REVIEW OF PERMIT APPLICATION
RMU-2, CWM MODEL CITY

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LIST OF EXHIBITS

1. INTRODUCTION

This report presents the results of review of a permit application for the proposed Residual Management Unit 2 (RMU-2) at the Model City Facility, located in the towns of Lewiston and Porter, New York, which is owned and operated by the CWM Chemical Services, LLC (CWM), a wholly owned subsidiary of Waste Management, Inc.

The comments presented here were prepared by Anirban De, Ph.D., P.E. on behalf of the Municipal Stakeholders, with reference to several critical components of the proposed design. These comments are related to the following elements:

a. Shear strength of the bottom liner system
b. Vertical separation between waste and historical high groundwater level
c. Nature of subgrade
d. Hydrostatic uplift pressure below subgrade
e. Hydraulic characteristics of the Glaciolacustrine silt/sand unit
f. Uncertainty regarding subsurface conditions below proposed RMU-2
g. Leachate generation rates

2. DOCUMENTS REVIEWED

The comments presented in this report are based on review of the following documents:

1. Engineering Report, Residuals Management Unit 2, Model City Facility, prepared by Arcadis for Chemical Waste Management, dated April 2003 and most recently revised on November 2013
2. Shear Strength Evaluation for Slope Stability Analyses Residuals Management Unit One (RMU-1) Model City Treatment, Storage, and Disposal Facility, dated March 2001
3. Hydrogeologic Characterization Update, Model City TSD Facility, Model City , New York, prepared by Golder Associates, dated January 2014.
4. 2009 Annual Water Budget Summary Report, prepared by CWM Chemical Services, Inc. for New York State Department of Environmental Conservation, dated November 2010.
5. 2011 Annual Water Budget Summary Report, prepared by CWM Chemical Services, Inc. for New York State Department of Environmental Conservation, dated December 2012.

The second document has been referred to as ‘ “Peer Review Panel Report, Shear Strength Evaluation for Slope Stability Analyses, RMU-1, Model City Treatment, Storage and Disposal Facility, Model City, New York", by Koerner, Gilbert, Stark, Adams, dated March 2001', and is provided in Appendix A-1 of the first document.

3. SHEAR STRENGTH OF LINER SYSTEM

According to Section 3.3.3 of the Engineering Report:

"As described above, the RMU-2 liner system is similar to that used in RMU-1, with the exception of the substitution of GCL for compacted clay in the primary liner system." (underline added)

Appendix A-1 of the Engineering Report is titled "Selection of Soil Properties for Geotechnical Evaluation of RMU-2 Design". This report, prepared by P. J. Carey & Associates, PC (PJCA), documents the engineering properties of various components of the landfill that were used in analyses in support of the permit application. Section 5 of Appendix A-1, titled "Landfill Material Shear Strengths" discusses the selection of shear strength properties used in the stability analyses pertaining to the base and side-slope liner materials.

According to the text at the beginning of Section 5 of the document:

"The shearing properties of the landfill materials … were examined in detail by a panel of experts commissioned by CWM in 2000. The panel completed its evaluation in March of 2001 and issued the report referenced in Section 1 of this Appendix. The report has been submitted to the NYDEC as a part of ongoing work at the site. PJCA has reviewed this report in detail and agrees with the recommendations pertaining to the landfill materials."

The shear strength properties of four materials of the landfill are discussed in the subsequent sections, which are: waste, baseliner (on the bottom), side-slope liner, and compacted clay liner.

According to Section 5.2, which contains discussion on the bottom liner:

"The recommended shear strength versus normal load function presented in the Peer Review Panel report was adopted for use in the analysis. The graph below represents the function used in the Geoslope analyses. The Peer Review Panel based these strengths on large displacement test values. Therefore no further reduction for

deformation potential need be applied. CWM intends to use similar materials for the bottom liner system as have been tested for RMU-1. Therefore, this strength envelope is applicable for application to RMU-2." (underline added)

A table containing the summary of shear strength properties used in stability analyses is presented in Section 6. However, the liner configuration CWM intends to use in RMU-2 differs from the one that was considered by the Panel report for RMU-1. Therefore, the recommendations of the Peer Review Panel Report for the bottom liner do not apply to the design of RMU-2.

Bottom liner considered by the Panel report

The bottom liner system considered in by the Peer Review Panel is described in Section 1.2.1 of the Panel report. The Panel conducted extensive research of published literature and undertook laboratory testing of material actually used at the site. Based on their analyses, the Panel concluded that the interface between the compacted clay and the non-woven geotextile filter was the most critical surface for stability. Accordingly, the recommendation for shear strength of the bottom liner was based on the strength of this interface.

The bottom liner section being proposed for RMU-2 is presented in Section 3.3.3 of the RMU-2 Engineering Report (as well as in Figure 1 of Sheet 15). This section is very similar to the one considered by the Panel report, except for one critical difference: *the 1.5-foot thick primary compacted soil liner considered by the Panel is replaced by a geosynthetic clay liner (GCL) in RMU-2*.

GCL is used in the bottom liner to provide enhanced protection against contaminant migration, as compared with compacted clay liner. However, it is known from recent technical literature that the internal shear strength of a hydrated GCL (i.e., when the GCL comes into contact with moisture) can be very low, especially under high normal stress. The interface shear strength between a GCL and a textured geomembrane can also be very low. Therefore, any analysis which does not consider the GCL in the liner configuration is flawed.

Since the GCL unit was not part of the cross section analyzed by the Panel, the recommendations of the Panel do not apply to RMU-2. A new report must be provided, with recommendations that include consideration of the GCL.

4. VERTICAL SEPARATION BETWEEN WASTE AND HISTORIC HIGH GROUNDWATER LEVEL

RMU-2 Landfill

According to information presented in Table 1 of Appendix C-4 of the Engineering Report, the piezometric surface represented by the historic high groundwater level within Cell 20 of RMU-2 is at an elevation of 316.6 feet above mean sea level (amsl). In the same cell, the bottom of liner/top of subgrade is at elevation 308.3 feet (amsl) (Table 3, Appendix C-4). Therefore, the historic high groundwater level will be 8.3 feet above the bottom of the liner. Since the liner is approximately 6 feet thick, the groundwater level is 2.3 feet higher than the bottom of waste/top of the operations unit.

The design proposed by the applicant will fail to meet several regulatory requirements, as follows:

- The requirement of 50 feet separation between the bottom of the landfill and highest groundwater level, as stipulated by 40 CFR 761.75 (b)(3)
- The requirement that "A minimum separation of five feet must be maintained between the base of the constructed liner system and the seasonal high groundwater elevation", as stipulated by 6 NYCRR § 360-2.13(d) [applicable to hazardous waste management facilities per 6 NYCRR § 373-1.1(b)(2)]
- The requirement that no waste be closer than 10 ft to an aquifer, as stipulated by 6 NYCRR § 373-2.14(b)(2)
- The requirement that "The owner or operator of a leak detection system that is not located completely above the seasonal high water table must demonstrate that the operation of the leak detection system will not be adversely affected by the presence of ground water", as stipulated by 6 NYCRR § 373-2.11(b)(3)(iv)

Fac Pond 5

According to information presented in Table 4 of Appendix C-4 of the Engineering Report, the piezometric surface represented by the historic high groundwater level within Fac Pond 5 is at an elevation of 313 feet above mean sea level (amsl). In the same pond, the design final grade is at elevation 307 feet (amsl) (Table 5, Appendix C-4). Therefore, the historic high groundwater level will be 6 feet above the final grade.

The design proposed by the applicant will fail to meet several regulatory requirements, as follows:

- The requirement that "the bottom of the impoundment liner must be a minimum of five feet above both seasonal high groundwater", as stipulated by 6 NYCRR § 360-4.2(b)(3)(viii) [applicable to hazardous waste management facilities per 6 NYCRR § 373-1.1(b)(2)]

- The requirement that a surface impoundment be "constructed a minimum of five feet above the seasonally high groundwater table", as stipulated by 6 NYCRR § 360-6.5(a) [applicable, as stated in the above bullet item]

- The requirement that "A minimum separation of five feet must be maintained between the base of the constructed liner system and the seasonal high groundwater elevation", as stipulated by 6 NYCRR § 360-2.13(d) [applicable, as stated in the first bullet item]

- The requirement that "The owner or operator of a leak detection system that is not located completely above the seasonal high water table must demonstrate that the operation of the leak detection system will not be adversely affected by the presence of ground water", as stipulated by 6 NYCRR § 373-2.11(b)(3)(iv)

5. NATURE OF SUBGRADE

The quality of subgrade which is present between the bottom of the landfill and the groundwater level at this site does not meet regulatory requirements. The Glaciolacustrine Clay (GC) unit is variable in thickness under the footprint of the proposed RMU-2 landfill.

Information on the thickness of the GC unit is provided in:

- A drawing titled "Thickness Contours of Glaciolacustrine Clay Unit", included as Attachment B, in Appendix C-4 of the Engineering Report (Exhibit A)
- A drawing titled "Thickness of Glaciolacustrine Clay", included as Figure 9 of the 2013 Hydrogeologic Characterization Update (Exhibit B)
- Section 2.2.1 Site Geology of the Engineering Report

Despite having almost identical titles, Exhibits A and B present contours of GC unit thickness that are not the same. However, contours shown in both figures indicate that the thickness of the GC unit is less than 5 feet in some areas below the proposed RMU-2 footprint.

