CDM

City of Chesapeake

Battlefield Golf Club
Hydrogeologic Investigation and Groundwater Modeling Report

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Report

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Section 1
Introduction

This Hydrogeologic Investigation and Groundwater Modeling Report has been prepared by Camp Dresser & McKee Inc. (CDM) for Huff, Poole & Mahoney, P.C. (HPM), Attorneys for the City of Chesapeake, Virginia (City). This report presents the results of investigation activities completed by CDM for the Battlefield Golf Club (site) located in Chesapeake, Virginia (**Figure 1-1**). The hydrogeologic investigation was performed to assess water quality at the site and to support the development of a groundwater model. The groundwater model was developed to assess the potential migration of constituents derived from fly ash that was deposited as fill beneath the golf course.

The general data needs for the groundwater modeling effort were identified in a previously prepared ***Preliminary Site Assessment Work Plan*** (CDM, September 2008). Efforts to fulfill the data needs identified in the work plan were performed by CDM and included groundwater and surface water monitoring, an aquifer performance test (APT), and fly ash sample collection and laboratory analyses.

Additional onsite work was completed by MACTEC Engineering and Consulting, Inc. (MACTEC) for Dominion Generation (Dominion) that included monitoring well installation, groundwater and surface water monitoring, characterization of the golf course cover and fly ash, and characterization of the hydrogeologic conditions. MACTEC previously submitted a Sampling and Analysis Plan (MACTEC, October 16, 2008) that detailed the work to be performed by MACTEC. The results of the MACTEC work were reported in the ***Post-Construction Ash Fill, Soil Cover and Groundwater Evaluation Report*** (MACTEC, December 17, 2009).

Work was also completed by URS Corporation (URS). The work performed by URS consisted of offsite monitoring well installation, groundwater and surface water monitoring, and collection of hydrogeologic data. This work was reported in the ***Water Supply Feasibility Study*** (URS, April 10, 2009).

The U.S. Environmental Protection Agency (EPA) completed several studies of the site that were performed by Tetra Tech, Inc. (Tetra Tech). These studies included the ***Draft Site Inspection for the Battlefield Golf Club Site*** (Tetra Tech, 2009) and the ***Final Site Inspection for the Battlefield Golf Club Site*** (Tetra Tech, 2010).

The remainder of this introduction section provides a brief project background and describes the site environmental setting. Section 2 describes CDM's data collection and Section 3 presents the investigation results. Sections 4, 5, 6, and 7 present the groundwater modeling effort and results. CDM's conclusions are provided in Section 8 and references are included in Section 9. Laboratory reports are included in the Appendices along with a water quality database and APT data.

1.1 Site Location

The site is located at 1001 Centerville Turnpike South on the east side of the road, south of Whittamore Road, and north of Murray Drive in the City of Chesapeake, Virginia (**Figure 1-2**). The site is bounded by residential properties to the south along Murray Drive. Additional residential properties are located to the east-northeast on Whittamore Road and to west beyond Centerville Turnpike. Agricultural properties exist to the north and east of the site and beyond the residential properties to the south.

The Battlefield Golf Club covers approximately 217 acres and opened to the public on October 13, 2007. Prior to the construction of the golf course, fly ash derived from the burning of coal was used as fill material. The fly ash was then covered with soil for the construction of the golf course. Groundwater wells have historically been used by residents in the vicinity of the golf course but these residents are now served by the City's municipal supply. Environmental concerns over the potential degradation of groundwater and surface water quality associated with the fly ash fill are the subject of CDM's investigation and modeling work.

1.2 Environmental Setting

1.2.1 Climate

Chesapeake is located within the Tidewater climate region of Virginia (University of Virginia Climatology Office). The area averages approximately 51 inches of precipitation annually and the average temperature is approximately 57 degrees Fahrenheit. The monthly average values are included on **Figure 1-3**.

1.2.2 Topography and Drainage

The topography of the site vicinity is very flat, with a gradual slope to the east toward the Atlantic Ocean. The west border of the site is at an elevation of approximately 20 feet above mean sea level (msl) and the east border is at an elevation of approximately 10 feet above msl. Prior to ash fill placement, the area of the golf course ranged in elevation from approximately 10 to 15 feet above msl. Fly ash fill and a soil cover were emplaced that reportedly created elevations on the golf course as high as 40 feet. Therefore, CDM assumes that the depth to the fly ash base is approximately 25 below land surface (bls) at the areas that have the highest elevations. The current topography of the golf course has not been surveyed but proposed topographic plans were prepared prior to the construction.

