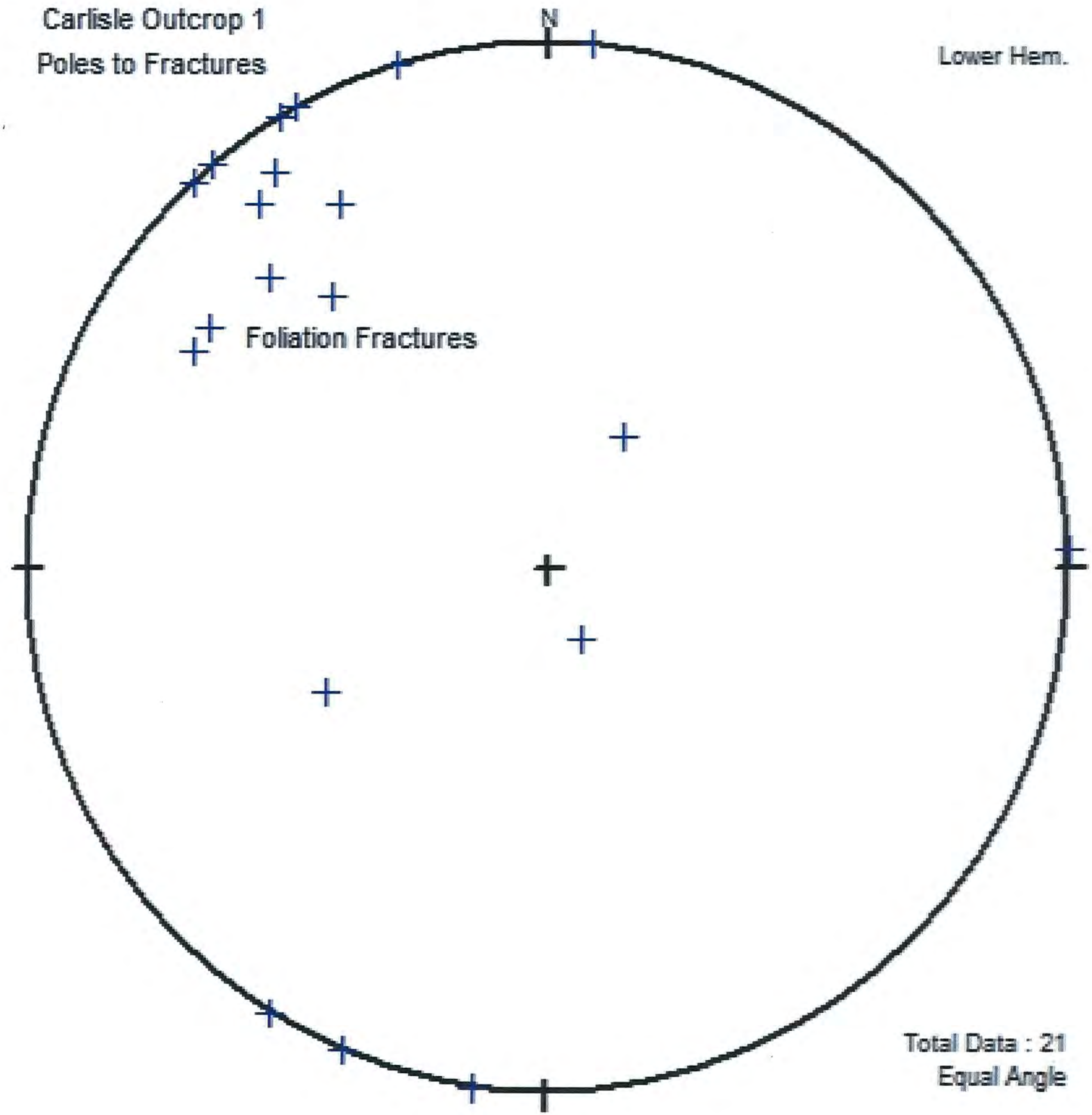
In Outcrop 1, the predominant fractures are those associated with metamorphic foliations. In this outcrop, these fractures strike northeastward (north 30 – 70 degrees east) and dip from 67 degrees to the southeast to vertical. Non-foliation fractures are less common and prominent in this outcrop, with strikes generally in the northwest/southeast quadrants and dips ranging from 17 degrees to vertical. Few steep, northwest trending fractures that cross foliation were observed, although these may occur in the gaps between exposed segments of the outcrops. A rose diagram depicting Outcrop 1 fracture strikes is found on Figure 3; the rose diagram and a lower hemisphere stereo plot of the poles to the measured fractures follow this text.