According to "Section 2.2.1 Site Geology" of the applicant's engineering report (revised November 2013):

"Within the RMU-2 footprint, the thickness of Glaciolacustrine Clay varies from less than 1 foot to 25 feet."

Please see additional discussions of the hydrogeological setting by Dr. Andrew Michalski, submitted separately on behalf of the Municipal Stakeholders.

The design proposed by the applicant will fail to meet the following regulatory requirements:

- The requirement that the landfill site be *"located in thick, relatively impermeable formations such as large-area clay pans…"*, as stipulated by to 40 CFR 761.75 (b)(1).

- The requirement that the soil beneath the facility have a hydraulic conductivity of 10^{-5} cm/s or less, stipulated by 6 NYCRR § 373-2.14(b)(1)

- The requirement that the bottom of the landfill be at least 50 ft above the historical high groundwater level, as stipulated by 40 CFR 761.75 (b)(3)

6. HYDROSTATIC UPLIFT

Background

A relatively high piezometric level is associated with the Glaciolacustrine Silt/Sand (GSS) unit at the site. As a result, excavation plans must take into account hydrostatic uplift pressure at the bottom of floor areas and sumps of proposed cells. The design must provide an acceptable factor of safety against uplift due to hydrostatic pressure, since the occurrence of an uplift will have very serious consequences.

The design proposed by the applicant fails to meet applicable standards and is not acceptable for several reasons, as follows.

- The thickness of GC unit and the elevation of top of GC unit, used in hydrostatic uplift analyses (Appendix C-4 of Engineering Report, 2013) are not supported by more recent information presented in Hydrogeologic Characterization Update (2014)
- The applicant has not provided data to establish how the “historical high” piezometric levels used in the Engineering Report were selected.
- The design piezometric levels used in uplift analyses (Appendix C-4) do not agree with the one used in stability analyses (Appendix C-5).
- The applicant has used a factor of safety of 1.0 and 1.2 against hydrostatic uplift for the sump area and cell floor area, respectively. Given the general lack of field data on subsurface characteristics below RMU-2, the high level of uncertainty associated with the design, and the contaminated nature of the aquifer, the applicant’s use of low factors of safety is not acceptable.

The above points are explained in the following paragraphs.

Thickness of GC Unit

GC unit thickness of between 11 ft and 21 ft in various cells of RMU-2 is used in the applicant’s hydrostatic uplift analysis (Table 1 of Appendix C-4 of the Engineering Report). A 2002 figure of thickness contours by Golder Associates (Exhibit A) is cited as the source of this information. However, thickness contours shown on a more recent drawing by Golder Associates (Hydrogeologic Characterization Update, 2014), included here as Exhibit B, are quite different from those shown in Exhibit A. According to Exhibit B, there are significant areas within RMU-2 where the thickness of the GC unit is 5 ft or less (e.g., near sumps of Cells 15 and 16 on the north part and near sump of Cell 18 on the west part).

Top of GC Unit

As part of the hydrostatic uplift analysis (Table 1 of Appendix C-4 of the Engineering Report), the applicant has shown the top of the GC unit to be at elevations of 293 ft and 295 ft amsl in Cells 15 and 16, respectively. A 2002 figure of top of GC elevation contours by Golder Associates (Exhibit C) is cited as the source of this information. However, elevation contours shown on a more recent (2014) drawing by Golder Associates (Exhibit D) are quite different from those shown in Exhibit C. According to Exhibit D, the elevation of the top of GC unit is between 303 ft and 310 ft amsl, in the north parts of Cells 15 and 16 (where the sumps are located), with most areas having an elevation of about 307 ft amsl.

Excavation into GSS Unit, Hydrostatic Uplift and Quick Condition

According to most recent drawings by Golder Associates, as part of Hydrogeologic Characterization Update (2014), the top of the GC unit is at an elevation of about 307 ft and the unit is about 5 ft thick in the area where the sumps of Cells 15 and 16 are to be located. According to Table 2 in Appendix C-4 of the Engineering Report, the lowest elevations of sumps in Cells 15 and 16 are to be 298.5 ft and 296.6 ft amsl, respectively.

To excavate the sump areas of Cells 15 and 16, it will be necessary to lower the elevations from 307 ft (top of GC unit) to 298.5 ft or 296.6 ft amsl (lowest sump elevation), respectively. This will require excavations of 8.5 ft or 10.4 ft (i.e., the difference between existing top of GC and proposed bottom of sump) of material, respectively. However, since the GC unit is only about 5 ft thick in these areas (Exhibit B), it is clear that, the bottom of the sumps will not be located within the GC unit. Rather, the excavation for the sumps will fully penetrate the GC unit and enter the underlying GSS aquifer unit.

The assumption regarding top of GSS unit, made in the hydrostatic uplift calculations (Attachment C of Appendix C-4 of the Engineering Report), is not consistent with the recent information on GC unit thickness and top elevation of GC unit (Exhibits B and D). Therefore, the soil pressure, which the applicant assumed would prevent uplift, is non-existent and the uplift analysis is invalid.

This will have several serious consequences. In areas where the clay will not be fully penetrated, an uplift/heaving of the clay subgrade will occur, since the relatively thin layer of remaining clay will not be sufficient to counteract the effect of hydrostatic pressure from the bottom. In areas where the clay will be fully penetrated by excavation, the GSS layer will be exposed. Groundwater pushing from the bottom will cause the cohesionless sand/silt to flow upwards into the excavation. This is known as a "quick condition" and it will be propelled by high hydraulic gradient and sustained by the presence of alluvial channels of high hydraulic conductivity in areas below the RMU-2 footprint.

It is also important to point out that the water in the underlying GSS unit, which will flow into the excavation in the mechanism described above, may be contaminated (see report by Dr. Andrew Michalski).

Unacceptable Factors of Safety

According to Section 3.3.3.1 of the Engineering Report:

"The subgrade grading plan (i.e., the bottom of the liner system) … has been designed based on the predicted hydrostatic uplift force on the bottom of the sumps and the cell floors resulting from the historical high groundwater elevations measured in May 2001."

The proposed bottom of the sump subgrade in each cell is designed so as to provide a minimum factor of safety of one (1.0) against uplift due to hydrostatic pressure. This means that, when the sump is excavated, the downward pressure of the soil will be exactly equal to the upward pressure of groundwater. The Engineering Report considers the design based on a factor of safety of one (1.0) acceptable because the highest groundwater table from 2001 was considered and the plan area of the sump floor is relatively small (15.5 feet by 21.5 feet). According to the Engineering Report, the factor of safety will be verified prior to construction by means of test pits and/or piezometric wells. Also, the applicant has proposed a factor of safety of 1.2 for the cell floor areas adjacent to sumps.

A factor of safety equal to one (1.0) indicates an equilibrium condition and 1.2 indicates a marginal equilibrium. Any deviation from the assumed design conditions may trigger a failure. As discussed earlier, there are significant inconsistencies between information presented by the applicant in two different reports (Engineering Report, 2013 and Hydrogeologic Characterization Update, 2014) and conditions presented in the more recent report indicate a more adverse situation with respect to uplift. Thus, the use of a factor of safety of 1.0 in the sumps or 1.2 in the floor areas cannot be accepted.

In case of Fac Pond 5, the "worst case piezometric head" is 313 ft amsl (Table 4 of Appendix C-4 of the Engineering Report). However, based on the design elevations, the "controlling piezometric head" for areas outside the sump is 312.4 ft amsl (Table 7). Thus, the design fails to meet the factor of safety of 1.2, required for areas adjacent to the sump.

Dr. Andrew Michalski's expert report raises questions about the applicant's characterization of the hydraulic conductivity and integrity of the subsurface units. In light of these concerns, it is not prudent to follow the design approach (with factor of safety of one) proposed by the applicant.

The applicant's approach of developing a design based on conditions which are unsupported by recent facts (e.g., the 2014 Hydrogeologic Characterization report) and a plan to verify factors of safety in the field during construction fails to provide a reasonable assurance of compliance.

Inconsistent Piezometric Levels in Different Analyses

The "high groundwater elevations" considered in the uplift pressure is inconsistent with the high groundwater level that was utilized in slope stability analyses. According to Uplift Calculations (Table 1 within Appendix C-4) the controlling piezometric head is 316.6 ft in Cell 20. In the slope stability analysis through Cell 20, represented by cross section 13+50 (Filename: RMU2X1350.gsz), the piezometric level is between 313 ft (on the east end) and 314.5 ft (on the west end). These levels are lower than both the 2001 head (316.6 ft) and the 2004 head (315.6 ft), as reported in Attachment C of Appendix C-4.

The piezometric head considered in the slope stability analyses is discussed in Section 2.3.1.1 of Appendix C-5 (RMU-2 Stability Analysis Final Buildout). According to this section:

"Effective stress in the analysis was computed including the long term piezometric surface reported by Golder in 2004. The piezometric heads were applied to all materials except the baseliner."

