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Volume 1

Engineer's Report
for
"FINGER LAKES STORAGE BRINE POND"

Town of Reading
Schuyler County, New York

Prepared for:

Finger Lakes Storage, LLC

Prepared by:
C.T. MALE ASSOCIATES, P.C.
50 Century Hill Drive
Latham, New York 12110
518-786-7400
FAX 518-786-7299

C.T. Male Project No: 08.8696
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ENGINEER'S REPORT
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1.0 INTRODUCTION / SITE ANALYSIS

1.1 Site Description

The project site is located immediately east (downhill) of the intersection of New York State Routes 14 and 14A in the Town of Reading, New York. The Village of Watkins Glen is located approximately 3 miles to the southeast and the Town of Reading approximately 1.5 miles west (uphill) of the site. Currently the site is an undeveloped area with significant vegetative growth across much of its surface. The existing ground surface of the area proposed for the brine pond's construction slopes downward to the east at an average inclination of 6 to 8 percent. Less than ½-mile downhill (east) of the site is Seneca Lake.

As shown on Figure 1, "Vicinity Map", there are several residences located near the project site, with the nearest being approximately 500 feet west (uphill) on the southbound side of State Route 14. No residences are located downhill of the project site. Land on the site's downhill side is owned by U.S. Salt Corporation. Norfolk-Southern Railway holds an easement adjacent and parallel to the edge of Seneca Lake, along which the railway runs. In addition, Conrail has an easement located approximately one-half to three-quarters of a mile west (uphill) of the project site.

One (1) public water supply well, identified by DEC as SY921, is located approximately 500 feet south of the site. No other known public water supply wells or aquifers are located within a one-mile radius of the site. All mapped DEC surface waters within the same radius are identified on Figure 1.

1.2 Project Description

Finger Lakes Storage is currently proposing to construct a surface impoundment (pond) that is designed to contain a brine solution. The brine solution will be removed from nearby subsurface caverns to develop storage for liquid petroleum gas (LPG). The amount of brine solution in the pond will be dependent on the volume of gas being stored in the underground caverns. During the fall season, the brine pond will be its fullest as gas storage at that time will be at its maximum volume in anticipation of the winter heating season. Conversely, the surface elevation of the brine solution will be the lowest in the pond in the spring when much of the solution has been pumped into the underground caverns to displace and transmit the gas through above ground piping to satisfy winter heating demands.

The project sponsor has indicated that the brine pond needs to have a minimum volume of 2.1 million barrels (88.2 million gallons) in order to meet the anticipated gas demand.

2.0 EXISTING CONDITIONS ASSESSMENT

2.1 Surface Conditions

2.1.1 Land Cover

The project site's land cover is mostly brush with some patches and rows of trees. There are a few pathways that have been cut through the stand of brush and trees to facilitate access to groundwater monitoring wells.

2.1.2 Drainage Patterns

Three (3) well defined drainage swales cross the site from west to east. The primary source of runoff to these swales is from uphill areas located west of the project site. Runoff from these areas is piped through culverts that pass under State Route 14 and the entrance/exit ramps of State Route 14A. In addition, the runoff from a portion of these roads (State Route 14 and the entrance ramp to State Route 14A) drains across the project site. On the project site's downhill side, water in the swales flows through culverts under a gravel road and eventually under the railroad, both of which are oriented in a north-south direction.

2.1.3 Visibility

The entrance ramp to State Route 14A and the north-bound lane of State Route 14 is visible from the pond site. It borders the western two-thirds of the site and runs along an embankment that elevates the ramp above the site. Accordingly, the site's relatively thick cover of brush is visible to motorists traveling along the ramp. The site is also visible from Seneca Lake and the other side of the valley the lake lies in. From these vantage points, the brush cover of the southern two-thirds of the site stands in contrast to the open, hay covered field which occupies the site's northern third.

2.2 Subsurface Conditions

2.2.1 General

The project site's subsurface conditions were investigated through the advancement of test borings, excavation of test pits, installation of groundwater monitoring wells and the performance of field and laboratory tests. This work was conducted in two phases, with the first being performed to provide an overall characterization of the overburden and groundwater conditions present and determine the type of bedrock and its depth below grade. For the initial investigation, six (6) test borings were advanced and, at four (4) of these locations, groundwater monitoring wells installed. This work was supplemented by the performance of field and laboratory tests.

The second phase of the program was conducted to further define the overburden profile, bedrock elevations and groundwater conditions. It included the advancement of ten (10) additional test borings, installation of four (4) more groundwater monitoring wells, excavation of nine (9) test pits and the performance of additional field and laboratory tests. The field testing involved the performance of several slug tests to determine the permeability of subsurface conditions encountered.

A brief description of the subsurface conditions disclosed by these investigations is included in the following sections. For a full description of the methods of investigation and the subsurface conditions encountered at the site, the reader is referred to Volume 2 of this report, entitled *"Geotechnical Evaluation, Finger Lakes Storage Brine Pond"* dated January 11, 2011. Appendix A of the Volume 2 report contains the Subsurface Investigation Plan, Subsurface Profiles and a Groundwater Contour Plan.

2.2.2 Overburden & Bedrock

Approximately 2 to 10 inches of topsoil was encountered across the site and found to be underlain by a deposit of silt 3 to 9 feet in thickness. The silt deposit contained little to near equal amounts of clay, trace amounts of fine sand, and occasional cobbles. The number of cobbles and the amount of sand present within the deposit was found to typically increase with depth.

A thin sequence of sand and silt was encountered directly beneath the silt deposit at six (6) of the explored locations. At these locations, the sequence was found to have a thickness of less than 5 feet.

Found beneath the sequence of sand and silt, or the silt deposit where the former was not present, was glacial till. Embedded within the till's fine sand/silt soil matrix was medium to coarse sand and fine to coarse gravel. Numerous cobbles and boulders were also found to be present as evidenced by the difficulty in advancing the test borings with depth as well as their presence in many of the test pit excavations.

Slug testing, performed to assess the permeability of the overburden/weathered bedrock, was conducted at eight (8) of the monitoring well locations. Two (2) of these tests were performed in wells screened only within the glacial till. The other tests were performed in wells screened across the interface of the glacial till and underlying weathered rock. Pressure transducers were placed near the base of the wells to avoid contact with the slug. Data collected by the transducers during the performance of these tests were transmitted directly to a laptop computer and, upon inputting the well parameters, the data was analyzed with the assistance of the computer program AQTESOLV, Version 4.50. Permeability values were typically computed using the rising head data and methods developed by Cooper et al. (1967) and Bouwer & Rice (1976). Permeability values for the glacial till were found to range from 1.3×10^{-6} to 7.0×10^{-6} centimeters per second (cm/s) while the permeability values of the weathered rock were found to range from 1.2×10^{-4} to 1.3×10^{-2} centimeters per second (cm/s). Example output from several of the computer analyses are contained in Appendix B of this report (Volume 1).