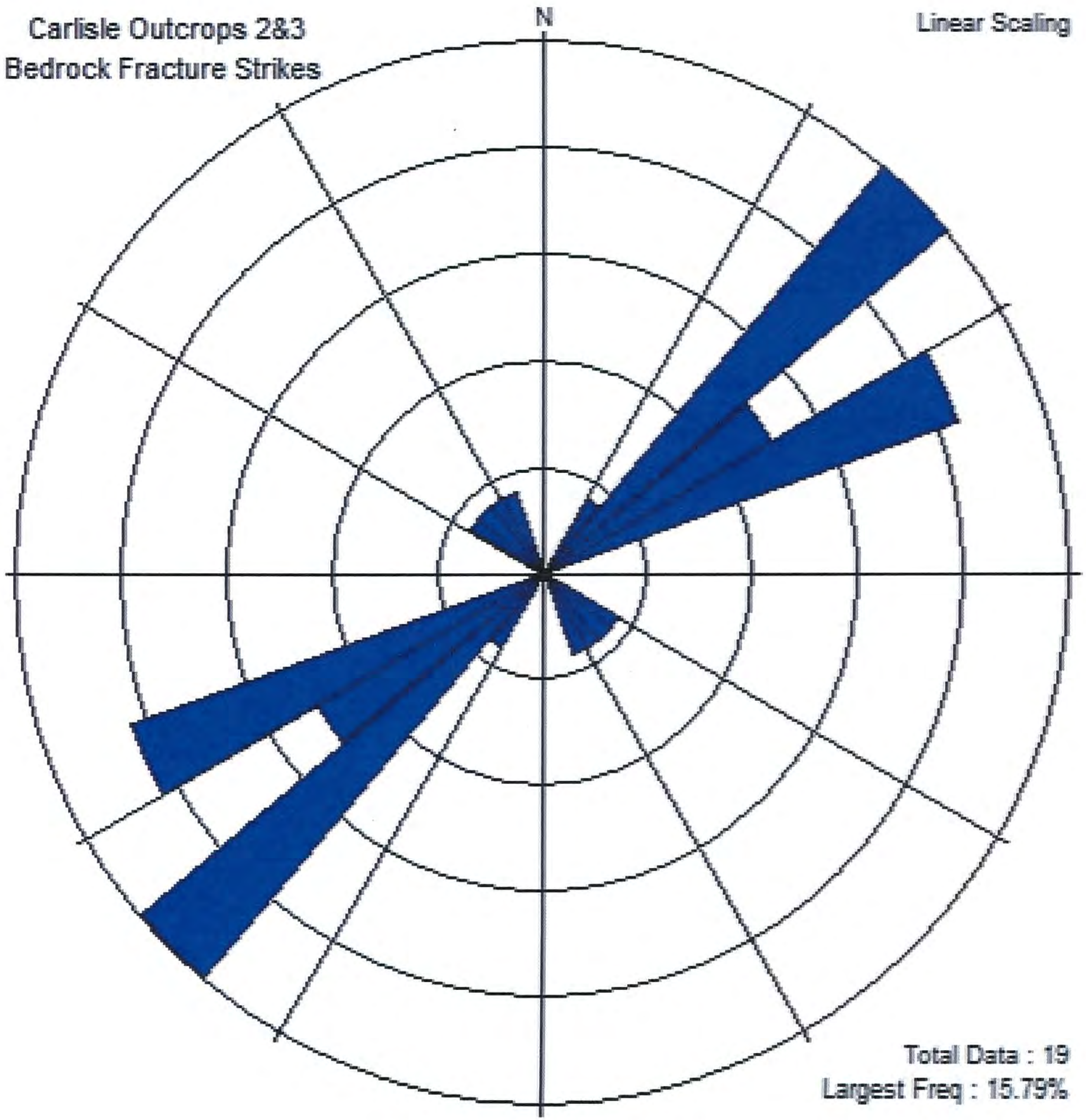
In Outcrop 2, the most common fracture strike direction is northwest (north 20 – 50 degrees west), with dip steep to the northeast or vertical. In the southern, more schistose end of the outcrop, northeast-striking foliation fractures are present.

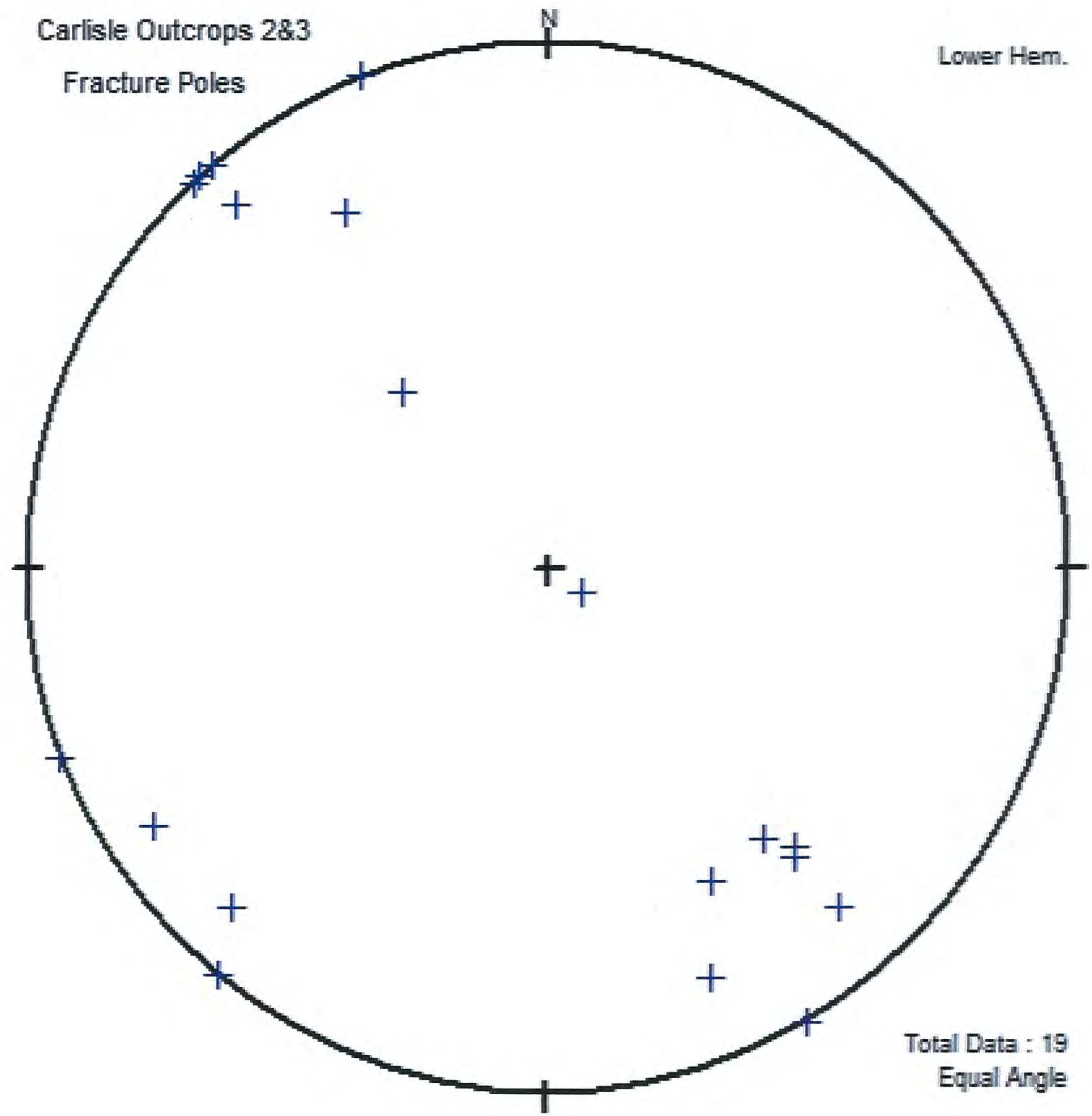
In Outcrop 3, most fractures are steep, strike either to the northwest or the northeast, and are not obviously associated with metamorphic foliation, which is indistinct in this outcrop. One exception is a vertical foliation fracture that strikes 84 degrees in the southeastern, more schistose portion of the outcrop. Fractures from Outcrops 2 and 3 combined are shown in a rose diagram on Figure 3 and following this text, and show that northeast-striking (north 40 – 70 degrees east) fractures still predominate, even if they are not obviously coincident with foliation in these outcrops. Steep fractures that strike northwest (north 20 – 60 degrees west, perpendicular to the northeast-striking fractures) are present, but less common than the northeast-striking fractures. A lower hemisphere stereonet plot of poles to the fractures in Outcrops 2 and 3 (part of this Attachment) shows that most of the fractures have moderate to steep dips.