The use of October 2004 piezometric level for slope stability analyses is not consistent with the assertion in Section 3.3.3.1 that the May 2001 groundwater elevation is the "historical high" value.

A high groundwater elevation or a high piezometric level generates upward pressure below the liner system. This creates forces which can lead to instability of the slopes. Therefore, it is normal practice to consider the highest anticipated groundwater elevation in long-term slope stability analyses, to account for the most adverse condition.

The same value of highest groundwater level should be used in both uplift analysis and slope stability analysis.

Basis for Historical High Groundwater Level

The "historical high groundwater" elevation used in design is the higher piezometric level between those measured in May 2001 and in October 2004. However, it is not clear over what time period such "historical" data was applicable. The applicant must provide water level elevations from different wells at the site to establish the historical high elevations before using them in the analyses.

The applicant has not indicated whether the most adverse high groundwater level considered in design takes into account possible impacts of climate change. Precipitation above historical levels may cause groundwater levels to be elevated beyond what the applicant has considered in design.

As presented by the applicant, the hydrostatic uplift analysis fails to meet the requirement of the applicable regulation as follows:

"The foundation beneath the landfill be capable of supporting the baseliner and pressure gradients so as to prevent liner failure due to compression or uplift", as stipulated by 6 NYCRR § 373-2.14(c)(1)(i)(b)

7. HYDRAULIC CHARACTERISTICS OF THE GLACIOLACUSTRINE SILT/SAND (GSS) UNIT

Hydraulic Conductivity of GSS Unit

The applicant has presented contradictory information regarding the hydraulic conductivity of one of the major units underlying the site. According to the expert report by Dr. Andrew Michalski, the applicant has failed to identify the existence of an alluvial sand and gravel unit of fluvial origin and has instead erroneously combined it with the glaciolacustrine silt/sand (GSS) unit. Further, (as explained in Dr. Michalski's report), the applicant has not recognized the existence of a buried alluvial valley bounded by ridges, which causes the groundwater flow directions of the GSS unit to vary sharply. The applicant has erroneously characterized the GSS unit with uniform conductivity based on widely varying hydraulic conductivity values.

According to Section 2.2.2.1 of the Engineering Report, the hydraulic conductivities of the GSS unit are 3×10^{-5} cm/s in the horizontal direction (k_H) and 1.6×10^{-5} cm/s in the vertical direction (k_V).

Section 3 of Appendix A-1 is titled "Permeability and Deformational Properties of Non-GC Strata" [GC = Glaciolacustrine Clay]. According to Section 3.2. of this Appendix, the hydraulic conductivities of the GSS unit are 2×10^{-4} cm/s in the horizontal direction (k_H) and 2×10^{-5} cm/s in the vertical direction (k_V).

The difference between the two reported values of k_H is relatively large (almost one order of magnitude between 3×10^{-5} cm/s and 2×10^{-4} cm/s). According to Dr. Michalski's expert report, the hydraulic conductivity in the buried central valley is two orders of magnitude larger than that on the northern side of the valley.

According to information reviewed here, the hydraulic conductivity of the GSS unit, which is a major component of the soil beneath the facility, does not meet the requirement of hydraulic conductivity stipulated in Part 373-2.14 (b)(1):
"The <u>soil beneath the facility shall have a hydraulic conductivity of 10^{-5} centimeters per second or less</u> as determined by in situ hydraulic conductivity test methods and shall be subject to the approval of the department." (underline added)

Furthermore, the applicant has presented contradictory information in different parts of the same Engineering Report regarding hydraulic conductivities of this material, by citing a value of $k_H = 3\times10^{-5}$ cm/s in Section 2.2.2.1 and $k_H = 2\times10^{-4}$ cm/s within Section 3.2 of Appendix A-1.

Thickness of GSS Unit
Attachment 1 of Appendix A-1 presents the "Thickness Contours of Glaciolacustrine Silt/Sand Unit" (Exhibit E). According to this drawing, the thickness of the GSS unit varies from 25 feet to less than 5 feet within the footprint of the proposed RMU-2 landfill. Thickness contours shown in Exhibit E are not consistent with those from a more recent drawing by Golder Associates (Hydrogeologic Characterization Update, 2014), included here as Exhibit F.

According to Section 3.2 of Appendix A-1 titled "Selection of Soil Properties for Geotechnical Evaluation of RMU-2 Design", prepared by P. J. Carey & Associates, PC (PJCA), the GC unit will experience compression (known as consolidation) under the load from the landfill unit. This process will generate excess pore water pressure.

According to the report, *"The GSS is the primary pathway for drainage of consolidation water from the GC as loads are applied. The thichness (sic) and continuity of the GSS, site wide, has been previously documented. For convenience Figures from the Golder 2002 study depicting the thickness of the GSS (Figure 3 of Golder) has been included in Attachment 1."*

The figure referred to in the last sentence is included here as Exhibit E ("Thickness contours of Glaciolacustrine Silt/Sand Unit").

The long-term slope stability of RMU-2 relies on the assumption that, while the GC unit consolidates under load, any excess pore water pressure will be drained through the GSS unit [Section 3.4.1 and Appendix C-5 of Engineering Report].

The hydraulic characteristics of the GSS unit is highly variable, as discussed in the expert report by Dr. Andrew Michalski. Both the hydraulic conductivity and the thickness of this unit vary widely within short distances. Table 5 of the Hydrogeologic Characterization Update by Golder Associates (2014) presents the hydraulic conductivity of the GSS unit, measured in different wells.

Two important conclusions can be drawn by reviewing hydraulic conductivities in wells near the northern part of the site (e.g., wells R202D through R208D). First, that the hydraulic conductivity varies over a wide range (as much as three orders of magnitude, from 1×10^{-3} to 1×10^{-6} cm/s), attesting to the variable and heterogeneous nature of the GSS unit. Second, there are areas below the RMU-2 footprint where the GSS unit has hydraulic conductivities on the order of 1×10^{-6} cm/s. Deposits with such low conductivity will not only not act as a pathway for drainage layer of consolidation water from the GC unit (contrary to the statement in the applicant's Engineering Report), but will in fact, itself undergo consolidation under undrained condition and exacerbate the problem of excess pore water pressure.

A review of Exhibits E and F indicates a general lack of borings under the proposed RMU-2 footprint. There are only a few contour lines through the footprint area, connecting points that are very far apart. This is especially true for the western portion of the site. Thus, the applicant has relied on borings outside the RMU-2 footprint and interpolation over large distances to calculate presumed thickness of the GSS unit.

According to the applicant's Engineering Report (Section 2.2.1), the thickness of the GSS unit varies between 0 ft and 25 ft below the RMU-2 footprint.

Therefore, the claim in the geotechnical evaluation (Appendix A-1) about the GSS unit being continuous sitewide is contradicted by the applicant's own statement that the GSS unit is absent (0 ft thick) in some area below the RMU-2 footprint. Given the complex nature of the unit and the general lack of information, the applicant's reliance on interpolation over large distances is not acceptable. This is especially true, given the critical role of the GSS unit in relieving excess pore water pressure and thus contributing to the stability of slopes of the proposed unit.

It is known that the GC unit will consolidate under load from the landfill and excess pore water pressure will develop and, if not allowed to dissipate, such pressure will have an adverse effect on the stability of the landfill. Therefore, the applicant must propose a viable method of dissipating that pressure.

The applicant must conduct a thorough soil boring program covering the footprint of RMU-2 to establish the hydraulic conductivity and thickness at different locations of the GSS unit, without having to rely on large-scale interpolation. Based on the results, the applicant may consider an underdrain system specifically engineered to dissipate excess pore water pressure during consolidation. In that event, the applicant must take into account the possible impacts of such an underdrain on the localized hydrologic conditions at the site.

8. UNCERTAINTY REGARDING SUBSURFACE CONDITIONS BELOW PROPOSED RMU-2

Thickness and elevation of important subsurface units below the proposed RMU-2 footprint are shown on drawings included in Exhibits A through F. Generally, the drawings indicate that there are only a relatively few boring locations within the proposed RMU-2 footprint. The applicant has relied on interpolation over large distances to draw the contour lines indicating presumed thickness and elevation of the GC and GSS units.

On the other hand, the same drawings show many boring locations immediately outside the RMU-2 footprint. The information from these borings generally indicate a highly variable nature of the unit thickness and elevation, such that the contour lines follow rather complex patterns.

The general absence of borings within the proposed RMU-2 footprint, together with evidence of highly variable nature of the subsurface units raises questions about the validity of the applicant's reliance on contour lines interpolated over large distances as shown in Exhibits A through F.

The applicant has not presented sufficient information to establish the thickness and elevation of the GC and GSS units with any degree of certainty. Additional borings will be necessary below the RMU-2 footprint to determine the nature of the these units, which are critical to the design.

9. LEACHATE GENERATION RATES

According to Section 4.2 (Site Preparation) of the Engineering Report, the applicant proposes to remove Fac Ponds 3 and 8 and construct Fac Pond 5. The new Fac Pond 5 will have a usable capacity of 21.9 million gallons. It will work in conjunction with existing Fac Ponds 1 and 2, which have a combined capacity of 19.3 million gallons.