For those test borings that fully penetrated the glacial till, bedrock deposit, an interbedded shale and siltstone, was encountered directly beneath the till at depths ranging from 16.5 to 33 feet. The bedrock was found to be weathered and broken to depths of 1 to 7 feet below its surface. Thereafter it became medium hard and sound. It was found to be thinly bedded at or near a horizontal orientation and to contain numerous fractures with intermittent soil seams within the depths explored.

Profiles of the subsurface conditions encountered at the site and the lines along which they were developed are presented in Appendix A of Volume 2 of this report. The Subsurface Exploration Logs, Test Pit Logs and Monitoring Well Construction Logs are presented in Appendix B of this report (Volume 1).

2.2.3 Groundwater Level

Groundwater levels in each of the monitoring wells were measured on several dates. The most recent observation was made on January 11, 2011; more than one month after slug testing was performed in several of the wells. On this date, the observed groundwater levels were considered to have stabilized and were found to be present 1.3 feet to 6.2 feet below the existing site grades. At one monitoring well, MW-16, a strong sulfurous odor was noted to be present during construction of the well. This odor was not noted during follow-up observations. It is not uncommon, however, for shale bedrock such as that encountered at the project site to contain sulfides which have a characteristic sulfurous odor.

Water was observed weeping into the test pit excavations through granular seams or partings within the overburden at depths ranging from 4 feet to 10 feet beneath the ground surface. At the test boring locations, water could be heard entering the auger casing once the augers fully penetrated the glacial till and were extended into the underlying layer of weathered/broken bedrock. Above these depths, seepage of water into the drill holes was not audible. The water levels in wells screened across the glacial till/weathered rock interface were typically within 6 to 12 inches of the levels observed in wells of the same couplet which were screened only in the overlying glacial till. Collectively these observations indicate that groundwater is contained in the layer of broken/weathered rock and confined by the overlying till which has a permeability 2 to 4 orders of magnitude less than that of the layer of broken/weathered rock. Groundwater is also contained within granular seams of the overburden which, at some locations, is perched above the piezometric surface of the confined groundwater table.

Appendix A of the Volume 2 report contains a Groundwater Contour Plan, Drawing SI-4, which was developed using the most recent groundwater level observations.

2.2.4 Groundwater Quality

Groundwater samples were collected from monitoring wells MW-1, MW-3, MW-4, MW-13 and MW-16 on January 12, 2011. The monitoring well locations are shown on the Subsurface Investigation Plan contained in Appendix A of the Volume 2 report. Prior to sampling, the water levels, as measured from the top of the PVC casing, were determined in each well utilizing a water level meter. The wells were developed by purging three (3) to five (5) well volumes utilizing a peristaltic pump and/or dedicated disposable bailer. Water levels within the wells were allowed to recover to a minimum of 90% of their pre-purging static water levels. Groundwater samples were collected in new laboratory supplied glass jars while wearing new gloves utilizing the peristaltic pump or bailer. New tubing for the pump was used at each of the well locations. The samples were placed in a cooler with ice and transported to Upstate Laboratories, Inc. in Syracuse, New York following proper chain of custody protocols. The samples were analyzed for 6NYCRR Part 360 baseline parameters. A field duplicate sample was collected from MW-4.

The following is a summary of the laboratory results:

- Volatile organic compounds (VOCs) were not detected in the groundwater samples. A table summarizing the laboratory results can be found in Appendix C (Table C-1).
- Several metals on a "totals" basis were detected in the monitoring wells. Total metal concentrations represent the total concentration of the metals in the groundwater samples without being filtered to remove suspended sediment in the samples prior to the samples being preserved with a fixative. The analytes detected included aluminum, barium, calcium, chromium, iron, magnesium, manganese, potassium, sodium, zinc and lead. Of the metals detected, aluminum, iron, magnesium, and sodium exceeded NYSDEC standards in some or all of the monitoring wells. Refer to Table C-2 in Appendix C.
- Table C-3 in Appendix C presents the laboratory results for dissolved metals. Dissolved metal concentrations represent the concentration of metals in the groundwater samples after being filtered to remove suspected sediments in the samples prior to being preserved with a fixative. The analytes detected in some or all the wells included barium, calcium, magnesium, manganese, sodium and zinc. Only magnesium and sodium exceeded their respective groundwater standards. As anticipated, their concentration was only slightly lower than the total concentration as these analytes are soluble in water.
- Table C-4 in Appendix C presents physical characteristics and field parameters and Table C-5 presents laboratory results for leachate indicators.

The values presented provide a baseline of the groundwater quality at the site. The analytes detected above NYSDEC standards are considered to be naturally occurring and not related to sources of contaminates within or nearby the project site. Elevated levels of metals in the "total" metals analysis are considered to be related to suspended sediments in the water samples collected and analyzed. This was confirmed through the analysis of "dissolved" metal whereby the samples were first filtered to remove the suspended sediments prior to being analyzed. A few metals (magnesium, sodium, etc.) above standards in the total and then again in the dissolved samples are typically not affected by filtering because they are very soluble in water.

A copy of the laboratory analytical results report is included in Appendix C of this report (Volume 1).

3.0 DESIGN ELEMENTS – BASIS OF DESIGN

The following elements are part of the design of the brine pond.

3.1 Pond Volume

The minimum desired storage volume of the brine pond is 2.1 million barrels. The pond is designed to hold 2.19 million barrels at its maximum operating pool elevation of 837.0 feet.

3.2 Embankment

In an attempt to fully utilize cut soils as fill material and approximately balance the earthwork required to construct the impoundment, an embankment will be constructed on the downhill (east) side of the brine pond. The top of this embankment will be established at elevation 841.0 feet (three (3) feet above the operating level of the pond) and, where the site is lowest in elevation, its toe will meet the existing site grades at elevation 790 feet. The resulting maximum embankment height will be 51 feet.