In summary, steep, northeast-striking fractures are the most commonly observed fractures in these outcrops, whether the rock is prominently foliated or not. Steep, northwest-striking fractures are somewhat common in the non-foliated rocks, but only rarely observed in the foliated rocks. Scattered fractures with shallower dips and other strike directions are also present. These results differ somewhat from the results of photolineament analysis, documented in Nobis' Phase 1 report (see Phase 1 report, Figure 3). The most common photolineament trend is northwest. Because some steep, northwest-striking fractures are present in the outcrops, the photolineaments may represent fracture traces, and the differences in predominant orientations between photolineaments and outcrops may be scale related. Based on the photolineament and outcrop data, northeast and northwest appear to be the most likely directions for groundwater flow in fractured bedrock in the area.


Carlisle Outcrop 1
Bedrock Fracture Strikes
N
Linear Scaling
Foliation Fractures
Total Data : 21
Largest Freq : 14.29%


Carlisle Outcrop 1
Poles to Fractures
N
Lower Hem.
Foliation Fractures
Total Data : 21
Equal Angle


Carlisle Outcrops 2&3
Bedrock Fracture Strikes
N
Linear Scaling
Total Data : 19
Largest Freq : 15.79%



Carlisle Outcrops 2&3
Fracture Poles
Lower Hem.
N
Total Data : 19
Equal Angle

ATTACHMENT 2 – SOIL ANALYSIS DATA

James Vernon

From:	Tomei, Joseph D <jdt@geotesting.com>
Sent:	Monday, April 20, 2015 7:36 AM
To:	James Vernon
Cc:	Marro, Ethan; samplereceiving
Subject:	Carlisle Hydrogeologic Study, LGH 40B proposal
Attachments:	Results 042015.pdf

James, test results for organic content and grain size are attached. Based on the gradation of the samples (high fines content, significant gravel content and slight presence of organics) an accurate estimate of the permeability of these samples is not possible. My best guess is a permeability range of 5.0×10^{-3} cm/sec to 1.0×10^{-5} cm/sec. The in-place density of the material would also factor considerably into the permeability rate.

14.2 ft/d 2.8 x 10^{-2} ft/d

I would recommend a permeability test be conducted if an accurate permeability value is required for your project analysis. If you choose to have a perm test performed, please provide the required density for the sample(s) to be compacted to. Thanks.

Joe Tomei
Laboratory Manager
GeoTesting Express
125 Nagog Park | Acton, MA 01720
Ph: 978-635-0424 | Direct Line: 978-893-1241 | Mobile: 978-505-1962

jdt@geotesting.com
www.geotesting.com

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Client:	Nobis Engineering, Inc.				
Project:	Carlisle Hydrogeologic Study, LGH 40B proposal				
Location:	Carlisle, MA			Project No:	GTX-303037
Boring ID:	---	Sample Type:	---	Tested By:	cam
Sample ID:	---	Test Date:	04/16/15	Checked By:	jdt
Depth :	---	Test Id:	327825		