According to the applicant's engineering report: *"These capacities will be sufficient to manage the annual volume of treated wastewater prior to annual discharges at the facility. Generally, one batch will be qualified and discharged per year in accordance with the SPDES permit. A typical volume is between 15 and 20 million gallons per year."*

Calculations for the Fac Pond storage capacity are presented by the applicant in Appendix L (Facultative Pond Capacity Evaluation) of the engineering report. Upon development of RMU-2, the Fac Ponds will receive treated leachate from RMU-1, RMU-2, five other landfills (SLF 1-6, -7, -10, -11 and -12) as well as additional "site waters" and any liquid hazardous waste receipts treated in the AWTS. The applicant has used projected leachate generation rates to estimate the discharge for years starting from 2013.

According to a footnote in Appendix L:

"Leachate generation rates for entirely capped/closed RMU-1 estimated based on the actual leachate generated from SFL-12 (sic) upon closure in 1995"

The approach used by the applicant is flawed for the following reasons:

1. SLF-12 has an area of 22 acres, while RMU-1 has an area of 47.1 acres. Thus, estimating leachate generation rates for the closed RMU-1, based on the actual leachate generated from SLF-12 (which is less than half in size) is not reasonable.

2. Data on predicted and actual leachate generation rates from different units at the site are available from Annual Water Budget Summary Reports for the site. The data show that there are often large differences between the predicted and actual leachate volumes. Tables from the 2009 report for SLF 1-6, SLF-7, SLF-10, SLF-11, and SLF-12, comparing predicted and actual figures from 1985 to 2009, are presented in Exhibit G. As can be seen from the tables, in numerous instances (e.g., 2000 to 2009 for SLF 1-6), the actual leachate volume was two or three times greater than what was predicted by the applicant for ten consecutive years.

3. In the 2009 report, the applicant has mentioned that *"Nonlinear trends are possibly due to the relatively unsophisticated leachate collection system design and the nature of the contained waste, which likely allows pockets of leachate to periodically break through toward the leachate pumps."*

The data in Exhibit G demonstrate that the applicant has underestimated leachate generation rates at this site over a significant period of time (from 1985 to 2009). The applicant has also acknowledged that nonlinear trends in leachate data makes it difficult to accurately predict leachate generation rates at this site. Based on these, the applicant's estimates for leachate capacity at Fac Pond 1&2 and Fac Pond 5, as presented in the engineering report, are not adequate.

10. CONCLUSIONS

Several deficiencies of the engineering report have been discussed.

- The shear strength properties of the bottom liner system, which are used in stability analyses, are flawed. They are erroneously based on a peer review panel report for an earlier unit, where a different liner configuration was used.

- The subgrade below the RMU-2 and Fac Pond 5 footprints is not of a suitable quality to protect the underlying aquifer. The proposed design does not allow necessary vertical separation between the bottom of RMU-2 and Fac Pond 5 and the historical high groundwater level, as required by regulation.

- The applicant has failed to take into account the inherent variabilities in the GSS unit underlying the site. The hydraulic conductivities stated in the different parts of the report are mutually inconsistent and do not meet the regulatory requirement.

- Several uncertainties exist regarding the condition of subgrade below the RMU-2 footprint. There is a general paucity of data points necessary to estimate the thickness of the GSS and GC units which underlie the site. Thus, the applicant has interpolated data between widely spaced borings, even though the available data indicate that the thickness of the units is widely variable across the site. In addition, the subsurface information provided in the Engineering Report (2013) conflicts with information presented in the more recent Hydrogelogic Characterization Update (2014).

- Calculations for uplift hydrostatic pressure are based on piezometric levels which are not consistent with more recent data presented in the Hydrogelogic Characterization Update (2014). Based on the 2014 information, the proposed design will result in excavation that will penetrate the GC unit and enter the underlying GSS unit. The applicant has ignored several uncertainties in the design and assumed a factor of safety of 1.0 for sump areas, which does not allow for any margin of error. In case of Fac Pond 5, the design elevation does not meet the necessary factor of safety of 1.2 for areas adjacent to the sump. The applicant's proposed approach of verifying factor of safety during construction fails to provide reasonable assurance of compliance. Also, the groundwater levels assumed in the uplift analyses and slope stability analyses are not in agreement with each other.

- The applicant has not provided data to support the selection of "historical high groundwater level" and has not stated if the design takes into account effects of future climate change.

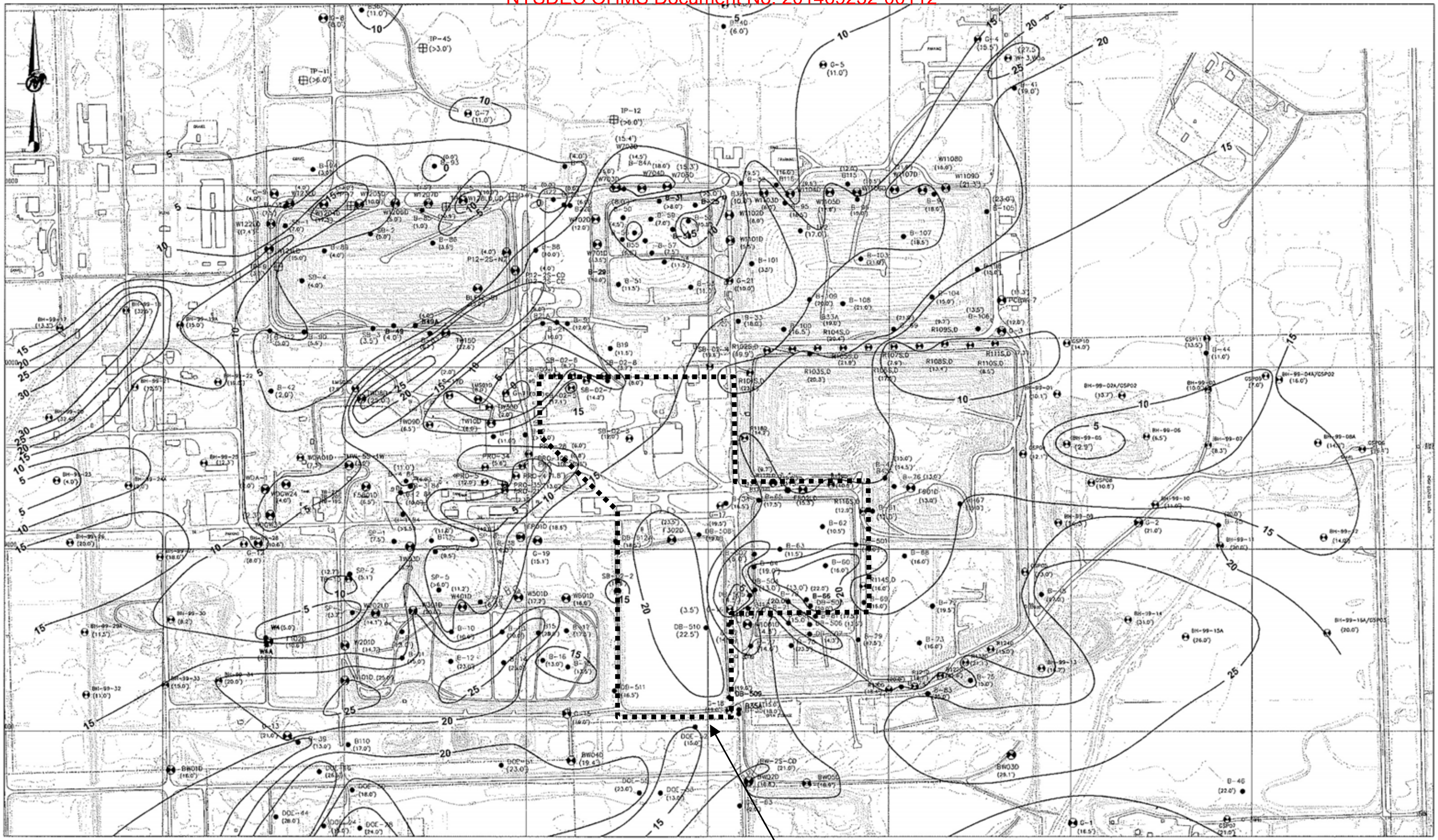
- Finally, based on predicted volumes of leachate generation, the applicant has assumed that the proposed facultative ponds will have sufficient capacity to handle all leachate at the site. Yet, in the past, the applicant's predicted leachate volumes have fallen short of the actual leachate generation rates by as much as a factor of three and the applicant has acknowledged a non-linear trend and unsophisticated leachate collection system at the site.

The engineering report presented by the applicant is inadequate, since it does not meet the requirements of applicable regulations and is not protective of human health and the environment.