The site vicinity has a network of surface water drainage ditches (Figure 1-1). The topographic map in Figure 1-1 was prepared in 2003. Since that time, many smaller drainage ditches have been filled from agricultural practices. Drainage in the ditches is generally from west to east. A portion of the drainage on the golf course is into the ponds that are used as water hazards and as source water for irrigation (Figure 1-2). Otherwise, the general runoff direction on the golf course is to the south into a drainage ditch that merges east of the site with the headwaters of a tributary, referred to in this report as the North Tributary to the Pocaty River. A second tributary to the Pocaty River is located further south and is referred to as the South Tributary in this report. The Intracoastal Waterway/Albemarle Canal is located approximately 2.5 miles north of the site.

1.2.3 Hydrogeology

The site is located within the Coastal Plain physiographic province of southeast Virginia. This area is underlain by an alternating sequence of aquifers and confining zones. The aquifers of interest to this report include the surficial aquifer, also referred to as the Columbia aquifer, and the underlying Yorktown-Eastover (Yorktown) aquifer. A generalized hydrogeologic section is shown on **Figure 1-4**.

The surficial aquifer is a heterogeneous aquifer, consisting of sand and gravel (Pope, 2008), that is locally interbedded with fine-grained sediments (McFarland, 2006). The top of the aquifer is at land surface and extends to an estimated depth of approximately 60 feet (McFarland, 2006) in the site vicinity. The surficial aquifer is unconfined and under water table conditions. The depth to groundwater in the site vicinity is generally less than 5 feet. The estimated transmissivity (T) of the surficial aquifer in the site vicinity ranges from 1,000 to 1,500 feet^2 per day (ft^2/d) (McFarland, 1998). The surficial aquifer is underlain by the Yorktown confining zone.

The Yorktown confining zone separates the underlying Yorktown aquifer from the overlying surficial aquifer and exhibits characteristics of both units (Pope, 2008). This is a heterogeneous zone generally defined as the uppermost silt/clay that is interbedded with glauconitic, phosphatic, and fossiliferous quartz sand. The Yorktown confining zone does not represent a distinct contact surface, but rather approximates a transition from the Yorktown aquifer to the surficial aquifer. The zone is approximately 20 to 30 feet thick in the site vicinity (McFarland, 2006). Because of the heterogeneity of this zone, the Yorktown confining zone may act as a semi-confining zone or may allow hydraulic communication between the surficial and the Yorktown aquifers on a localized basis. The estimated vertical leakance of the Yorktown confining zone is from 0.0001 to 0.001 inches per day (in/d) (McFarland, 1998).

The Yorktown aquifer is a heterogeneous unit composed of glauconitic, phosphatic, and fossiliferous quartz sand with interbedded silt/clay. The lower part consists of abundantly fossiliferous sands. The Yorktown aquifer is commonly used for domestic water supplies. This aquifer is present at an estimated depth of approximately 80 to 90 feet in the site vicinity. The T of the Yorktown aquifer in the site vicinity ranges from 1,000 to 2,000 ft^2/d (McFarland, 1998). The Yorktown aquifer is underlain by the Saint Mary's confining zone at an estimated depth of 130 feet to 140 feet and the estimated vertical leakance is from 0.00001 to 0.0001 in/d (McFarland, 2006).

Both the surficial aquifer and the Yorktown aquifer are used locally for residential water supplies. The average residential water supply well depth in Chesapeake is approximately 80 feet BLS (Pope, 2008). There are approximately 200 residential supply wells in the site vicinity. All of the residences in the immediate site vicinity, including those along Murray Drive and Whittamore Road, are currently supplied by the municipal provider.

1.3 Project Scope

The scope of the investigation completed by CDM included groundwater and surface water monitoring, an APT, fly ash sample collection, and laboratory analyses. **Table 1-1** includes a summary of the monitoring locations utilized by CDM and summarizes the sampling performed by others as well. For the purposes of this report, CDM has established a location code index that provides each location with a unique code. The unique codes used in this report are included in Table 1-1 along with corresponding alias codes used by others. This systematic approach to location codes is beneficial because non-unique location codes exist from the previous work and can be a source of confusion. These locations are mapped on **Figure 1-5**.