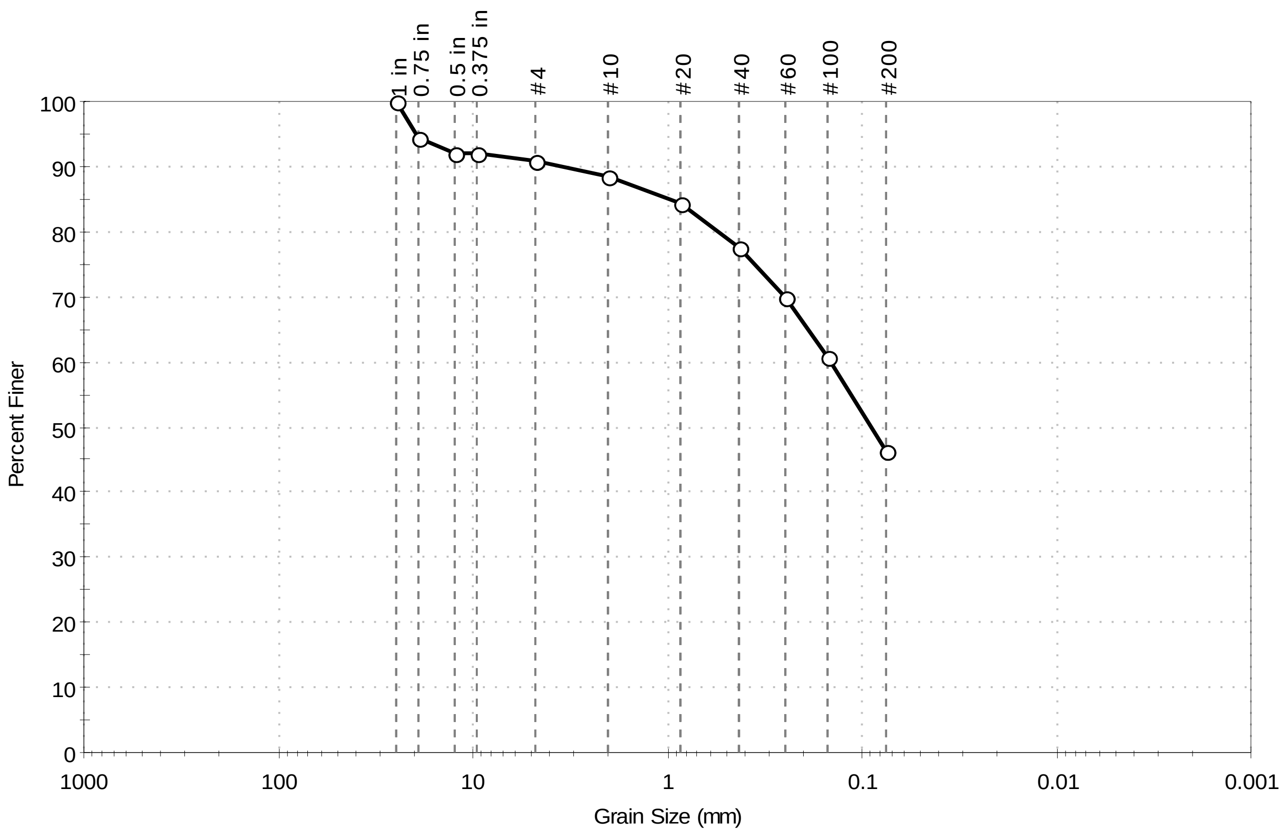
Moisture, Ash, and Organic Matter - ASTM D2974

Boring ID	Sample ID	Depth	Description	Moisture Content, %	Ash Content, %	Organic Matter, %
1	Carlisle 1	21.5 in	Moist, dark yellowish brown silty sand	32	98.3	1.7
2	Carlisle 2	23 in	Moist, light olive brown clayey sand with gravel	24	96.9	3.1

Notes: Moisture content determined by Method A and reported as a percentage of oven-dried mass; dried to a constant mass at temperature of 105º C
Ash content and organic matter determined by Method C; dried to constant mass at temperature 440º C

Client:	Nobis Engineering, Inc.		
Project:	Carlisle Hydrogeologic Study, LGH 40B proposal		
Location:	Carlisle, MA	Project No:	GTX-303037
Boring ID: 1	Sample Type: bag	Tested By:	jbr
Sample ID: Carlisle 1	Test Date: 04/15/15	Checked By:	jdt
Depth : 21.5 in	Test Id: 327822		
Test Comment:	---		
Sample Description:	Moist, dark yellowish brown silty sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	9.1	44.5	46.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	94		
0.5 in	12.50	92		
0.375 in	9.50	92		
#4	4.75	91		
#10	2.00	89		
#20	0.85	84		
#40	0.42	78		
#60	0.25	70		
#100	0.15	61		
#200	0.075	46		

Coefficients

D_{85} = 0.9874 mm D_{30} = N/A

D_{60} = 0.1451 mm D_{15} = N/A

D_{50} = 0.0893 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

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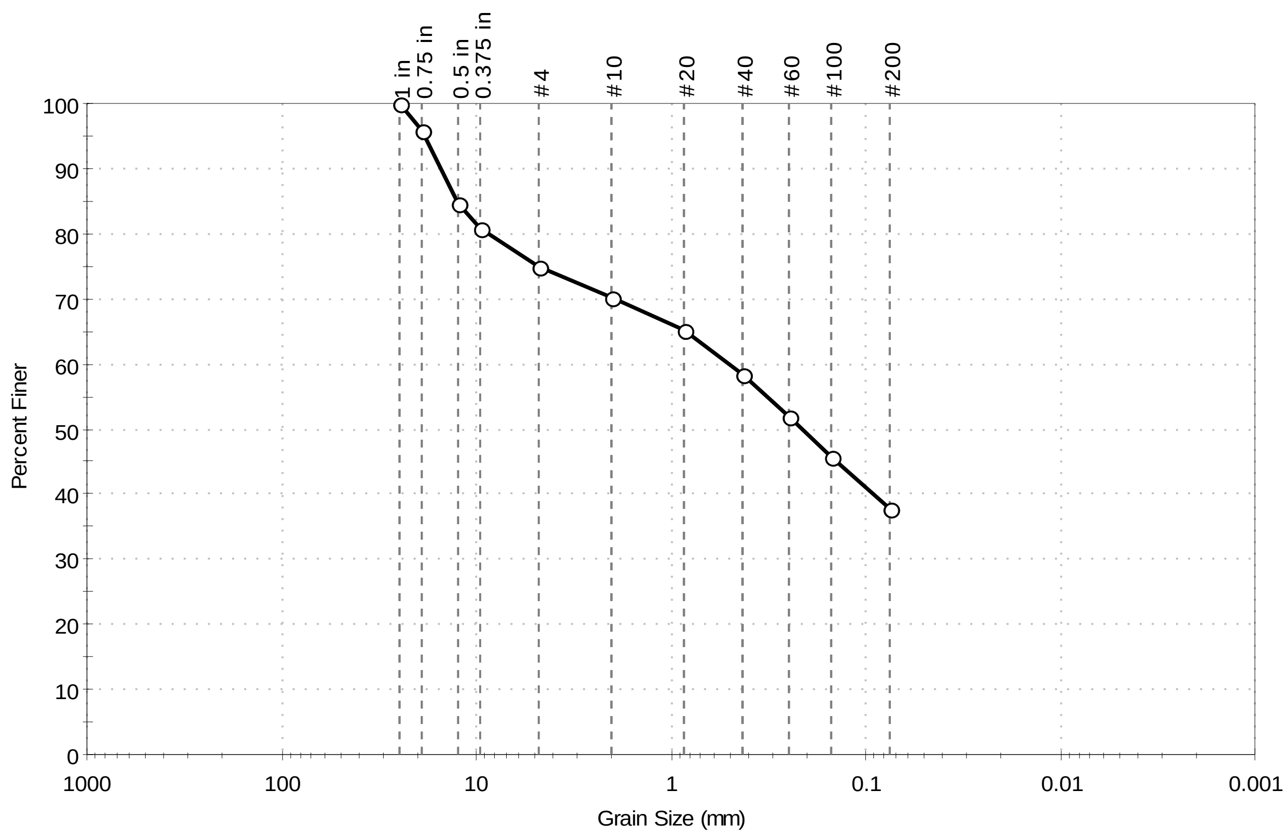
Client:	Nobis Engineering, Inc.
Project:	Carlisle Hydrogeologic Study, LGH 40B proposal
Location:	Carlisle, MA
Project No:	GTX-303037
Boring ID:	2
Sample Type:	bag
Tested By:	jbr
Sample ID:	Carlisle 2
Test Date:	04/15/15
Checked By:	jdt
Depth :	23 in
Test Id:	327823
Test Comment:	---
Sample Description:	Moist, light olive brown silty sand with gravel
Sample Comment:	---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	25.2	37.1	37.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	96		
0.5 in	12.50	85		
0.375 in	9.50	81		
#4	4.75	75		
#10	2.00	70		
#20	0.85	65		
#40	0.42	58		
#60	0.25	52		
#100	0.15	46		
#200	0.075	38		