Exhibits

Exhibit A

("Thickness Contours of Galciolacustrine Clay Unit",
included as Attachment B, in Appendix C-4 of the Engineering Report)



Approximate outline of RMU-2 footprint

SCALE 200 0 200 400 FEET

LEGEND

—— 10 ——	THICKNESS CONTOUR IN FEET
309.65	GROUND WATER ELEVATIONS
BW01D	BORING/WELL DESIGNATION
(16.0')	THICKNESS OF GLACIOLACUSTRINE CLAY IN FEET
⊕	BORINGS BY GOLDER ASSOCIATES
•	BORINGS BY OTHERS

NOTES

1.) CONTOUR INTERPRETATION IS BASED ON BORING DATA FROM THE 1985 HYDROGEOLOGIC CHARACTERIZATION AND SUBSEQUENT STUDIES AT THE MODEL CITY FACILITY CONDUCTED BY GOLDER ASSOCIATES AND OTHERS.

2.) DATA CONCERNING THE VARIOUS STRATAS HAVE BEEN OBTAINED FROM THE BOREHOLE LOCATIONS ONLY. CONSISTENT WITH GLACIAL ENVIRONMENTS, ABRUPT CHANGES IN SOIL STRATIGRAPHY HAVE BEEN OBSERVED BETWEEN BOREHOLES OVER SHORT DISTANCES. CONSEQUENTLY, ALTHOUGH CONTOUR INTERPRETATION HAVE BEEN INFERRED FROM THE BOREHOLE DATA, THE ACTUAL CONTOURS MAY VARY FROM THAT SHOWN.

3. DATA WITH A LINE THROUGH THE VALUE ARE NOT USED IN THE INTERPRETATION DUE TO UNCERTAINTY OF DEPTH PICKS OR CONFLICTING LOCAL STRATIGRAPHY INFORMATION.

REFERENCES

1.) BASE MAP COMPILED BY PHOTOGRAMMETRIC METHODS FROM AERIAL PHOTOGRAPHY DATED MAY 31, 2001 BY AIR SURVEY CORP., DULLES, VIRGINIA.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
CWM CHEMICAL SERVICES, L.L.C.
MODEL CITY, NEW YORK

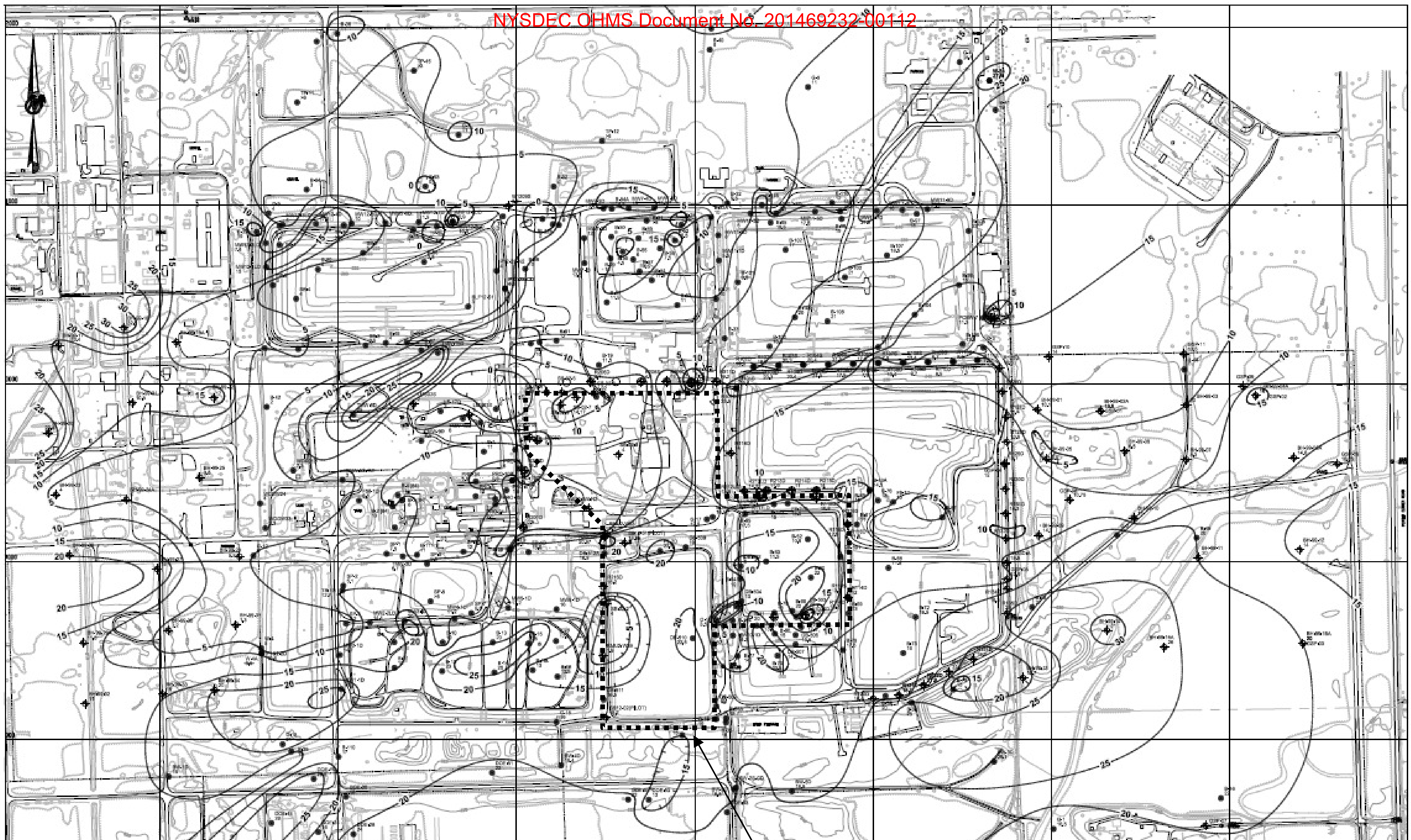
TITLE
THICKNESS CONTOURS OF GLACIOLACUSTRINE CLAY UNIT

Golder Associates
Buffalo USA

PROJECT No.	013-9309		FILE No.	0139309C152
DESIGN	JPR	01/07/02	SCALE AS SHOWN	REV. 0
CADD	AM/JPR	05/12/03	FIGURE 2	
CHECK	JPR	05/12/03		
REVIEW	DCW	05/12/03		

Exhibit B

(“Thickness of Galciolacustrine Clay”,
included as Figure 9 of the 2013 Hydrogeologic Characterization Update)



LEGEND

- 1993 AND NEWER GROUND PENETRATIONS
- PRE 1993 GROUND PENETRATIONS
- BW03D GLACIOLACUSTRINE CLAY UNIT PENETRATION DESIGNATION
- 20 THICKNESS OF GLACIOLACUSTRINE CLAY (FT)
- 5 LINE OF EQUAL POTENTIAL

NOTES

1.) THICKNESS CONTOURS SHOWN INCLUDE ENTIRE CLAY UNIT WHERE IT OCCURS AS A SINGLE UNIT AND ONLY THE UPPER PORTION WHERE THERE IS AN UPPER AND LOWER CLAY UNIT.

2.) DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. CONSISTENT WITH GLACIAL ENVIRONMENTS, ABRUPT CHANGES IN SOIL STRATIGRAPHY HAVE BEEN OBSERVED BETWEEN BOREHOLES OVER SHORT DISTANCES. CONSEQUENTLY, ALTHOUGH CONTOUR INTERPRETATIONS HAVE BEEN INFERRED FROM THE BOREHOLE DATA, THE ACTUAL CONTOURS MAY VARY FROM THAT SHOWN.

3.) DATA WITH A LINE THROUGH THE VALUE ARE NOT USED IN THE INTERPRETATION DUE TO THE UNCERTAINTY OF DEPTH PICKS OR CONFLICTING STRATIGRAPHIC INFORMATION.

REFERENCES

1.) BASE MAP COMPILED BY PHOTOGRAMMETRIC METHODS FROM AERIAL PHOTOGRAPHY DATED MAY 31, 2001 BY AIR SURVEY CORP., DULLES, VIRGINIA.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
1	1/31/14	RJW	[illegible]	RJW	DCW	DCW