On the embankment's interior, its side slopes will be inclined at and between 1:4 and 1:3 (Vertical: Horizontal). As groundwater may weep from the cut slope made along the pond's west side, this slope will be inclined 1:4 to enhance its stability under such potential seepage conditions. Opposite this slope, the embankment will be a fill section and, as such, the interior slope of the embankment will not be subject to such seepage. The embankment's interior slope on this side of the impoundment will be 1:3 (V:H). Along the north and south ends of the pond between its east and west sides, the embankment's interior side slopes will gradually transition between these inclinations of 1:4 and 1:3. The exterior slope of the embankment facing Seneca Lake will be graded at 1:4 to enhance its stability even though a steeper side slope would have a more than adequate factor of safety against a slope failure. Section 3.8 of this report (Volume 1) presents the stability analyses of the embankment's side slopes.

Numerous cobbles and boulders are expected to be present within the excavated site soils. Rather than be wasted or hauled off-site for disposal, these materials will be placed along the exterior slope of the embankment facing Seneca Lake to further enhance its stability. Placement and compaction criteria for the site soils placed to construct the impoundment's embankment are provided in Section 4.0 and Appendix K of this report (Volume 1).

A perimeter access road will be constructed around the top of the entire brine pond. The road will be surfaced with GRASSPAVE2, a product of Invisible Structures, Inc. This product consists of lightweight injection-molded plastic units that are bedded on a subbase of granular material and filled with sand. Topsoil and seed will be placed over the sand-filled units to develop a grass cover over a durable underlayment that will be capable of withstanding vehicular traffic and will inhibit erosion and the development of wheel ruts. It will be approximately 12 feet wide and be accessed via a same surfaced drive that connects to the existing gravel drive present directly east of the pond's embankment.

3.3 Hazard Classification

Although the embankment to be constructed for the brine pond will not be regulated as a "dam" according to the Dam Safety Section of DEC, two elements of its design compatible with DEC's Dam Safety criteria have been applied. These design elements are the freeboard and factor of safety against failure of the embankment's side slopes. For these two design elements, the "Hazard Classification" of the structure must be assessed. The hazard classification involves examining the area downhill of the impoundment and evaluating the potential losses that would occur in the event its embankment were to fail. In the case of the brine pond, there are no residential areas or major roadways between the pond and the lake that would be affected by the embankment's failure but there is a minor railroad that could be damaged in the event of such a failure. Consequently, if this impoundment structure were regulated as a dam, the appropriate hazard classification would be Class "B". Due to the height of the embankment being greater than 40 feet, it would then be further classified as a Large Class "B" dam.

3.4 Freeboard

The pond's design freeboard is 36 inches, that is the distance between the top elevation of its embankment of 840.0 feet less the operating high brine level of 837.0 feet. Following DEC's Guidelines for the Design of Dams dated January 1989, the minimum required freeboard for a new Class "B" dam is 2.0 feet. Accordingly the impoundment has been designed with a freeboard one (1) foot greater than that required for a large dam with a hazard classification of "B".

The design flood for a new Class "B" dam is 40 percent of the Probable Maximum Flood (PMF). For the region of New York State where the project is located, the PMF results from a probable maximum precipitation (PMP) event of approximately 25 inches of rainfall as identified in Figure 18 of the National Oceanic and Atmospheric Administration's (NOAA) "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian." Forty (40) percent of 25 inches equates to 10 inches of rainfall. This amount of rainfall when added to the operating level of the brine pond will leave 26 inches of freeboard.

Under an extreme wind event, wave action may develop in the brine pond and result in elevated water/brine levels that also infringe upon the pond's design freeboard of 36 inches. From NOAA website, wind speeds recorded at monitoring stations in Syracuse and Binghamton have monthly peak wind gust velocities ranging from 40 to 55 miles per hour. Using this data, a wind gust velocity of 50 miles per hour was considered appropriate for the brine pond and, although wind gusts are not sustained in nature, an equivalent sustained wind velocity of 50 miles per hour was assumed to assess the potential maximum wave heights that could develop on the pond. Wind acting in the long direction of the pond was assumed as the worst case scenario for these calculations. A total rise in the water surface elevation (wind setup plus wave run-up) of 20 inches (1.66 feet) was calculated, indicating that maximum wave height that could potentially develop on the pond should be well within the design freeboard of 36 inches. Furthermore, under the unlikely combination of extreme rainfall and sustained wind of high velocity, the maximum level of water/brine should not result in overtopping of the pond's embankment.

Again, although the embankment does not fall under the classification of a "dam" according to the Dam Safety Section of DEC, the freeboard provided in the impoundment's design is compatible with the dam safety design criteria for a Large Class "B" hazard dam.

3.5 Diversion Channels

The pond will interrupt flow of three existing drainage swales and runoff from land between the adjacent roads and the impoundment. A diversion channel will be constructed to intercept and divert the flow from these sources around the impoundment into detention basins. From these basins, the flow will discharge into 42 and 54 inch diameter HDPE culvert pipes and then into the existing swales and culvert pipes downstream of the pond.

The three existing swales convey stormwater runoff from culverts that pass under State Route 14 and the entrance/exit ramps of State Route 14A, and direct it towards Seneca Lake. The watershed for these culverts was determined using USGS quadrangle sheets and design plan information obtained from the NYSDOT. Using HydroCAD to model the watershed and estimate the flow from these culverts during a 100-year storm event, the diversion channel was sized to convey the calculated runoff. Flow velocities were computed and the drainage swales leading from the culverts (from the property line to the diversion swale) and the diversion channel itself were lined with rock and/or turf reinforced mats to protect them from erosion. Calculations supporting the design of the swales and diversion channel are presented in Appendix A. Since the culverts under the state roadways were likely designed to carry the runoff from a lesser storm event and there is some additional watershed downhill from these highways, these culverts have less capacity than the proposed diversion swale and the piping which will receive flow from the same. Accordingly, the proposed diversion swales and piping are conservatively designed.

The design capacity of the diversion channel and the downstream piping was commensurate with the size of the tributary watershed. In addition, the existence of the brine pond will capture rainfall that falls on it effectively reducing the size/area of the watershed of the diversion channel. Consequently, the peak flows in the pre-existing drainage swales downhill of the brine pond will be slightly less than what they were under preconstruction conditions.