Coefficients

D_{85} = 12.6076 mm D_{30} = N/A

D_{60} = 0.5024 mm D_{15} = N/A

D_{50} = 0.2129 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

printed 4/20/2015 7:24:01 AM

ATTACHMENT 3 – SEPTIC SYSTEM DESIGN FLOW

Attachment B

Brem_145_01.05.2015

The Birches 1/5/2015

Computation of Sewage Flows

165gpd/br

110gpd/br

Proposed Well (See Figure 2)

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Septic System 1:

Unit	1	3 BR	495	330	GPD	A11
	2	3 BR	495	330	GPD	A10
	3	3 BR	495	330	GPD	A10
	4	3 BR	495	330	GPD	A1
	5	3 BR	495	330	GPD	A1
	6	3 BR	495	330	GPD	A2
		18BR	2970	1980	GPD	

Septic System 2:

Unit	7	3 BR	495	330	GPD	A2
	8	3 BR	495	330	GPD	A3
	9	3 BR	495	330	GPD	A3
	10	3 BR	495	330	GPD	A4
	18	3 BR	495	330	GPD	A8
	19	3 BR	495	330	GPD	A9
		18BR	2970	1980	GPD	

Septic System 3:

Unit	11	2 BR	330	220	GPD	A4
	12	2 BR	330	220	GPD	A5
	13	3 BR	495	330	GPD	A5
	14	3 BR	495	330	GPD	A6
	15	3 BR	495	330	GPD	A6
	16	3 BR	495	330	GPD	A7
	17	2 BR	330	220	GPD	A8
		18BR	2970	1980	GPD	

20 (existing home) ? A9 (see below)

Annotations by Nobis 2/13/15 (Well info from "Neighborhood Well and Septic Exhibit" by LGH, dated 11/4/14, updated 1/8/15)

Notes: ① number of bedrooms for existing home unknown, so flow (design) to existing septic system unknown. Assume 4 bedrooms; 660 gpd

② LGH assumed 110gpd/bedroom; Carlisle BOH considering 165gpd/bedroom

ATTACHMENT 4 – NITRATE MASS BALANCE CALCULATIONS

Nitrate Loading Analysis
100 Long Ridge Road, Carlisle, Massachusetts
Nobis Engineering, Inc.
Apr-15

Input Type	Input Source	Applicant	Town	Comments
Wastewater Daily Volume (gpd)	Proposed Septic Area 1	1980	2970	Applicant assumes 110 gpd per bedroom; Town assumes 165 gpd per bedroom; see note below
	Proposed Septic Areas 2/3	3960	5940	
	Existing House	165	660	Applicant assumes 3 persons X 55gpd per person; Town assumes 4 bedrooms X 165 gpd per bedroom
Wastewater Nitrate Concentration (mg/L)	Proposed Septic Area 1	19	19	Concentrations from MassDEP Guidance
	Proposed Septic Areas 2/3	19	19	
	Existing House	35	35	
Nitrate load from fertilizer (mg/yr)	5000 sq ft of lawn (per applicant)	34019428		Using applicant's area and DEP load rate of 3 lb/1000 sq ft; includes leach rate of 0.25
Recharge from precipitation (in/yr)	20% of annual average precipitation	8.16		Stormwater neglected from calculations because will be recharged on site.

Notes

1. Wastewater Daily Volume for Applicant is based on Brem_145_01052015, Computation of Sewage Flows, which used 110 gpd per bedroom. However, NGI's Mass Balance Mass Loading Analysis in NGI 3/25/15 report (Brem 193-03-25-2015, assumed 55 gpd per person and 3 persons per house.
2. Assume fertlizer applied to 0.11 acres of lawn at 3 lb per 1000 sq ft

Nitrate Loading Analysis
100 Long Ridge Road, Carlisle, Massachusetts
Nobis Engineering, Inc.
Apr-15

Input Type	Input Source	Applicant	Town		Applicant	Town	Comments
Wastewater Daily Volume (gal/day)	Proposed Septic Area 1	1980	2970	Wastewater Daily Volume (L/yr)	2735708.58	4103562.87	Applicant assumes 110 gpd per bedroom; Town assumes 165 gpd per bedroom; see note below
	Proposed Septic Areas 2/3	3960	5940		5471417.16	8207125.74	
	Existing House	165	660		227975.715	911902.86	Applicant assumes 3 persons X 55gpd per person; Town assumes 4 bedrooms X 165 gpd per bedroom
Wastewater Nitrate Concentration (mg/L)	Proposed Septic Area 1	19	19				Concentrations from MassDEP Guidance
	Proposed Septic Areas 2/3	19	19				
	Existing House	35	35				
Nitrate load from fertilizer (mg/yr)	5000 sq ft of lawn (per applicant)	34019428					Using applicant's area and DEP load rate of 3 lb/1000 sq ft; includes leach rate of 0.25
Recharge from precipitation (in/yr)	20% of annual average precipitation	8.16		Recharge from precipitation (ft/yr)	0.68		Stormwater neglected from calculations because will be recharged on site.