In addition to the monitoring, CDM also completed an APT that consisted of pumping a test well (TW-1) for a period of 72 hours while recording aquifer response in two piezometers (PZ-1 and PZ-2) and select monitoring wells. Additional hydrogeologic data were collected by URS and MACTEC by performing slug tests on select monitoring wells.

In an attempt to collect leachate samples from the fly ash, CDM constructed boreholes into the fly ash at three locations (LW-1, LW-2, and LW-3). Fly ash samples were collected from these locations. A field decision was then made at each location as to whether leachate was present in sufficient quantities to allow sample collection from a temporary well point. One leachate well was installed (LW-1) but did not provide sufficient recharge to allow sample collection.

Section 2
Hydrogeologic Investigation

2.1 Fly Ash Sampling

On August 11th and 12th of 2009, CDM collected fly ash samples for analyses from three borings (LW-1, LW-2, and LW-3). CDM had intended to install a monitoring well at these locations to collect water samples from within the fly ash to characterize the associated water quality. However, insufficient evidence of water was observed during drilling at two locations (LW-1 and LW-3) to warrant well installation. A well was installed at location LW-2 but this well went dry upon attempts to purge the well and no water sample was collected.

CDM collected depth-composited samples from the fly ash at three depth intervals from each boring. The samples were analyzed for total metals, ammonia nitrogen, nitrate, nitrite, and moisture content. **Table 2-1** provides a summary of the laboratory data from the fly ash samples and **Appendix A** includes the laboratory report. The boring locations are shown on Figure 1-5.

2.2 Groundwater and Surface Water Analyses

CDM collected 22 groundwater samples and 10 surface water samples in September of 2009 and 20 groundwater samples in October 2009. Water levels were also collected from 24 wells and 9 surface water staff gauges. The surface water data are summarized in **Table 2-2** and the groundwater data are summarized in **Table 2-3**. The full laboratory reports are included in **Appendix B**. **Table 2-4** includes a summary of duplicate water sample analyses.

Groundwater was purged from the wells using a peristaltic pump following the low-flow technique at flow rates less than 200 milliliters per minute. Parameters monitored during purging included temperature, pH, conductivity, dissolved oxygen, oxidation reduction potential, and turbidity. These parameters were monitored and purging continued until they had stabilized to within ten percent. Once the parameters had stabilized, samples were collected directly from the peristaltic pump tubing. All pump tubing was new upon use and discarded between each well location. The surface water samples were collected using the dip technique. The sample container was dipped into the water to allow the sample to flow into the sample container. The groundwater and surface water samples were analyzed for total metals, nitrate, nitrite, and ammonia by Test America in Pittsburgh, PA. Field analyses were also performed for ferrous iron, total iron, sulfate, and sulfide. These results are included in **Table 2-5**.

2.3 Aquifer Performance Test

During November 2009, CDM conducted an APT at test well TW-1 located up gradient of the site. The APT pumping was performed over a three day period. Water level loggers were installed in 5 monitoring wells (MW-3A, -3B, -3C, -5A, and -5C) and 2 piezometers (PZ-1 and -2). The piezometers and test wells are fully penetrating wells in the surficial aquifer. The MW-3 series wells are near the APT location and the MW-5 wells were used to monitor ambient water level fluctuations. Water levels were monitored continuously from November 16th until November 20th. The pumping was initiated on November 17th at 9:37 a.m. and flow rate of 32.85 gallons per minute was sustained throughout the three day pumping period. The pumping was concluded at 9:00 a.m. on November 20th and data were then collected for the recovery portion of the APT. Over the three day period, the water level in the pumping well declined 6.48 feet. Hydrographs prepared from the APT data are included in **Appendix C**.

2.4 Investigation Derived Waste

Cuttings from the fly ash borings were contained in drums along with the drilling decontamination solutions. Groundwater derived from monitoring well purging was also placed in drums. The drummed wastes were characterized for disposal purposes and the laboratory reports are included in Appendix B. Based on the laboratory results on the individual fly ash samples discussed above and additional analyses on a composite sample, the wastes were considered to be non-hazardous. The disposal services were provided by PetroChem Recovery Services.