3.6 Interceptor Trenches

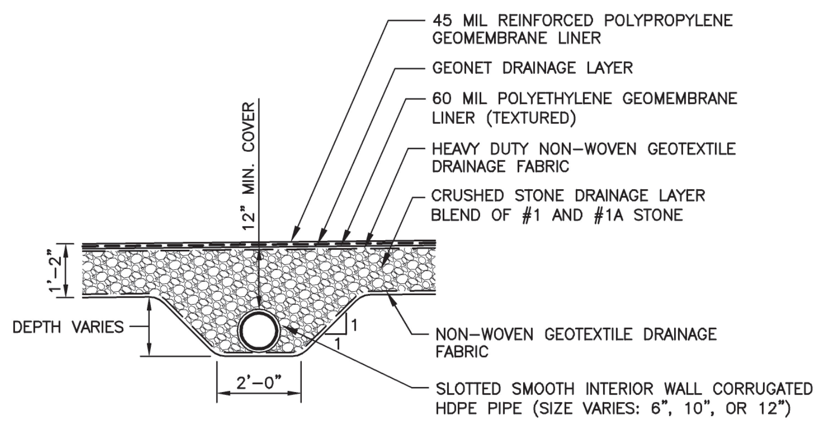
Interception and removal of groundwater is essential to not only facilitate construction of the impoundment's interior side slopes in areas of cut but also to inhibit uplift of the excavation as it progresses with depth (particularly in areas where bedrock is present at the most shallow depths below the final excavation grade).

An interceptor trench will be constructed along the eastern edge of the diversion channel to intercept seepage from water bearing seams of the overburden. With an underdrain installed at its base, the trench will be backfilled with crushed stone wrapped in geotextile. Adjacent to the high point of the diversion channel, the underdrain will have an invert elevation of 833.0 feet, i.e. it will be installed approximately 4 feet below the bottom of the diversion channel. The stone and underdrain will be pitched to the north and south from this high point of the channel and will daylight within the channel at the edge of the northern and southern ends of the impoundment's embankment at the approximate locations shown on Drawing P3 of the Project Drawings. Flow discharged from this interceptor drain will be insignificant in comparison to the flow the diversion channel will carry as the glacial till into which the interceptor drain is installed has a very low permeability. The flow it will carry will be essentially equal to that which seeps from granular seams found to be present in the test pit excavations.

Interceptor trenches/underdrains will also be installed along the toe of the impoundment's interior side slopes (east and west sides), their purpose being to help guard against excavation heave while also serving as collection pipes for the lower liner's underdrainage system discussed in Section 3.7 below. Invert elevations established for these underdrains are shown on Drawing P3. They have been carefully chosen considering the estimated depth to the weathered/broken layer of bedrock and the hydrostatic head which may be contained within the same. Underdrains along the west side of the pond near its base are to be installed at the locations shown before the excavation progresses to the base elevation of the pond. The purpose of this sequence of work is to effectively lower the hydrostatic head of water within the weathered/broken rock layer to guard against sloughing of the excavation face as it progresses further with depth. Additional underdrains intermediate to these will be installed if field conditions such as excavation base heave and/or subgrade instability dictate their need. Calculations supporting the size of the underdrains are contained in Appendix E. These calculations are conservative (i.e. they err on the safe side) as the underdrains have been sized assuming that the permeability of the weathered/broken rock is as great as 100 times the highest permeability determined through slug testing. The actual flow they will transmit and add to that which drainage swales east of the impoundment must carry is not expected to exceed 1.5 cubic feet per second (8″ diameter pipe at 1% flowing full) at any given discharge point. This additional hydraulic loading on the drainage ways leading to Seneca Lake represents an increase of less than 1 percent during the design storm event.

3.7 Geomembrane Liner System

The impoundment will be constructed using the double-liner and leak detection system schematically shown below.



Typical Geomembrane Liner System Cross-Section at Underdrain Location

Permanent drainage of groundwater below the pond is necessary to guard against hydrostatic uplift of the liner system at times when the pond level is seasonally lowered to its lowest operating level (i.e. nearly fully drained). As shown in the detail above, the liner system across the base of the pond will be underlain by a drainage course of crushed stone that being a 14-inch thick layer of an equal blend of No. 1 and No. 1A sized crushed stone. Groundwater collected by this drainage course and the interceptor drains installed at the toe of the embankment's interior slopes will be directed to five (5) header pipes, three of which will be aligned along or very close to the pre-existing drainage swales discussed in Section 3.5. At the header pipe locations, the drainage course will be locally increased an amount equal to the outside diameter of the header pipes. Estimates of the groundwater flow these pipes may carry have been made and, for their design pitch, the pipe diameters shown on the drawings are capable of transmitting many times the estimated rates of groundwater flow. Accordingly this underdrainage system will guard against hydrostatic uplift of the liner system.

In areas of cut along the western side of the pond, groundwater may also seep from the impoundment's side slope. To intercept and direct this groundwater seepage to the crushed stone drainage course placed across the base of the pond, the drainage course will be extended up the embankment's interior side slopes. Along the impoundment's west side, the drainage course will be extended up the interior side slope to elevation 825 feet. Progressing around the north and south ends of the pond, the depth of cut gradually diminishes as does the potential for groundwater seepage from water bearing seams in the indigenous glacial till. Extension of the drainage course up these interior side slopes will gradually diminish to the limit shown on Detail 4 of Drawing D2. On these slopes the depth of the drainage course will be reduced to eight (8) inches except where local unstable subgrade conditions are encountered. Where such conditions are encountered, the unstable subgrade soils will be locally undercut and the drainage course increased in depth an amount equal to the depth of undercut.

Groundwater collected by the underdrainage system described above will be discharged to drainage manholes or open swales to allow for its periodic sampling and testing. Beyond these structures, the collected groundwater will flow into drainage swales present downhill from the impoundment. Using Darcy's Law and the results of the slug testing conducted at the groundwater monitoring well locations, the quantity of seepage into the excavation was estimated to be on the order of 50 to 100 gallons per minute as presented in Appendix E. As such, the diameter of the underdrains will vary in size from 6 to 12 inches to capture and discharge this expected seepage.

A 60-mil textured HDPE geomembrane will be installed above the drainage course but will be underlain by a heavy duty geotextile to separate the liner from the underlying drainage course of crushed stone and prevent it from being punctured by the same. A leak detection system consisting of geocomposite (geonet) will be installed above this geomembrane and itself be covered with the primary geomembrane, a 45-mil reinforced polypropylene (rPP) geomembrane. The upper (primary) geomembrane will contain the brine solution while the lower (secondary) liner will contain any potential leaks in the primary liner. The upper rPP geomembrane has been selected due to its flexibility in handling, excellent weathering characteristics and chemical/UV resistance and manufacturer's warranty period of 20 years. The lower textured HDPE geomembrane has been selected due to its enhanced sliding resistance over the heavy duty geotextile and its greater resistance to being punctured by the underlying crushed stone drainage course.