Notes

1. Wastewater Daily Volume for Applicant is based on Brem_145_01052015, Computation of Sewage Flows, which used 110 gpd per bedroom. However, NGI's Mass Balance Mass Loading Analysis in NGI 3/25/15 report (Brem 193-03-25-2015, assumed 55 gpd per person and 3 persons per house.
2. Assume fertlizer applied to 0.11 acres of lawn at 3 lb per 1000 sq ft

Scenario 1
Entire Site and Applicant Values

$$\frac{\textbf{Load}}{\textbf{Volume}} \quad \frac{\text{[Wastewater Yearly Volume (L/yr) x Wastewater Nitrate Concentration (mg/L)] + Nitrate load from fertilizer (mg/yr)}}{\text{Wastewater Yearly Volume (L/yr) + ([Recharge from precipitation (ft/yr) x Area (ft}^2\text{)] x gal/ft}^3\text{ x L/gal)}} = \frac{\text{mg}}{\text{L}}$$

Variables

Wastewater Yearly Volume Area 1 (L/yr)	=	2735709
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	5471417
Wastewater Yearly Volume Existing (L/yr)	=	227975.7
Area (ft^2)	=	428630.4

Constants

Wastewater Nitrate Concentration Area 1 (mg/L)	=	19
Wastewater Nitrate Concentration Areas 2/3 (mg/L)	=	19
Wastewater Nitrate Concentration Existing (mg/L)	=	35
Nitrate Load from Fertilizer (mg/yr)	=	34019428
Recharge from Precipitation (ft/yr)	=	0.68

$$\text{Concentration} = \frac{\textbf{Load (mg/yr)}}{\textbf{Volume (L/yr)}} = \frac{1.98\text{E}+08}{16687976} = \boxed{11.9 \text{ mg/L}}$$

Scenario 2
Entire Site and Town Values

$$\frac{\textbf{Load}}{\textbf{Volume}} \quad \frac{\text{[Wastewater Yearly Volume (L/yr) x Wastewater Nitrate Concentration (mg/L)] + Nitrate load from fertilizer (mg/yr)}}{\text{Wastewater Yearly Volume (L/yr) + ([Recharge from precipitation (ft/yr) x Area (ft}^2\text{)] x gal/ft}^3\text{ x L/gal)}} = \frac{\text{mg}}{\text{L}}$$

Variables

Wastewater Yearly Volume Area 1 (L/yr)	=	4103563
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	8207126
Wastewater Yearly Volume Existing (L/yr)	=	911902.9
Area (ft^2)	=	428630.4

Constants

Wastewater Nitrate Concentration Area 1 (mg/L)	=	19
Wastewater Nitrate Concentration Areas 2/3 (mg/L)	=	19
Wastewater Nitrate Concentration Existing (mg/L)	=	35
Nitrate Load from Fertilizer (mg/yr)	=	34019428
Recharge from Precipitation (ft/yr)	=	0.68

$$\text{Concentration} = \frac{\textbf{Load (mg/yr)}}{\textbf{Volume (L/yr)}} = \frac{3E+08}{21475466} = \boxed{14.0 \text{ mg/L}}$$

Scenario 3
Site West of the Brook, Applicant Values

$$\frac{\textbf{Load}}{\textbf{Volume}} \quad \frac{[\text{Wastewater Yearly Volume (L/yr)} \times \text{Wastewater Nitrate Concentration (mg/L)}] + \text{Nitrate load from fertilizer (mg/yr)}}{\text{Wastewater Yearly Volume (L/yr)} + ([\text{Recharge from precipitation (ft/yr)} \times \text{Area } (ft^2)] \times gal/ft^3 \times L/gal)} = \frac{mg}{L}$$

Variables

Wastewater Yearly Volume Area 1 (L/yr)	=	2735709
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	5471417
Wastewater Yearly Volume Existing (L/yr)	=	227975.7
Area (ft^2)	=	317447

Constants

Wastewater Nitrate Concentration Area 1 (mg/L)	=	19
Wastewater Nitrate Concentration Areas 2/3 (mg/L)	=	19
Wastewater Nitrate Concentration Existing (mg/L)	=	35
Nitrate Load from Fertilizer (mg/yr)	=	34019428
Recharge from Precipitation (ft/yr)	=	0.68

$$\text{Concentration} = \frac{\textbf{Load (mg/yr)}}{\textbf{Volume (L/yr)}} = \frac{1.98E+08}{14547245} = \boxed{13.6 \text{ mg/L}}$$

Scenario 4
Site West of the Brook, Town Values

$$\frac{\textbf{Load}}{\textbf{Volume}} \quad \frac{[\text{Wastewater Yearly Volume (L/yr) x Wastewater Nitrate Concentration (mg/L)}] + \text{Nitrate load from fertilizer (mg/yr)}}{\text{Wastewater Yearly Volume (L/yr)} + ([\text{Recharge from precipitation (ft/yr) x Area (ft}^2)] \text{ x gal/ft}^3 \text{ x L/gal)}} = \frac{\text{mg}}{\text{L}}$$

Variables

Wastewater Yearly Volume Area 1 (L/yr)	=	4103563
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	8207126
Wastewater Yearly Volume Existing (L/yr)	=	911902.9
Area (ft^2)	=	317447