2.5 Additional Data Sources

Several investigations have recently been conducted for the site and include reports prepared for the EPA, Dominion Generation, and the City of Chesapeake. Data from these additional sources include extensive laboratory data for groundwater and surface water, fly ash characterization data, and aquifer hydraulic property data. For data analysis purposes, CDM prepared a comprehensive water quality database from these investigations and the database is included in **Appendix D**. These additional data sources are referenced below.

MACTEC, ***Post-Construction Ash Fill, Soil Cover and Groundwater Evaluation Report, Battlefield Golf Club Ash Reuse Site***, Chesapeake, Virginia, prepared for Dominion Generation, December 17, 2009.

Tetra Tech, ***Final Site Inspection for the Battlefield Golf Club Site***, City of Chesapeake, Virginia, prepared for U.S. EPA Region 3, April 16, 2010.

Tetra Tech, ***Draft Site Inspection for the Battlefield Golf Club Site***, City of Chesapeake, Virginia, prepared for U.S. EPA Region 3, March 30, 2009.

URS, ***Task 7 Off Site Groundwater Investigation***, Battlefield Golf Club Water Project, prepared for the City of Chesapeake, November 5, 2009.

URS, ***Water Supply Feasibility Study***, Battlefield Golf Club Water Project, prepared for the City of Chesapeake, April 10, 2009.

Section 3
Hydrogeologic Investigation Results

The primary focus of the hydrogeologic investigation was to characterize the hydrogeology and the groundwater/surface water quality in the site vicinity to the extent necessary to support conclusions regarding the groundwater flow characteristics and potential constituent fate and transport characteristics. Based on the data collected by CDM and others, a hydrogeologic characterization was completed that described the site-specific geologic layers, provided initial estimates of the hydrogeologic properties, and described the groundwater flow patterns. In addition, CDM assembled and evaluated a water quality database to assess constituents in groundwater and surface water in the site vicinity. This database includes all applicable data from the sources identified in Section 2.5 of this report.

3.1 Site-Specific Geology

The geologic layers investigated at the site include the surficial aquifer, the Yorktown confining zone, and the Yorktown aquifer. **Figure 3-1** includes a boring log summary from monitoring well installation. These boring logs were used to identify the elevations of the geologic layers and provide the layer descriptions. Additional elevation data for these layers on a regional basis were obtained from regional literature (McFarland, 2006).

Surficial Aquifer – In the site vicinity, CDM divided the surficial aquifer into two zones: an upper clay zone that occurred from land surface to depths ranging from 1 to 15 feet bls and a sand zone that was beneath the clay zone. The average thickness of the clay zone was approximately 5.5 feet. Two primary lithologies were identified for this clay. The dominant lithology was a sandy to silty clay that was typically brown to gray. The less frequent lithology of upper zone was black, organic-rich clay. The remainder of the surficial aquifer from the base of the upper clay zone to the top of the Yorktown confining zone consisted primarily of sand, ranged in thickness from 28 to 61 feet, and was approximately 39 feet thick on average. The typical lithology consisted of fine- to medium- to coarse-grained sand. Structure contours drawn on the surface of the surficial aquifer sand zone are shown on **Figure 3-2**.

Yorktown Confining zone – The Yorktown confining zone was found to have variable lithologies, as indicated by the regional data. Most of the Yorktown confining zone consisted of clay with less dominated layers of primarily sand. The depth to the top of the Yorktown confining zone was from 29 to 61 feet bls and averaged approximately 44.5 feet bls. The thickness of this zone ranged from 30 to 51 feet and was approximately 41.5 feet thick on average. The clay in the Yorktown confining zone was typically sandy to silty and at several locations included shell fragments and mica. The sand was typically silty to clayey, fine- to medium- to coarse-grained and also contained shell fragments. Structure contours drawn on the surface of the Yorktown confining zone are shown on **Figure 3-3**.

Yorktown Aquifer – The Yorktown aquifer was found to consist entirely of sand layers that ranged from fine- to coarse-grained and was less typically silty. Shell fragments and mica were also common admixtures. The depth to the top of the Yorktown aquifer was from 76 to 96 feet bls and averaged approximately 88.5 feet bls. The thickness of this zone could not be determined from the 6 borings that reached this zone because these borings did not fully penetrate this zone. Structure contours drawn on the surface of the Yorktown aquifer are shown on **Figure 3-4**.