At four (4) locations across the base of the pond, the lower geomembrane will be locally depressed along where leakage collection pipes will be installed below the upper rPP geomembrane. The collection pipes along these lines will be 6-inch diameter perforated/slotted HDPE pipes with double-walled solid (non-perforated) outfalls booted through the secondary geomembrane liner. Each pipe will be pitched to discharge by gravity flow to their respective collection/sampling points, allowing for identification of which section of the impoundment any potential leak occurs in. The perforated/slotted sections of these pipes will be bedded within pea stone. Profiles of the leakage detection piping from the pond to the 5,000 gallon collection tank are shown on Drawing PR1.

At the collection tank, flows from each leak detection pipe will be manually measured on a daily basis using a calibrated bucket and stop watch. Daily flow readings will also be recorded with respect to the elevation of the brine in the pond as a means of proactively monitoring the upper liner system's containment performance. The end of each pipe will be capped with a plate, the bottom of which will have a ¾ inch diameter orifice. In the event the measured flows from the leak detection pipes equipped with this orifice plate exceed 0.55 gallons per minute (gpm), the brine pond operator will record the brine pond elevation and visually inspect the pond's upper liner at or near its wetted perimeter for any obvious defects that could have contributed to the increased flow rates exceeding the 0.55 gpm operational threshold. The 0.55 gpm operational threshold is conservatively set and establishes a proactive maximum flow rate for the upper liner's leak detection zone monitoring that would signal to the brine pond operator that a defect location and repair service would be necessary for the next cycle of the brine pond draw down, if the inspection did show an obvious defect that could be repaired at this time.

The maximum hydraulic design flow rate of the upper liner system's leak detection zone geocomposite drainage layer is such that the head of brine inducing a flow rate in the detection zone being greater than 25 gpm will begin to increase the head on the lower liner such that it will be greater than the difference in the invert elevation of the leak detection pipes where they are booted through the secondary liner and the invert elevation of the pipes at the collection tank. This maximum design flow rate of 25 gpm has been set as the maximum allowable rate such that the geocomposite (geonet) drainage layer between the liners will not become fully saturated and begin to place any significant head of brine on the secondary liner if the 25 gpm threshold is not exceeded. Calculations supporting the selection of the maximum allowable design flow rate from the geocomposite leak detection system's flow rate of 25 gallons per minute for each pipe are included in Appendix K. These calculations include verification that this rate of flow will not exceed the tranmissivity of the geocomposite and, as such, demonstrates that the action limit for the daily monitoring of the leak detection system of 0.55 gpm is generally 45 times less than the maximum design capacity of the leak detection system and will ensure against liner system leakage induced groundwater impacts.

A valve will be installed on each leak detection pipe that will remain open and free flowing at all times. Two (2) submersible pumps in the collection tank will each have a discharge capacity of 200 to 250 gallons per minute. The second pump will be installed as a backup to the first. A high level alarm will be installed to provide the operator notification that the first pump has either failed or its pumping capacity has been exceeded and the second pump has been activated; thereby, providing sufficient time to evaluate the cause of the high level alarm. In the event that the first pump has failed, it will be repaired or replaced to re-establish the back up pumping system. Thus, via this proactive upper liner performance monitoring approach, in the event that one or more collection pipes are discharging excessive flows greater than the operational threshold of 0.55 gpm and, even in the unlikely event of maximum design threshold of 25 gpm being achieved, the valves on the leak detection pipes can remain open.

For operational exceedances of 0.55 gpm, Finger Lakes will plan for when removal of brine from the brine pond is next expected so that defects can be located and repairs can be made to eliminate the source of leakage from the primary liner. Finger Lakes will report all incidents where liner repairs will be necessary to the Department 30 days prior to the repairs being made.

In the very unlikely event that the maximum design threshold of 25 gpm is exceeded, Finger Lakes will notify the Department within 7 days of the exceedance and submit a corrective action plan to the Department for approval.

3.8 Embankment Stability

Again, although the impoundment's embankment does not fall under the classification of a "dam", the stability of the embankment's side slopes was analyzed following procedures identified in the DEC publication "Guidelines for Design of Dams". These guidelines reference the use of methods of analyses outlined in the U.S. Army Corps of Engineers (Corps) publication EM 1110-2-1902, *Slope Stability*. Of the seven (7) load cases listed in this publication, only four (4) were deemed applicable to this project. A description of each of these loading conditions, the slopes analyzed, and the minimum factors of safety recommended by the Corps for each load case are listed in Table 1 on the following page.

Table 1
Required Factors of Safety

Case No.	Loading Condition	Pool Elev.	Slope Requiring Analysis	Required Factor of Safety
I	End-of-Construction	837.0′	Inside & Outside	1.3
II	Long-Term with Steady Seepage and Maximum Storage Pool (including Design Rainfall Event)	837.8′	Outside	1.5
IV	Rapid Drawdown	837.0′	Inside	1.3
VII	Earthquake – Case II with Seismic Loading	As Noted Above	Inside & Outside	1.0

The stability of the embankments was analyzed using the computer program GEOSLOPE, Version 5.0. As noted in the aforementioned Corps publication, soil properties for the embankment at "End-of-Construction" are based upon a total stress analysis using undrained strength parameters. For the remaining load cases, long-term ("drained") strength parameters were utilized in the analysis.

As discussed in Volume 2 of this report, entitled *"Geotechnical Evaluation, Finger Lakes Storage Brine Pond"*, soil properties for each of the soil layers encountered were conservatively estimated based upon properties presented in published literature and our past experience with soils of these types. Long-term strength parameters for the in-situ glacial till and glacial till utilized as fill for embankment construction were estimated from the references provided in the Volume 2 report. As the glacial till is generally of a granular nature, it has been assumed that the drained and undrained parameters are equal. Due to the plasticity of samples recovered from the silt deposit, this deposit was modeled as a cohesive soil. Undrained and drained strength parameters for the silt were estimated based upon both laboratory and field testing of similar soils.

The peak ground acceleration utilized in Load Case VII (Earthquake Loading) was obtained from the USGS Interactive Hazard Deaggregation website for a seismic event with a 2-percent chance of exceedence in 50 years (a return period of 2,475 years). This return period corresponds to the 10-percent chance of exceedence in 250 years noted in Section 3.3(h)(i) of the August 20, 2010 letter from DEC regarding SEQR Review for the project. As also noted in the referenced section, "a seismic impact zone refers to an area with a 10 percent or greater probability that the maximum horizontal acceleration in lithified earth material, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10g in 250 years as delineated on the most current version of the United States Geological Survey Map". For the project site, this peak ground acceleration is equal to 0.071g, which does not exceed 0.10g. As such, according to the referenced section, a seismic analysis would not be required for this site. Irrespective of this, a seismic analysis of the impoundment's embankment stability was still conducted utilizing a peak ground acceleration of 0.071g.