Constants

Wastewater Nitrate Concentration Area 1 (mg/L)	=	19
Wastewater Nitrate Concentration Areas 2/3 (mg/L)	=	19
Wastewater Nitrate Concentration Existing (mg/L)	=	35
Nitrate Load from Fertilizer (mg/yr)	=	34019428
Recharge from Precipitation (ft/yr)	=	0.68

$$\text{Concentration} = \frac{\textbf{Load (mg/yr)}}{\textbf{Volume (L/yr)}} = \frac{3E+08}{19334735} = \boxed{15.5 \text{ mg/L}}$$

TABLE 4

MASS-BALANCE NITRATE LOADING ANALYSES - MULTIPLE SCENARIOS

100 Long Ridge Road
Carlisle, Massachusetts

Scenario 1 — **Variables - Applicant Assumptions**

Wastewater Yearly Volume Area 1 (L/yr)	=	W_{A1}	=	2735708.58
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	$W_{A1/2}$	=	5471417.16
Wastewater Yearly Volume Existing (L/yr)	=	W_E	=	227975.715
Entire Site Area (ft^2)		A_S	=	428630.4

	$(W_{A1} \times N_{A1}$	+	$W_{A1/2} \times N_{A1/2}$	+	$W_E \times N_E)$	+	N_F			=	Load		
Load	51978463.02		103956926		7979150		34019428			=	197933967.1	mg	
													= **11.9 mg/L Scenario 1**
Volume	$(W_{A1}$	+	$W_{A1/2}$	+	$W_E)$	+	$(R_P \times A_S$	x	7.48 gal / ft^3 x 3.79 L / gal)	=	Volume		
	2735708.58		5471417.16		227975.7		291468.7	x	28.31	=	16687976.28	L	

Scenario 2 — **Variables - Town Assumptions**

Wastewater Yearly Volume Area 1 (L/yr)	=	W_{A1}	=	4103562.87
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	$W_{A1/2}$	=	8207125.74
Wastewater Yearly Volume Existing (L/yr)	=	W_E	=	911902.86
Entire Site Area (ft^2)	=	A_S	=	428630.4

	$(W_{A1} \times N_{A1}$	+	$W_{A1/2} \times N_{A1/2}$	+	$W_E \times N_E)$	+	N_F			=	Load		
Load	77967694.53		155935389.1		31916600		34019428			=	299839111.7	mg	
													= **14.0 mg/L Scenario 2**
Volume	$(W_{A1}$	+	$W_{A1/2}$	+	$W_E)$	+	$(R_P \times A_S$	x	7.48 gal / ft^3 x 3.79 L / gal)	=	Volume		
	4103562.87		8207125.74		911902.9		291468.7	x	28.31	=	21475466.29	L	

Scenario 3 — **Variables - Applicant Assumptions**

Wastewater Yearly Volume Area 1 (L/yr)	=	W_{A1}	=	2735708.58
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	$W_{A1/2}$	=	5471417.16
Wastewater Yearly Volume Existing (L/yr)	=	W_E	=	227975.715
Site West of the Brook Only Area (ft^2)	=	A_W	=	317447

	$(W_{A1} \times N_{A1}$	+	$W_{A1/2} \times N_{A1/2}$	+	$W_E \times N_E)$	+	N_F			=	Load		
Load	51978463.02		103956926		7979150		34019428			=	197933967.1	mg	
													= **13.6 mg/L Scenario 3**
Volume	$(W_{A1}$	+	$W_{A1/2}$	+	$W_E)$	+	$(R_P \times A_W$	x	7.48 gal / ft^3 x 3.79 L / gal)	=	Volume		
	2735708.58		5471417.16		227975.7		215864	x	28.31	=	14547244.58	L	

Scenario 4 — **Variables - Town Assumptions**

Wastewater Yearly Volume Area 1 (L/yr)	=	W_{A1}	=	4103562.87
Wastewater Yearly Volume Areas 2/3 (L/yr)	=	$W_{A1/2}$	=	8207125.74
Wastewater Yearly Volume Existing (L/yr)	=	W_E	=	911902.86
Site West of the Brook Only Area (ft^2)	=	A_W	=	317447

	$(W_{A1} \times N_{A1}$	+	$W_{A1/2} \times N_{A1/2}$	+	$W_E \times N_E)$	+	N_F			=	Load		
Load	77967694.53		155935389.1		31916600		34019428			=	299839111.7	mg	
													= **15.5 mg/L Scenario 4**
Volume	$(W_{A1}$	+	$W_{A1/2}$	+	$W_E)$	+	$(R_P \times A_W$	x	7.48 gal / ft^3 x 3.79 L / gal)	=	Volume		
	4103562.87		8207125.74		911902.9		215864	x	28.31	=	19334734.6	L	

Scenario Constants

Wastewater Nitrate Concentration Area 1 (mg/L)	=	N_{A1}	=	19
Wastewater Nitrate Concentration Areas 2/3 (mg/L)	=	$N_{A2/3}$	=	19
Wastewater Nitrate Concentration Existing (mg/L)	=	N_E	=	35
Nitrate Load from Fertilizer (mg/yr)	=	N_F	=	34019428
Recharge from Precipitation (ft/yr)	=	R_P	=	0.68

ATTACHMENT 5 – NITRATE DISPERSION ANALYSIS TECHNICAL MEMORANDUM

MEMORANDUM

To: File 89220

From: J. Lambert

Subject: Nitrate dispersion analysis, 100 Long Ridge Road, Carlisle, MA

Date: April 20, 2015

This memorandum presents the results of groundwater modeling to evaluate potential overburden nitrate concentrations at downgradient property lines and at sensitive receptors, including two locations on the brook in the eastern portion of the property and seven existing and proposed water supply well locations. The memorandum first describes the analytical model used and the selected parameters, then describes the results of the modeling.