PROJECT
CWM CHEMICAL SERVICES, L.L.C.
2013 HYDROGEOLOGIC CHARACTERIZATION UPDATE
MODEL CITY, NEW YORK

TITLE
THICKNESS OF GLACIOLACUSTRINE CLAY

Golder Associates
Buffalo, NY

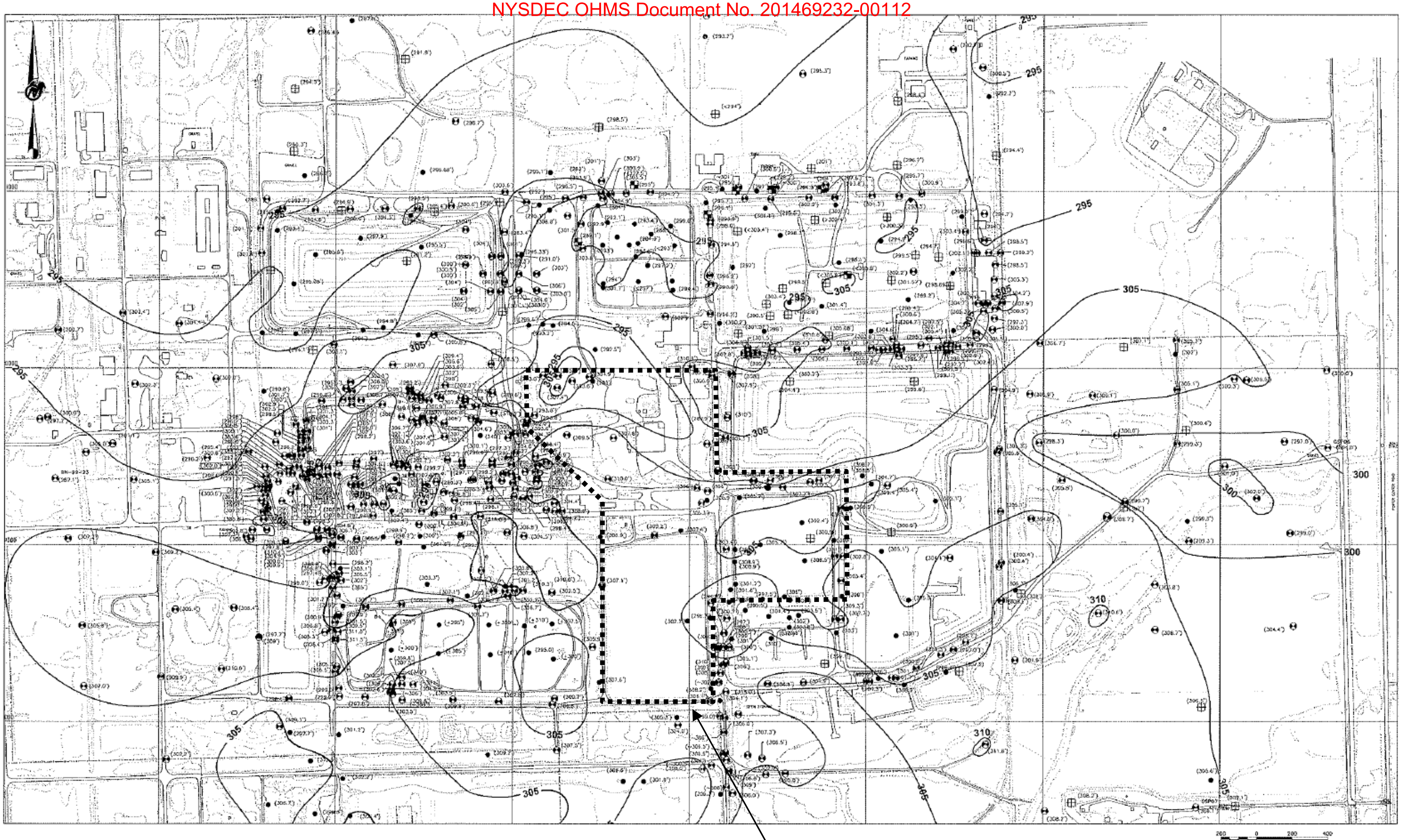
PROJECT No.		123-89194
DESIGN	AML	7/15/13
CADD	RJW	1/21/14
CHECK	RJW	1/21/14
REVIEW	DCW	2/3/14

FILE No. [illegible]
SCALE AS SHOWN REV. 1

FIGURE 9

Exhibit C

("Top of Glaciolacustrine Clay Unit Contours"
included as Attachment A to Appendix C-4 of the 2013 Engineering Report)



LEGEND

280	ELEVATION CONTOUR IN FEET SITE DATUM
BW010	BORING/WELL DESIGNATION
(281.2)	TOP OF GLACIOLACUSTRINE CLAY UNIT IN FEET SITE DATUM
	BORINGS BY GOLDER ASSOCIATES
	BORINGS BY OTHERS
	MONITORING TRENCHES BY OTHERS
	TEST PIT BY OTHERS

NOTES

1.) CONTOUR INTERPRETATION IS BASED ON BORING DATA FROM THE 1986 HYDROGEOLOGIC CHARACTERIZATION AND SUBSEQUENT STUDIES AT THE MODEL CITY FACILITY CONDUCTED BY GOLDER ASSOCIATES AND OTHERS.

2.) DATA CONCERNING THE VARIOUS STRATAS HAVE BEEN OBTAINED FROM THE BOREHOLE LOCATIONS ONLY. CONSISTENT WITH GLACIAL ENVIRONMENTS, ABRUPT CHANGES IN SOIL STRATIGRAPHY HAVE BEEN OBSERVED BETWEEN BOREHOLES OVER SHORT DISTANCES. CONSEQUENTLY, ALTHOUGH CONTOUR INTERPRETATION HAVE BEEN INFERRED FROM THE BOREHOLE DATA, THE ACTUAL CONTOURS MAY VARY FROM THAT SHOWN.

3.) DATA WITH A LINE THROUGH THE VALUE ARE NOT USED IN THE INTERPRETATION DUE TO UNCERTAINTY OF DEPTH PICKS OR CONFLICTING LOCAL STRATIGRAPHY INFORMATION.

4.) AN INTERVAL OF FIVE FEET WAS USED TO CONTOUR THE TOP OF GLACIOLACUSTRINE CLAY IN THE EASTERN AREA (EAST OF RMU-1), WHILE A TEN FOOT INTERVAL WAS USED FOR THE REST OF THE SITE.

REFERENCES

1.) BASE MAP COMPILED BY PHOTOGRAMMETRIC METHODS FROM AERIAL PHOTOGRAPHY DATED MAY 31, 2001 BY AIR SURVEY CORP., DULLES, VIRGINIA.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
CWM CHEMICAL SERVICES, L.L.C.
MODEL CITY, NEW YORK

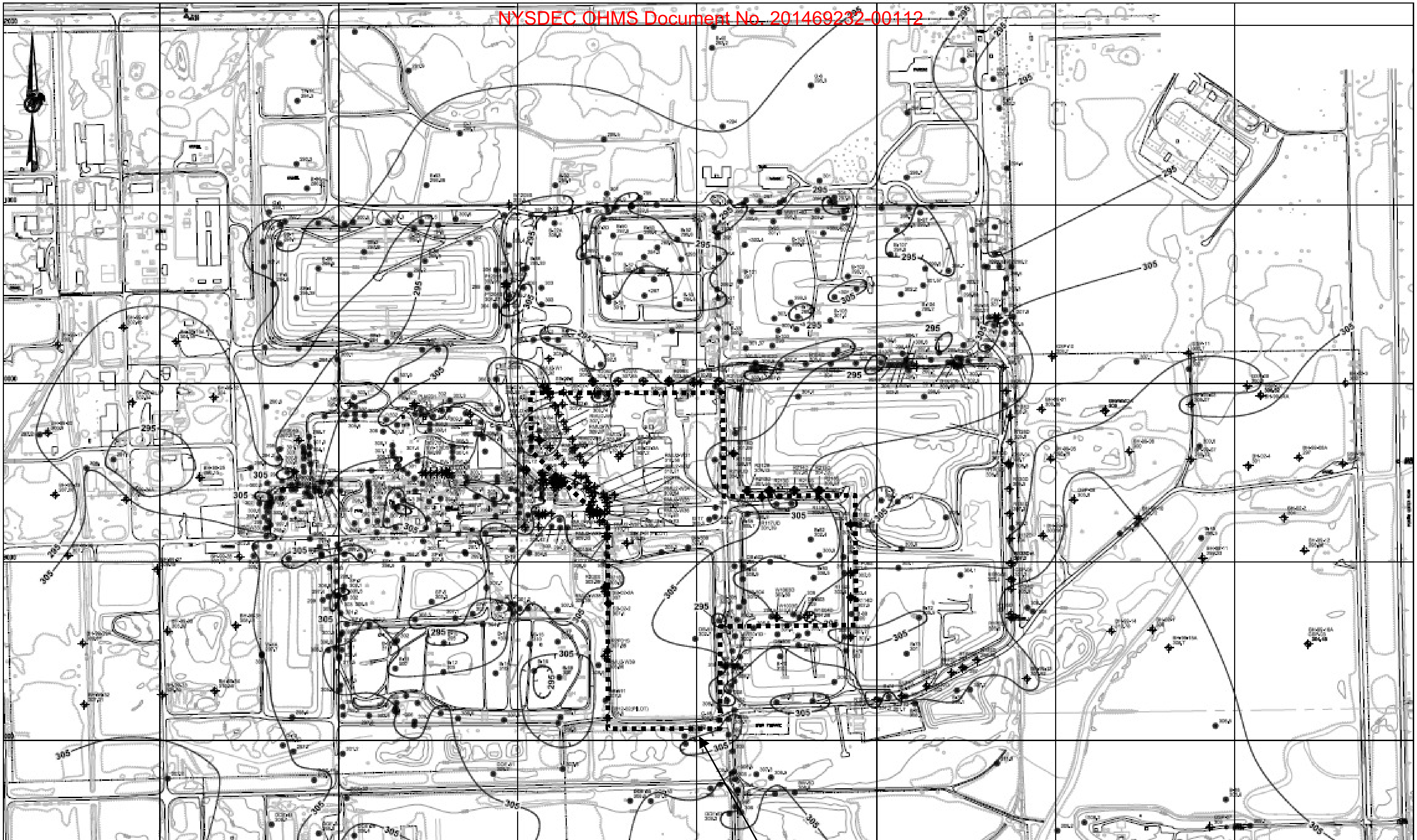
TITLE
TOP OF GLACIOLACUSTRINE CLAY UNIT CONTOURS

Golder Associates
Buffalo USA

PROJECT No.		013-6309	FILE No.	01303080153
DESIGN	JPR	01/07/02	SCALE AS SHOWN	REV. 0
CADD	AM/JPR	10/03/02		
CHECK	JPR	5/12/03	FIGURE 1	
REVIEW	DCW	05/12/03		