3.2 Groundwater Flow

Previous mapping of the surficial aquifer potentiometric surface at the site has been performed and all of these maps have indicated a general southeast groundwater flow direction. This flow direction is consistent with the regional expectations and the direction of the Pocaty River. **Table 3-1** includes a summary of groundwater level depths, potentiometric surface elevations, and surface water elevations used to construct potentiometric surface maps for the surficial aquifer and the Yorktown aquifer in the site vicinity.

Figure 3-5 is a potentiometric surface map for the surficial aquifer. To construct this map, CDM used water levels collected on September 15, 2009, from monitoring wells. Estimates of the surface water elevations along the ditch that borders the site to the south and an in the onsite ponds were estimated from average values reported for December 3rd and 10th of 2008 and for July 15, 2009 (MACTEC, 2009). From this figure, the overall groundwater flow direction is east. However, the groundwater flow patterns are influenced by the onsite ponds and the ditch on the south boundary of the site. The North Tributary appears to form the surficial aquifer's local hydraulic base level in the site vicinity with groundwater flowing toward the tributary and eventually discharging to the tributary as surface water.

Figure 3-6 is a potentiometric surface map for Yorktown aquifer. To construct this map, CDM used water levels collected on September 15, 2009, from monitoring wells. Surface water data were not used to construct this map because the deeper Yorktown aquifer is not hydraulically connected with the surface waters. From this figure, the overall groundwater flow direction is east-northeast. Water level elevations also decrease to the northwest toward MW-1C to elevations below sea level. Pumping in this direction is a good possibility and evidence of pumping in this unit was observed during the APT.

3.3 Groundwater and Surface Water Quality

The comprehensive water quality database in Appendix D includes a total of 161 water sample locations. The locations include groundwater from 48 site-specific monitoring wells and 80 residential wells that are located within close proximity to the site. From the 128 groundwater sample locations, over 4,100 analytical results are available. In general these results include metals and general water quality parameters that vary slightly among the samples. In addition to groundwater, 33 surface water locations are included in the database with a total of approximately 1,300 analyses.

3.3.1 Initial Data Screening

Table 3-2 includes summary level statistical data from the database. Normal probability plots of the database are included in **Appendix E**. The probability plots, or quantile plots, include the constituent concentration in micrograms per liter (ug/L) on the y axis and the x axis is the quantile of the distribution. CDM initially evaluated the data using percentile plots but the percentile plots appeared to bias the results toward possible "false positive" conclusions related to assessing potential water quality effects associated with the fly ash. The constituent quantile values were calculated using an Excel workbook application called PPLOT (Chappell, modified 2010). A quantile is a measure of relative standing and a description provided in EPA guidance (EPA, 2000) is provided below.

> "A quantile is similar in concept to a percentile; however, a percentile represents a percentage whereas a quantile represents a fraction. If 'x' is the pth percentile, then at least p% of the values in the data set lie at or below x, and at least (100-p) % of the values lie at or above x, whereas if x is the p/100 quantile of the data, then the fraction p/100 of the data values lie at or below x and the fraction (1-p)/100 of the data values lie at or above x. For example, the .95 quantile has the property that .95 of the observations lie at or below x and .05 of the data lie at or above x."

Non detections are included at the reported detection limits. A steep rise in the quantile plot near the beginning of the line or near the end of the line typically indicates data outliers that are not consistent with the "population" distribution. Normally distributed data approximate a straight line on the normal plots and log-normal distributed data approximate a straight line on the log plots. The appearance of more than one straight line on a quantile plot can indicate multiple "populations" within the dataset or a population that is not normally or log-normally distributed.

Preliminary conclusions based on the summary statistics and the quantile plots are included in Table 3-2. This analysis was used as a screening tool to identify data sets to evaluate in more depth. From this screening analysis, 15 constituents of the 27 constituents were recommended for additional analysis. The 12 constituents excluded from additional analysis had insufficient detections to support additional analysis. Possible high- and low-concentration outliers were identified in the initial analysis and these outliers were removed from the database prior to further analysis as presented in the section below.