Groundwater Transport Model

The groundwater advection/dispersion numerical model used is based on Domenico, 1987 and shown below:

$$C(X,0,0,t) = \frac{C_0}{2} exp\left\{\left(\frac{X}{2\alpha_x}\right)\left[1-\left(1+\frac{4\lambda\alpha_x}{v_x}\right)^{\frac{1}{2}}\right]\right\} erfc\left\{\frac{X - v_x t\left(1+\frac{4\lambda\alpha_x}{v_x}\right)^{\frac{1}{2}}}{2(\alpha_x v_x t)^{\frac{1}{2}}}\right\}$$

$$erf\left\{\frac{Y}{4(\alpha_y x)^{\frac{1}{2}}}\right\} erf\left\{\frac{Z}{2(\alpha_z x)^{\frac{1}{2}}}\right\}$$

Where:

C = predicted concentration at given location (x,0,0) and time (t)

C_0 = initial concentration at source

X = distance from source (source dimension in the x direction)

α_x = longitudinal dispersivity

λ = decay constant (λ = 0.693/half-life)

v_x = groundwater velocity

α_y = horizontal transverse dispersivity

α_z = vertical transverse dispersivity

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18 Chenell Drive
Concord, NH 03301
T (603) 224-4182

Y = source dimension in the y direction

Z = source dimension in the z direction

In this model, nitrate is assumed to be conservative; therefore, retardation is not incorporated into the equation and the decay constant is 0. This simplifies the equation to the following:

$$C(X,0,0,t) = \frac{C_0}{2} erfc\left\{\frac{X - v_x t}{2(\alpha_x v_x t)^{\frac{1}{2}}}\right\} erf\left\{\frac{Y}{4\left(\alpha_y x\right)^{\frac{1}{2}}}\right\} erf\left\{\frac{Z}{2(\alpha_z x)^{\frac{1}{2}}}\right\}$$

Parameters Used

The Groundwater Impact Analysis (NGI Report, 2015) developed transport parameters from site data and/or using inferred values based on local conditions. These parameters were evaluated individually, and in some cases area-specific parameters were used instead for the modeling. The selected parameters and their use in the model are described below. Units were converted to meters for calculations.

Hydraulic Conductivity

NGI, 2015 set the hydraulic conductivity as the geometric mean of observed slug test values (9 feet/day). Instead of using a bulk hydraulic conductivity for the entire site, we used the values associated with each septic disposal area. The revised hydraulic conductivities are as follows:

- Septic disposal area 1 (flow lines 1 through 6): 19.1 feet/day (average of MW-1 and MW-1A)
- Septic disposal area 2/3 and nearby flow lines (flow lines 7, 11, and 12): 10.3 feet/day (average of MW-2, MW-2A, MW-3, and MW-3A)
- Septic disposal area 2/3 and MW-4 (flow lines 8 and 9): 8.7 feet/day
- Septic disposal area 2/3 and MW-5 (flow line 10): 9.5 feet/day

Nitrate Loading/Dilution

Nobis used a starting nitrate concentration of 19 mg/L for the Proposed Septic Disposal Areas, consistent with Mass DEP Guidance. Nobis did not model nitrate dispersion from the existing septic system that serves the existing house.

Effective Porosity

NGI, 2015 set the effective porosity to be 0.2 based on literature values; Nobis retained this value.

Soil Bulk Density

NGI, 2015 set the bulk density to be 1.7 kg/L based on literature values; Nobis retained this value.

Dispersivity

NGI, 2015 set the longitudinal dispersivity (α_X) of 10% of the plume length, transverse dispersivity (α_Y) to 10% of α_X, and vertical dispersivity (α_Z) to 10% of α_Y. Nobis used the same dispersivity assumptions in one set of calculations, but the calculations presented here use alternate dispersivity calculations according to Xu and Ekstein (1995). The equation used is provided below.

$$\alpha_X = 0.83[log_{10}(X)]^{2.414}$$

According to Xu and Ekstein, the longitudinal dispersivity is a logarithmic function of distance along the flow line, instead of a linear function. With the linear dispersivity coefficients, predicted nitrate concentrations were somewhat lower. Nobis believes the logarithmic results are more accurate.

Decay Coefficient

NGI used a first-order decay coefficient (λ) of 0.0000344 hr^{-1} based on a half-life of 2.3 years. Nobis assumed that the nitrate would be conservative; that is it would have a λ of 0.

Flow Line Length

Flow line length (X) was determined for each segment shown on Table 1. The flow line was assumed to start at the center of the edge of each septic disposal area closest to the target. Flow lines are shown on Figure 1.

Source Dimensions

The source width (Y) was assumed to be the edge of the septic disposal area perpendicular to each flow line. No attempt was made to correct for the angle of the source for flow lines that were not perpendicular to the proposed septic disposal areas. Septic system 1 has a planned length (east-west) of 55.95 feet and width (north-south) of 52.15 feet. Septic system 2 and 3, which are intended to be next to each other, were considered to be a single source for the model. The dimensions of the combined hypothetical system are 110 feet northwest-southeast, and 50.6 feet northeast-southwest.

NGI assumed the source thickness (Z) to be 6 feet; this assumption was carried through.

Results

See Table 5 for results. The results show predicted nitrate concentrations of 3.8 mg/L, 17.8 mg/L, and 8.4 mg/L at three property line locations (Flow Lines 1, 7, and 12). The results show predicted nitrate concentrations of 1.4 mg/L and 0.8 mg/L at two staff gauge locations on the brook (Flow Lines 4 and 10). The results show predicted nitrate concentrations of 1.7 to 4.7 mg/L **in the overburden** at three new proposed on-Site well locations. The results show predicted nitrate concentrations of 1.4 to 3.4 mg/L **in the overburden** at four existing neighbors' well locations.

References

Domenico, P. A. (1987). An analytical model for multidimensional transport of decaying contaminant species. Journal of Hydrology, 91: 49-58.

Xu, M. and Eckstein, Y. 1995. Use of weighted least-squares method in evaluation of the relationship between dispersivity and field scale. Ground Water, vol. 3, no. 6, pg 905-908.