Exhibit D

("Top of Glaciolacustrine Clay"
included as Figure 10 of the 2013 Hydrogeologic Characterization Update)



LEGEND

- 1983 AND NEWER GROUND PENETRATIONS
- PRE 1983 GROUND PENETRATIONS
- BW03D — UNIT PENETRATION DESIGNATION
- TOP OF GLACIOLACUSTRINE CLAY ELEVATION
- 305 — LINE OF EQUAL POTENTIAL

NOTES

1.) DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. CONSISTENT WITH GLACIAL ENVIRONMENTS, ABRUPT CHANGES IN SOIL STRATIGRAPHY HAVE BEEN OBSERVED BETWEEN BOREHOLES OVER SHORT DISTANCES. CONSEQUENTLY, ALTHOUGH CONTOUR INTERPRETATIONS HAVE BEEN INFERRED FROM THE BOREHOLE DATA, THE ACTUAL CONTOURS MAY VARY FROM THAT SHOWN.

2.) DATA WITH A LINE THROUGH THE VALUE ARE NOT USED IN THE INTERPRETATION DUE TO THE UNCERTAINTY OF DEPTH PICKS OR CONFLICTING STRATIGRAPHIC INFORMATION.

REFERENCES

1.) BASE MAP COMPILED BY PHOTOGRAMMETRIC METHODS ... PHOTOGRAPHY DATED MAY 31, 2001 BY ... SURVEY CO...

200 0 200 400
SCALE FEET

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
CWM CHEMICAL SERVICES, L.L.C.
2013 HYDROGEOLOGIC CHARACTERIZATION UPDATE
MODEL CITY, NEW YORK

TITLE
TOP OF GLACIOLACUSTRINE CLAY

Golder Associates
Buffalo, NY

PROJECT No.	103-89496		FILE No.	[illegible]
DESIGN	AML	07/15/13	SCALE AS SHOWN	REV. 0
CADD	AML	11/25/13		
CHECK	RJM	1/21/14		
REVIEW	DCW	2/3/14		

FIGURE 10

Exhibit E

("Thickness Contours of Glaciolacustrine Silt/Sand Unit",
included as Attachment 1 of Appendix A-1 of the Engineering Report)

LEGEND

— 10 —	THICKNESS CONTOUR IN FEET SITE DATUM
BW01D	BORING/WELL DESIGNATION
(10.0)	THICKNESS OF GLACIOLACUSTRINE SILT/SAND UNIT IN FEET
⊕	BORINGS BY GOLDER ASSOCIATES
●	BORINGS BY OTHERS

NOTES

1.) CONTOUR INTERPRETATION IS BASED ON BORING DATA FROM THE 1985 HYDROGEOLOGIC CHARACTERIZATION AND SUBSEQUENT STUDIES AT THE MODEL CITY FACILITY CONDUCTED BY GOLDER ASSOCIATES AND OTHERS.

2.) DATA CONCERNING THE VARIOUS STRATAS HAVE BEEN OBTAINED FROM THE BOREHOLES LOCATION ONLY. CONSISTENT WITH GLACIAL ENVIRONMENTS, ABRUPT CHANGES IN SOIL STRATIGRAPHY HAVE BEEN OBSESRVED BETWEEN BOREHOLES OVER SHORT DISTANCES. CONSEQUENTLY, ALTHOUGH CONTOUR INTERPRETATION HAVE BEEN INFERRED FROM THE BOREHOLE DATA, THE ACTUAL CONTOURS MAY VARY FROM THAT SHOWN.

3.) DATA WITH A LINE THROUGH THE VALUE ARE NOT USED IN THE INTERPRETATION DUE TO UNCERTAINTY OF DEPTH PICKS OR CONFLICTING LOCAL STRATIGAPHY INFORMATION.

REFERENCES

1.) BASE MAP COMPILED BY PHOTOGRAMMETRIC METHODS FROM AERIAL PHOTOGRAPHY DATED MAY 31, 200_ BY AIR SURVEY CORP., DULLES, VIRGINIA.

200 0 200 400
SCALE FEET

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
CWM CHEMICAL SERVICES, L.L.C.
MODEL CITY, NEW YORK

TITLE
THICKNESS CONTOURS OF GLACIOLACUSTRINE SILT/SAND UNIT

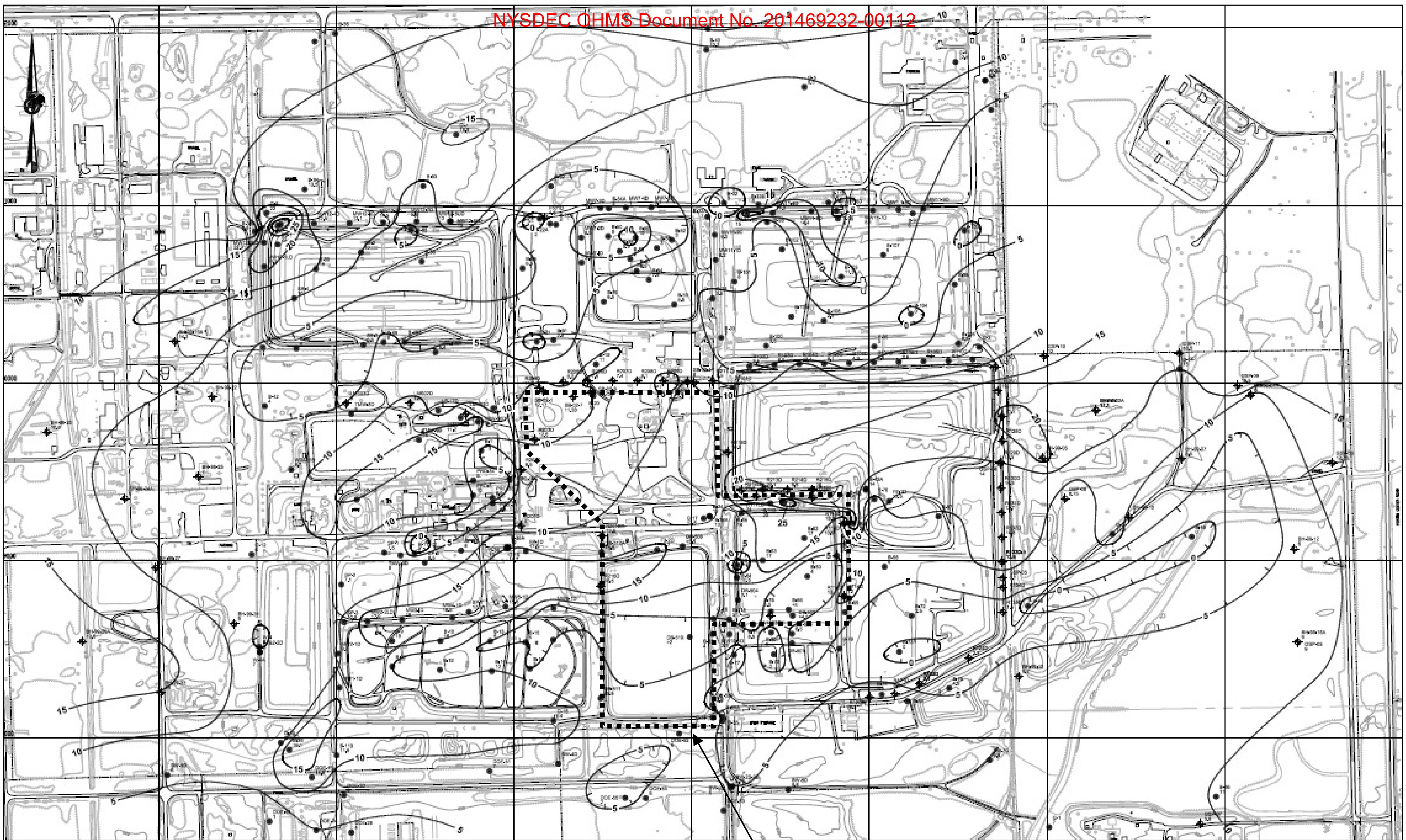
Golder Associates
Buffalo USA

PROJECT No.	013-9309		FILE No.	0139309B151
DESIGN	JPR	01/07/02	SCALE AS SHOWN	REV. 0
CADD	AM/JPR	10/03/02		
CHECK				
REVIEW				

FIGURE 3

Exhibit F

("Thickness of Glaciolacustrine Silt/Sand"
included as Figure 8 of the 2013 Hydrogeologic Characterization Update)



LEGEND

✦	1993 AND NEWER GROUND PENETRATIONS
●	PRE 1993 GROUND PENETRATIONS
BW03D	GLACIOLACUSTRINE SILT/SAND UNIT PENETRATION DESIGNATION
	THICKNESS OF GLACIOLACUSTRINE SILT/SAND
10 ———	LINE OF EQUAL POTENTIAL

NOTES

1.) DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. CONSISTENT WITH GLACIAL ENVIRONMENTS, ABRUPT CHANGES IN SOIL STRATIGRAPHY HAVE BEEN OBSERVED BETWEEN BOREHOLES OVER SHORT DISTANCES. CONSEQUENTLY, ALTHOUGH CONTOUR INTERPRETATIONS HAVE BEEN INFERRED FROM THE BOREHOLE DATA, THE ACTUAL CONTOURS MAY VARY FROM THAT SHOWN.