As a means of determining the sensitivity of the analysis to peak ground acceleration, one (1) additional analysis was conducted for the section with the lowest factor of safety under Load Case VII. In this analysis, the magnitude of the peak ground acceleration was **doubled** over that provided by the USGS (0.071g to 0.142g). All other parameters were unchanged from the original analysis. Even under this greatly increased peak ground acceleration, the calculated factor of safety was determined to be in excess of 1.3, well above the minimum required safety factor of 1.0.

The stability analyses were conducted along each of three (3) sections: one (1) section each along the northern and southern ends of the impoundment in an east-west direction; and one (1) section in a north-south direction through the northern and southern end of the impoundment. Using the strength parameters discussed above and the design elements presented previously, the range of minimum calculated factors of safety for the various sections analyzed under each Loading Condition is shown in Table 2 on the following page. Summary tables showing the minimum factors of safety for each load case at each section analyzed are provided in Appendix I of the Volume 2 report. Computer generated output from GEOSLOPE is also contained in Appendix I of the Volume 2 report.

Table 2

Computed Factors of Safety for Embankment Side Slope Stability

Case No.	Loading Condition	Slope Analyzed	Range of Calculated Factors of Safety	Min. Req. Factor of Safety
I	End-of-Construction	Inside & Outside	2.25 to 4.2	1.3
II	Long-Term with Steady Seepage and Maximum Storage Pool (including Design Rainfall Event)	Outside	2.25 to 2.47	1.5
IV	Rapid Drawdown	Inside	1.84 to 2.35	1.3
VII	Earthquake – Case II with Seismic Loading	Inside & Outside	1.74 to 3.92	1.0

Based upon the analyses presented above, the computed factors of safety for the impoundment's side slopes/embankment under each of the loading conditions are well in excess of the minimum required factors of safety.

Due to the difficulties presented in recovering "undisturbed" samples of the glacial till, no actual laboratory strength testing has been performed. Rather, as previously stated, the strength parameters of the glacial till, both in-situ (undisturbed) and remolded (placed as embankment fill), were estimated from our experience and using published data. However, to determine the effect of the soil strength parameters upon the calculated factors of safety, sensitivity analyses were also conducted with the aid of the computer program GEOSLOPE. These analyses were conducted for the "worst-case" loading condition, that being the "Rapid Drawdown" condition. In these analyses, the assumed soil strength parameters were systematically lowered and the factor of safety against a slope failure computed. Soil strength parameters, comprised of a frictional component only, were used in the sensitivity analyses. The frictional component was reduced for both the undisturbed and remolded glacial till until the lowest factor of safety allowable for this loading condition (1.3) was calculated. In order to reach this factor of safety, the sensitivity analyses indicated that the internal angle of friction (with no assumed cohesion) for both the undisturbed and remolded glacial till must be no less than 22 degrees. In our experience, and from the referenced publications, such a low strength parameter has not been observed in the results of laboratory shear strength testing of the same. Accordingly, on the basis of these sensitivity analyses, no testing of the glacial till, in-situ or remolded, is considered necessary.

3.9 Liner Stability Analysis

Typical cross-sections of the proposed liner system along each of the embankment sections are shown on Drawing D2 of the Project Drawings. A total of three (3) interfaces were analyzed to determine their stability against sliding under the proposed design. These analyses evaluated the liner system stability for the worst case; i.e. for the longest and steepest interior slope of 1V:3H. The stability analyses were conducted utilizing methods outlined in "Analysis and Design of Veneer Cover Soils", prepared by R.M. Koerner and T.-Y. Soong and published in 2005 (a copy of which is included in Appendix F). Although this paper was prepared specifically for landfill design, the equations and conclusions presented in the paper are applicable for use in any cover system where a material overlies geosynthetics.

In the analyses, product data sheets were utilized in estimating material properties for each of the liner components. These sheets are included in Appendix G. Interface friction values for each of the analyzed interfaces were estimated based upon published data. Interface adhesion, although listed in the published data, was assumed to be zero, thus making the stability analyses conservative (i.e. making the calculated factor of safety less than they would be otherwise). The stability computations for each of the liner interfaces are presented in Appendix H and the results of the analyses shown below in Table 3.

Table 3
Computed Factors of Safety for Liner System Stability

Interface Description	Calculated Factor of Safety
Geonet over Lower Geomembrane	2.36
Lower Geomembrane over Geotextile	1.53

Geotextile over Subgrade	1.50

On the basis of the assumptions described above, the safety factors listed in Table 3 indicate that the critical interface (i.e. that which had the lowest factor of safety against sliding) is between the geotextile and the subgrade soil.

Minimum acceptable factors of safety for stability analyses of landfill cover systems are provided in the aforementioned paper by Koerner and Soong. This information was used as a basis for assessing the stability of the liner system for the brine pond. According to Tables 4 & 5 of the Koerner & Soong paper, the factor of safety is directly related to the expected duration of the facility, the concern level, and the type of waste stored at the location. It is expected that the brine pond will be in operation for several decades, and accordingly will be considered as "permanent." Due to the presence of Seneca Lake immediately downstream of the brine pond and concerns over potential environmental impacts, the brine pond has been assessed a "critical" concern level, with a corresponding ranking of "High."

As the chemical composition of the brine solution does not meet any of the requirements outlined in Title 6 NYCRR Part 360, we have assigned the site a "non-hazardous waste" designation with a corresponding minimum factor of safety for liner stability of 1.5. Accordingly, under the conservation assumptions described above (e.g., an assumed interface adhesion of zero), all of the interfaces of the geomembrane liner system exceed this minimum required factor of safety. Technical specifications for installation of the liner system are presented in Appendix L and include the need to perform direct shear testing to assess and verify that the minimum required interface shear strength parameters are, in fact, greater than those specified.