3.3.2 Detailed Constituent Analysis

Background constituent concentrations are assumed to present a "population" and based on the initial analysis the distribution are generally log-normal. A higher concentration "population" as compared to background could possibly represent exceedances of background associated with water quality effects from the site. For each constituent identified for further analysis in Table 3-2, additional quantile plots were prepared that segregate the data into one of the following three "populations."

- Baseline Data – These data points are assumed to be the least likely "populations" to be affected by the site and consist primarily of background concentrations. The baseline data were derived from the following locations: residential wells, offsite monitoring wells, upgradient monitoring wells, onsite monitoring wells completed at the base of the surficial aquifer, and offsite surface water samples. Although results from some of these data points are possibly influenced by the site, the influence should be small compared with the two site-related "populations" described below. The effect of including data points that are possibly influenced by the site in the baseline data is to make the analysis conservative toward minimizing false-positive identification of site effects on water quality.

- Onsite Ponds – These data points are assumed to be one of the most likely "populations" to be affected by the site and can be compared to the baseline data to assess possible background exceedances.

- Onsite "A" Wells – These data points consist of onsite monitoring wells completed in the upper-most portion of the surficial aquifer. These wells are assumed to be one of the most likely "populations" to be affected by the site and can be compared to the baseline data to assess possible background exceedances.

The data plots for these three "populations" are included in Appendix E, **Figures E-16** through **E-30**. Additional statistical information for these data sets, which excludes the previously identified outliers and non-detect results, is provided in **Table 3-3**. The statistical information includes the mean, or average, and the 95% confidence intervals of the mean. The bar plots of the confidence intervals include the central mean and the range associated with the confidence interval of the mean. A simple explanation of the confidence interval's significance is that a 95% confidence exists that the average or mean concentration of the subject population is within the confidence interval based on the data supplied. Where the mean value of the onsite pond data or the onsite "A" well data exceed the upper limit (UL) of the 95% confidence interval of the baseline data, a statistical exceedance in concentration of that "population" over the baseline "population" was assumed to exist. These results are further discussed below for each constituent.

3.4 Groundwater and Surface Water Quality Conclusions

Data collected by CDM and the others identified in Section 2.5 of this report were used to assemble the comprehensive water quality database and all of the data were used to complete the data analysis and formulate the following conclusions.

3.4.1 Constituent Concentrations

Aluminum – The mean for the onsite ponds exceeds the baseline UL. The quantile plots on Figure E-16 indicate that the distribution difference between the "A" wells and the baseline is small although the "A" wells quantiles have higher concentrations. Two samples collected from MW-8A were identified as high concentration outliers for aluminum. The onsite pond quantiles are clearly higher in concentration than the baseline on these plots. Aluminum concentrations in the onsite ponds and the onsite "A" wells are possibly influenced by the site.

Ammonia – The mean ammonia concentration for onsite "A" wells exceeds the baseline UL. However, the quantile plots on Figure E-17 indicate that the distribution differences between the "A" wells, the ponds, and the baseline are small beyond the lower quantiles. Ammonia concentrations in the onsite ponds are consistent with the baseline water quality. The onsite "A" wells are possibly influenced by the site based on the ammonia data.

Antimony – Antimony concentrations are assumed to not be affected by the site because of the low number of detections, approximately 17%.

Arsenic – The mean arsenic concentration for both the onsite "A" wells and the onsite ponds is below the baseline UL. The quantile plots on Figure E-18 indicate similar distributions from the onsite ponds and the "A" wells to the baseline. MW-3C was identified as high-concentration outlier for arsenic. MW-3C is not likely affected by the fly ash because it is upgradient and in a lower aquifer. The arsenic concentrations in the onsite ponds and "A" wells are consistent with the baseline water quality.

Barium – The mean barium concentration for the onsite ponds exceeds the baseline UL. The quantile plots on Figure E-19 indicate that the distribution difference between the onsite ponds and the baseline is small beyond the lower quantiles. PW-25 and MW-3A were identified as high-concentration outliers for barium and all are offsite or upgradient wells. Barium concentrations in the onsite "A" wells and onsite ponds are consistent with the baseline water quality.

Beryllium – Beryllium concentrations are assumed to not be affected by the site because of the low number of detections, approximately 26%.