2.) DATA WITH A LINE THROUGH THE VALUE ARE NOT USED IN THE INTERPRETATION DUE TO THE UNCERTAINTY OF DEPTH PICKS OR CONFLICTING STRATIGRAPHIC INFORMATION.

REFERENCES

1.) BASE MAP COMPILED BY PHOTOGRAMMETRIC METHODS FROM AERIAL PHOTOGRAPHY DATED MAY 31, 2001 BY AIR SURVEY CORP., DULLES, VIRGINIA.

200 0 200 400
SCALE FEET

PROJECT
CWM CHEMICAL SERVICES, L.L.C.
2013 HYDROGEOLOGIC CHARACTERIZATION UPDATE
MODEL CITY, NEW YORK

TITLE
THICKNESS OF
GLACIOLACUSTRINE SILT/SAND

Golder Associates
Buffalo, NY

FIGURE 8

Exhibit G

(Excerpts from “2009 Annual Water Budget Summary Report)

MODIFIED LINEAR REGRESSION OF SLF 1-6

Correlation of Yearly Flows VS. Time (Gal/Yr)

Years After Closure	Year	Leachate Removed (Actual)	Leachate Removed (Predicted)
1	1985	984,600	238,014
2	1986	376,724	225,798
3	1987	166,413	214,326
4	1988	120,377	203,556
5	1989	87,839	193,428
6	1990	163,109	183,906
7	1991	209,380	174,936
8	1992	285,905	166,494
9	1993	205,017	158,538
10	1994	224,322	151,038
11	1995	96,113	143,964
12	1996	60,998	67,800
13	1997	169,171	67,800
14	1998	201,010	67,800
15	1999	121,108	120,000
16	2000	149,532	67,800
17	2001	147,799	65,540
18	2002	128,995	63,280
19	2003	119,500	61,020
20	2004	145,171	56,500
21	2005	147,496	54,240
22	2006	124,872	51,980
23	2007	149,140	47,460
24	2008	137,476	46,330
25	2009	132,155	45,200
26	2010		44,070
27	2011		42,940
28	2012		41,810
29	2013		40,680
30	2014		39,550

Notes:

For 1995 and earlier, Rust Environment & Infrastructure calculated the Predicted Leachate Removed using a Modified Linear Regression Analysis. For 1996 and later, Golder Associates calculated the Predicted Leachate Removed using a calculated Infiltration "I" factor.

MODIFIED LINEAR REGRESSION OF SLF 7

Correlation of Yearly Flows VS. Time (Gal/Yr)

Years After Closure	Year	Leachate Removed (Actual)	Leachate Removed (Predicted)
1	1985	533,700	134,358
2	1986	254,603	108,954
3	1987	76,870	89,166
4	1988	25,587	73,614
5	1989	36,480	61,260
6	1990	32,588	51,366
7	1991	37,770	43,386
8	1992	42,317	36,888
9	1993	35,407	31,554
10	1994	33,039	27,162
11	1995	37,352	23,508
12	1996	47,543	33,900
13	1997	57,610	33,900
14	1998	47,891	33,900
15	1999	44,672	36,160
16	2000	31,491	33,900
17	2001	30,773	32,770
18	2002	37,864	31,640
19	2003	29,061	30,510
20	2004	36,777	28,250
21	2005	56,037	27,120
22	2006	35,733	25,990
23	2007	25,324	23,730
24	2008	33,134	23,165
25	2009	69,850	22,600
26	2010		22,035
27	2011		21,470
28	2012		20,905
29	2013		20,340
30	2014		19,775

Notes:

For 1995 and earlier, Rust Environment & Infrastructure calculated the Predicted Leachate Removed using a Modified Linear Regression Analysis. For 1996 and later, Golder Associates calculated the Predicted Leachate Removed using a calculated Infiltration "I" factor.

Prior to August 2008, both SLF 7 and SLF 11 leachates were collected in tank T-108, and it was assumed that the total leachate volume in this tank was comprised of 15% SLF 7 and 85% SLF 11, based upon historical information. Since August 2008, SLF 7 and SLF 11 leachate have been collected and measured separately in tanks T-107 and T-108, respectively.

MODIFIED LINEAR REGRESSION OF SLF 10

Correlation of Yearly Flows VS. Time (Gal/Yr)

Years After Closure	Year	Leachate Removed (Actual)	Leachate Removed (Predicted)
1	1987	133,922	134,904
2	1988	38,562	105,690
3	1989	50,951	83,838
4	1990	69,142	67,278
5	1991	56,254	54,564
6	1992	54,185	44,700
7	1993	36,156	36,960
8	1994	40,954	30,816
9	1995	28,508	25,902
10	1996	40,881	30,000
11	1997	35,356	30,000
12	1998	44,324	30,000
13	1999	32,525	39,000
14	2000	40,717	35,000
15	2001	41,516	32,000
16	2002	45,940	30,000
17	2003	36,090	29,000
18	2004	71,937	28,000
19	2005	64,899	27,000
20	2006	60,493	25,000
21	2007	58,348	24,000
22	2008	54,879	23,000
23	2009	48,204	21,000
24	2010		20,500
25	2011		20,000
26	2012		19,500
27	2013		19,000
28	2014		18,500
29	2015		18,000
30	2016		17,500

Notes:

For 1995 and earlier, Rust Environment & Infrastructure calculated the Predicted Leachate Removed using a Modified Linear Regression Analysis. For 1996 and later, Golder Associates calculated the Predicted Leachate Removed using a calculated Infiltration "I" factor.

MODIFIED LINEAR REGRESSION OF SLF 11

Correlation of Yearly Flows VS. Time (Gal/Yr)

Years After Closure	Year	Leachate Removed (Actual)	Leachate Removed (Predicted)
1	1992	686,761	666,936
2	1993	550,320	611,844
3	1994	572,622	562,056
4	1995	503,300	517,002
5	1996	377,071	91,120
6	1997	251,147	85,760
7	1998	229,441	83,080
8	1999	253,144	160,800
9	2000	178,441	142,040
10	2001	174,368	134,000
11	2002	214,570	117,920
12	2003	164,681	109,880
13	2004	208,406	104,520
14	2005	317,546	93,800
15	2006	202,487	85,760
16	2007	143,502	80,400
17	2008	150,829	77,720
18	2009	137,678	75,040
19	2010		72,360
20	2011		67,000
21	2012		64,320
22	2013		61,640
23	2014		56,280
24	2015		54,940
25	2016		53,600
26	2017		52,260
27	2018		50,920
28	2019		49,580
29	2020		48,240
30	2021		46,900

Notes:

For 1995 and earlier, Rust Environment & Infrastructure calculated the Predicted Leachate Removed using a Modified Linear Regression Analysis. For 1996 and later, Golder Associates calculated the Predicted Leachate Removed using a calculated Infiltration "I" factor.

Prior to August 2008, both SLF 7 and SLF 11 leachates were collected in tank T-108, and it was assumed that the total leachate volume in this tank was comprised of 15% SLF 7 and 85% SLF 11, based upon historical information. Since August 2008, SLF 7 and SLF 11 leachate have been collected and measured separately in tanks T-107 and T-108, respectively.

The reported 2005 volume is likely due to the lowering of the leachate compliance level for SLF 11 from one foot over the top of operations layer to one foot over the top of the primary liner.

MODIFIED LINEAR REGRESSION OF SLF 12

Correlation of Yearly Flows VS. Time (Gal/Yr)

Years After Closure	Year	Leachate Removed (Actual)	Leachate Removed (Predicted)
1	1996	578,847	605,500
2	1997	363,240	259,500
3	1998	194,073	103,800
4	1999	114,015	198,950
5	2000	142,807	159,160
6	2001	77,273	136,670
7	2002	61,321	117,640
8	2003	67,446	103,800
9	2004	66,410	91,690
10	2005	86,658	86,500
11	2006	69,430	76,120
12	2007	62,631	70,930
13	2008	43,816	67,470
14	2009	45,318	60,550
15	2010		55,360
16	2011		51,900
17	2012		50,170
18	2013		48,440
19	2014		46,710
20	2015		43,250
21	2016		41,520
22	2017		39,790
23	2018		36,330
24	2019		35,465
25	2020		34,600
26	2021		33,735
27	2022		32,870
28	2023		32,005
29	2024		31,140
30	2025		30,275

Note:

Predicted Leachate Removed per year was calculated by Golder Associates using a calculated Infiltration "I" factor.