Also included in Appendix H are anchor trench calculations supporting the length of trench within which the geomembrane liners are embedded/anchored. Dr. Koerner of the Geosynthetics Research Institute was consulted regarding the design methodology and concerns regarding overstressing of the various components of the liner system. Details of the anchor trench indicate that the geocomposite between the geomembranes terminates a few inches into the anchor trench, this being necessary to avoid its overstressing. The anchor trench details also indicate that the soil mixed with bentonite will be placed in the anchor trench as backfill where the ends of the geomembranes terminate and that drainage of the granular backfill placed above the geomembranes will be accomplished through the installation of geotextile wrapped perforated HDPE pipe. This pipe will be spaced 50 feet on-center around the perimeter of the pond and will be pitched for gravity drainage to the outside of the pond's embankment.

3.10 Liner Uplift (Wind Induced)

As the upper geomembrane is exposed and not covered by any soil, or brine solution at certain times of the year, uplift pressures due to wind were analyzed and considered in the design of the geomembrane and anchor trenches. Anticipated uplift pressures were determined according to the methodology outlined in "Uplift of Geomembranes by Wind" by J.P. Giroud, T. Pelte, and R.J. Bathurst, published in Geosynthetics Institute, Vol. 2, Number 6 in 1995. A copy of this article has been included in Appendix F.

The uplift pressure on the upper geomembrane was calculated for a peak gust wind speed obtained from climatic wind data available from NOAA's website. Data presented on this website was accumulated between 1930 and 1996 and is not inclusive of the entire 66-year period. Monitoring stations were located in Syracuse and Binghamton, and monthly peak gust wind velocities were generally found to range from 40 to 55 miles per hour at these stations. Using this data, a design wind velocity of 50 miles per hour was selected for the uplift calculations for this project. Material properties such as weight per unit area and allowable tensile capacity were estimated from the Product Data Sheets presented in Appendix G.

From this analysis, it was determined that a 45-mil rPP geomembrane did not possess the required weight to resist wind pressures. Accordingly, the design includes the installation of weight tubes along the surface of the geomembrane across portions of the pond bottom where it is anticipated that pressures will be less than 3 pounds per square foot. These tubes are to be sand filled and bonded to the liner at intervals of 5 feet. Calculations supporting their need and use are presented in Appendix I.

3.11 Stability of State Route 14A Embankment

As construction of the impoundment involves some grade modifications along the western property line bordering the entrance ramp of State Route 14A, the stability of its embankment was analyzed along the section where construction of the diversion channel included excavation of ground below (directly east of) the roadway embankment. The section analyzed was located where construction of the diversion channel will remove the greatest depth of soil, this being a few feet south of the most southern culvert which passes beneath the roadway's embankment. Soil conditions similar to those encountered by the test boring advanced closest to the section (SB-5) were assumed as the embankment's foundation conditions while the embankment was assumed to be constructed of soils similar in composition to that which the pond's embankment is to be constructed. These assumptions are considered to be representative of highway construction practice wherein soils removed from areas of nearby cut are generally used as embankment fill materials.

Soil strength and unit weight parameters the same as those used to analyze the pond's embankment stability were applied in the analysis using the computer program GEOSLOPE. Appendix I contains the computer output and indicates that the factor of safety against a slope failure of the roadway's embankment is 1.40. This value is in excess of the minimum value of 1.3 commonly accepted in geotechnical practice as being adequate for this type of structure.

4.0 CONSTRUCTION CONSIDERATIONS & QUALITY CONTROL

Construction of the impoundment's embankment must be performed in a controlled manner. The overburden present at the site is laden with cobbles and boulders, and contains water bearing granular seams. Particles greater than 12 inches in size will be culled from the excavated soils as they are placed as fill. In addition, as the excavated material may be "wet" upon its excavation, it will be disked as required to aerate and "dry" it to a moisture content at which it can be compacted to the degree required.

The fill will be placed in maximum loose lift thicknesses of 12 inches and be compacted to a dry density equal to at least 95 percent of the material's maximum dry density as it is defined by the Standard Proctor Compaction Test, ASTM D-698. The moisture content at which this degree of compaction is achieved is specified to be at or within 3 percent **below** the optimum moisture content defined by this standard.

Placement of fill to construct the impoundment's embankment must be performed under the full-time observation of the Geotechnical Engineer-of-Record. Field in-place density tests will be performed on a frequent basis to ascertain that the compaction specification is being met. In the performance of this testing, numerous laboratory "Proctor" compaction tests will be performed to confidently identify the optimum moisture-maximum density relationships of the fill and one-point "Proctor" compaction tests will be performed in the field to verify that the appropriate optimum moisture-maximum dry density relationship is being used to assess the percent compaction.

The subgrade soils composed of glacial till, whether natural or placed as fill to construct the impoundment embankment, will be sensitive to disturbance. The natural soils are expected to be "wet" and will tend to rut and/or pump and weave under repeated traffic of earthmoving equipment. Final preparation of the subgrade surface across the impoundment's bottom will include back-blading of the surface to infill any ruts and "light " proofrolling of its surface to provide a relatively smooth and stable surface for the placement of geotextile and the overlying drainage blanket of crushed stone. Placement of geotextile and crushed stone on the interior side slopes of the impoundment will receive like treatment. Above where these materials are placed, large particles such as cobbles will be removed so that they do not protrude from the surface of the side slopes. The final subgrade surface in these areas will receive repeated fine grading and proofrolling to maintain a stable, relatively uniformly graded surface upon which geotextile and the overlying textured HDPE liner will be placed.

5.0 POST-CONSTRUCTION OPERATIONS

5.1 General

The location of the wells and the brine pond is best illustrated by examining the map shown in Figure 2, "Site Operations Plan". This map shows the location of the proposed brine pond relative to the location of the injection and extraction wells. Additionally, this plan shows the location of the ponds and wells relative to State Routes 14 and 14A, Seneca Lake, the railroad, the Casella Site and U.S. Salt site.

5.2 Cavern Creation

The caverns already exist and they are filled with brine solution. The brine solution will be displaced from the underground caverns when the LPG is injected into them. The brine will flow up the tubing string into the brine line at the surface, which is a sealed system, and through the flare stack to remove any possible gas product that may accompany the brine solution and into the proposed brine pond. The caverns will always have fluid in them.

5.3 Introduction of Product

The product (LPG) will be injected into the caverns from the above ground tanks located at the facility on State Route 14A, or directly from the pipeline. The product will be transported to the above ground tanks from the transmission pipeline and rail siding and be injected into the caverns by the use of the LPG injection pumps located off

State Route 14 (See Figure 2). These pumps are needed to overcome the differential head pressure created by the brine column in the brine tubing string in the well. During normal operations, product will be injected into the caverns over the summer months when the demand for heating fuel greatly diminishes. During the injection season, the brine solution will be displaced from the cavern into the brine pond until it is needed to displace the LPG back to the above ground facilities. The design has included the welding of a 10-foot by 12-foot piece of reinforced polypropylene geomembrane at the discharge point as added protection of the upper liner against the brine solution's relatively high temperature discharge point. This temperature may be as great as 120^0 F at the discharge point with temperatures within the pond typically ranging from 50^0 to 90^0 F during spring and summer months.