Boron – The mean boron concentration for both the onsite "A" wells and the onsite ponds is below the baseline UL and the quantile plots on Figure E-20 indicate that the distribution difference between the onsite "A" wells, the onsite ponds, and the baseline is small. Boron concentrations in the onsite "A" wells and onsite ponds are consistent with the baseline water quality.

Cadmium – Cadmium is assumed to not be influenced by the site because of the low number of detections, approximately 18%.

Chromium – The mean chromium concentration for the "A" wells exceeds the baseline UL and the onsite ponds mean is below the baseline UL. The quantile plots on Figure E-21 indicate a similar concentration distribution from the onsite "A" wells to the baseline. The concentration differences are small as compared to the confidence interval and these differences are not significant. Chromium concentrations in the onsite "A" wells and onsite ponds are consistent with the baseline water quality.

Cobalt – Cobalt is assumed to not be influenced by the site because of the low number of detections, approximately 39%.

Copper – Copper is assumed to not be influenced by the site because of the low number of detections, approximately 17%.

Iron – The mean iron concentration for the onsite "A" wells exceeds the baseline UL. The mean iron concentration for the onsite ponds wells is below the baseline UL. The quantile plots on Figure E-22 indicate that the distribution for the onsite "A" wells is higher than the baseline and the onsite ponds have a distribution that is below the baseline. Iron in the onsite "A" wells is possibly influenced by the site.

Lead – The mean lead concentration for the onsite "A" wells and the onsite ponds are below the baseline UL. The quantile plots on Figure E-23 indicate that the distribution for the onsite "A" wells is higher than the baseline for the middle quantiles. The onsite ponds have a distribution that is consistent with the baseline. Lead concentrations in the onsite "A" wells and onsite ponds are consistent with the baseline water quality.

Magnesium – The mean magnesium concentration for the onsite "A" wells exceeds the baseline UL. The quantile plots on Figure E-24 indicate that the distribution for the onsite "A" wells is only higher than the baseline in the middle quartiles. The onsite ponds have a distribution that is lower than the baseline. Magnesium in the onsite "A" wells is possibly influenced by the site.

Manganese – The mean manganese concentration for the onsite "A" wells exceeds the baseline UL. The quantile plots on Figure E-25 indicate that the distribution for the onsite "A" wells is higher than the baseline and the onsite ponds have a distribution that is lower than the baseline. Manganese in the onsite "A" wells is possibly influenced by the site.

Mercury – Mercury is assumed to not be affected by the site because of the low number of detections, approximately 17%.

Molybdenum – Molybdenum is assumed to not be affected by the site because of the low number of detections, approximately 20%.

Nickel– The mean nickel concentration for the onsite "A" wells and onsite ponds exceed the baseline UL. The quantile plots on Figure E-26 indicate that the distribution for the onsite "A" wells is higher than the baseline and the onsite ponds have a distribution that is slightly higher than the baseline. MW-5A and -8A were identified as high-concentration outliers for nickel in two samples from each well. However, MW-5A is an offsite well. Nickel concentrations in the onsite "A" wells and the onsite ponds are possibly influenced by the site.

Nitrate – The mean nitrate concentration for the onsite "A" wells exceeds the baseline UL. The quantile plots on Figure E-27 indicate that the onsite "A" wells have insufficient detections for further analysis. The onsite ponds mean nitrate concentration is below the baseline UL. Nitrate concentrations in the onsite "A" wells are possibly influenced by the site.

Nitrite – The mean nitrite concentration for the onsite "A" wells exceeds the baseline UL. The quantile plots on Figure E-28 indicate that the onsite "A" wells have insufficient detections for further analysis. The onsite ponds mean nitrite concentration is below the baseline UL. Nitrite concentrations in the onsite "A" wells are possibly influenced by the site.

Selenium – Selenium is assumed to not be affected by the site because of the low number of detections, approximately 8%.

Silver – Silver is assumed to not be affected by the site because of the low number of detections, approximately 7%.

Sulfate – The mean sulfate concentration for the onsite "A" wells exceeds the baseline UL. The quantile plots on Figure E-29 indicate that the distribution for the onsite "A" wells is higher than the baseline. The onsite ponds mean sulfate concentration is below the baseline UL. Sulfate in the onsite "A" wells is possibly influenced by the site.