5.4 Extraction of Product

During normal operations, extraction of LPG will begin to occur in the fall season and last through the winter season when the demand for heating fuel is the greatest. The product will be withdrawn from the cavern by injecting the brine solution from the brine pond with the use of the 3 small brine pumps located on the east side of the pond. These are low pressure high volume pumps which pull the brine from the pond and inject it into the brine pipelines which are connected to the wells. The displaced LPG will either be stored in above ground tanks or be pumped into rail cars or tanker trucks for delivery. The tank, rail car and truck filling will occur on the former Casella property (now owned by Finger Lakes LPG Storage, LLC) located west of the brine pond site and off of State Route 14A.

5.5 Brine Pond Level Control

The brine pond is open to the atmosphere such that there will be a combination of evaporative losses and rainfall/snowmelt accumulation. At the project site, the average historical annual rainfall exceeds the evaporative losses, so over time there should be a net accumulation of brine solution once the pond is placed into operation. The level in the brine pond will be monitored on a daily basis and in particular during and following periods of intense rainfall. If the brine level ever exceeds elevation 837.0' and begins to encroach on the freeboard, then some of the brine solution should be pumped to the U.S. Salt brine pond located south of the main brine pond. The pumped off brine solution will be used to generate more salt product and taken permanently out of the cavern loop.

5.6 Leak Detection Monitoring

In the unlikely event of a leak, the geocomposite placed between the upper and lower geomembrane liners will allow any leaked brine solution to migrate to the leak collection pipes installed along four (4) lines across the pond bottom. At these locations, perforated collection pipes will capture any leaked solution and convey it through the lower liner to a 5,000 gallon pre-cast concrete holding tank located between the brine pump building and the flare stack located along the gravel drive near the downhill toe of the impoundment's embankment.

Flow from the four (4) collection pipes will be monitored on a daily basis as set forth in Section 3.7 of this report. The pipes at their point of entry into the holding tank will be numbered to match the number designation of the area of the liner they service. The end of each pipe will be equipped with an orifice plate to facilitate monitoring their flow rate and, as previously described, establish the rate above which the geocomposite (geonet) will begin to become saturated and result in a head of brine to be imposed on the secondary liner.

The accumulated brine solution will be pumped out of the tank's containment chamber/sump back into the brine pond using the submersible pumps installed within the tank. If the recorded leakage exceeds 25 gallons per minute at any pipe, steps will be taken to determine the vicinity of the leak(s) in the primary liner. The pond will then be drained and investigations will begin to locate leaks by visual observation and testing existing seams. Drainage of the pond will be conducted at the end of the winter heating season. If the leak locations are not visible, a specialized leak detection contractor will be hired to perform electronic detection. Upon locating the leak(s), repair(s) will be made utilizing the appropriate method for the specific type of leak to be repaired.

5.7 Geomembrane Monitoring

As it is difficult to accurately predict the service life of the primary geomembrane liner, several strips of geomembrane will be welded onto the primary liner to allow for their recovery and testing at 5-year intervals of the liner's service life. These pieces will be 18 inches by 36 inches in plan and will be welded to the primary liner across their short ends. They will be positioned at and below the seasonal high operating level of the pond (elevation 837 feet and below) and just below the injection point where the brine will be delivered at elevated temperatures. The locations of these test strips are shown on Drawing No. P3.

Parameters for which the samples will be tested will include thickness, density, tensile strength, and tear and puncture resistance. Sampling and testing standards to be followed will be those designated in the specifications for the primary geomembrane. Results will be compared to those during manufacture of the material. Acceptable values for tests run during manufacturing are listed in the specification for synthetic membranes. The allowable service life reduction in these values will be established based upon recommendations provided by the Geosynthetic Research Institute.

5.8 Groundwater Quality Monitoring

The lower geomembrane liner provides a secondary line of defense against the leakage of brine into the groundwater. In order to document the quality of the groundwater during the life of the pond, a series of monitoring wells will be installed. The location of these monitoring wells is shown on Figure 3, "Groundwater Monitoring Well Locations". Monitoring groundwater quality will consist of collecting groundwater samples from two (2) up-gradient and three (3) down-gradient monitoring wells and, in addition, one (1) well to be located at the pond's south end near the base of the embankment's exterior slope. The up-gradient and down-gradient wells will be screened within the overburden whereas the monitoring well installed at the south end of the pond will be screened within bedrock. The samples from all the wells will initially be analyzed for "Baseline Parameters" per Title 6 NYCRR Part 360 regulations. Based on the test results the owner may petition DEC to reduce the list of analytical parameters to more closely match that of the brine solution, i.e. chlorides and certain other parameters. The groundwater quality will be monitored on a quarterly basis. More frequent groundwater monitoring will be conducted in the event a leak in the primary liner system is identified. If the concentration of targeted parameters increases in the down-gradient monitoring wells (but not in the up-gradient wells), steps will be undertaken to further investigate the cause of such an occurrence.

Groundwater monitoring wells previously installed within the footprint of the brine pond will be decommissioned in accordance with the provisions set forth by CP-43: Groundwater Monitoring Well Decommissioning Policy and its addendum, Groundwater Monitoring Well Decommissioning Procedures. Appendix M contains copies of these documents. This work will be performed under the observation of the

Engineer-of-Record prior to initiating site clearing.

5.9 Closure Planning

When the facility is slated for closure, brine will be drained from the pond and the 5,000 gallon holding tank and any solid residues remaining in each removed from the same. The residues appropriately disposed of along with the holding tank, liners, geocomposite and leak detention piping. The impoundment area will be backfilled with the materials used to construct the impoundment's embankment, returning and shaping the site grades to those which previously existed. Provisions set forth in 6 NYCRR Part 360-6.6(c)(3) will be followed for the facility's closure.

6.0 SUMMARY / CONCLUSIONS

The above referenced criteria were used as the basis of design and construction of the brine pond. Provided that the impoundment is constructed in conformance with the accompanying design plans and technical specifications presented herein, the impoundment should have a design life of at least 20 years. Testing of strips of the primary liner will provide the basis for assessing the validity of this judgment and determine at what time the primary liner requires replacement.

FIGURE 1

Vicinity Map