Sulfide – Sulfide is assumed to not be affected by the site because of the low number of detections, approximately 2%.

Thallium – Thallium is assumed to not be affected by the site because of the low number of detections, approximately 3%.

Vanadium – Vanadium is assumed to not be affected by the site because of the low number of detections, approximately 39%.

Zinc – The mean for zinc in the onsite "A" wells exceeds the baseline UL. The quantile plots on Figure E-30 indicate that the distribution for the onsite "A" wells is higher than the baseline and the onsite ponds have a distribution that is lower than the baseline. Zinc in the onsite "A" wells is possibly influenced by the site.

3.4.2 Constituent Spatial Distributions

Figures 3-7 and **3-8** include spatial plots of the data for the eight of the constituents concluded to possibly reflect water quality influences associated with the site. Plots were not prepared for nitrate and nitrite because of the low number of detections in onsite groundwater. The plots were prepared by posting the sample collection location with a symbol that is proportionate in size to the sample concentration. From these plots, the constituent spatial concentration distributions can be further assessed to identify potential distribution patterns associated with the site.

For aluminum, ammonia and magnesium, the spatial distribution of high concentrations do not present an obvious site-wide association although higher concentrations do appear near the southwest corner of the site. Constituents that do appear to have higher concentrations on the site include iron, nickel, and zinc. Areas that consistently include the higher concentrations are along the south boundary and eastern portion of the site. Both of these areas are in the direction of groundwater flow from the areas where fly ash was used for fill.

Section 4
Groundwater Model Data Review

Prior to conducting the groundwater modeling and transport analysis described in Sections 6 and 7, CDM performed a review of available data, including, but not limited to, modeling studies prepared for previous reports on the site, unsaturated zone modeling used to generate estimates of leachate production and infiltration, and post-construction water quality data. Regional hydrogeologic reports and models prepared by the United States Geological Survey (USGS) were also reviewed for this study.

The purpose of the data review was to gather information for the groundwater flow and transport modeling, and to critically review assumptions made by others concerning site conditions in the light of post-construction data gathered primarily in 2008 and 2009.

4.1 Modeling Review and Previous Reports

CDM performed a review of the principal reports that have been issued for the Battlefield Golf Course site. These include a study by GAI Consultants, Inc. (GAI) to determine how ammonia associated with the fly ash may impact groundwater in the site vicinity (GAI, 2003). GAI assumed that ammonia would be converted rapidly in the environment into nitrate, which is highly soluble and mobile in groundwater. Total ammonia concentrations in the fly ash were expected to be approximately 5–10 milligrams per kilogram (mg/kg), accounting for volatilization in handling during processing.

GAI used the model BUFFER1, a one-dimensional model to simulate uniform vertical flow, was used to simulate nitrate transport in the unsaturated zone and predict nitrate concentrations at the water table. Retardation was not simulated. Vertical transport was simulated through 5 feet of ash and 5 feet of natural clay above the water table. The model predicted that nitrate-N concentrations would exceed 1 mg/L beneath the golf course site area for a period of 17 years.

Groundwater flow and transport was simulated by GAI using QUICK DOMENICO.xls, a quasi-three dimensional transport model assuming a constant hydraulic gradient. Nitrate concentrations 450 feet away from the source were predicted to be a maximum of 4.2 milligrams per liter (mg/L) nitrate-N, and 3.6 mg/L after a period of 17 years.

Many assumptions and parameter values presented in this report are generally consistent with the modeling described in Sections 6 and 7 of this report. Conclusions of the GAI report regarding offsite contaminant transport were not consistent with the modeling described in this report, because of their assumption that the hydraulic gradient is spatially uniform and a lack of consideration of the impact of the drainage ditch system.

CDM also reviewed the report prepared by URS (2001b). Prior to the construction of the golf course, URS conducted a study to evaluate the leachability of metals from stabilized fly ash and performed modeling to predict concentrations of ash-related constituents in groundwater at the site boundary. This study identified seven chemicals of potential concern. Of these, selenium and arsenic were assessed to be of greatest potential concern. URS used an "Integrated Pathway Model" approach, combining unsaturated zone modeling using EPA's HELP and VLEACH models, combined with groundwater flow and transport in the saturated aquifer using MODFLOW and MT3D. CDM reviewed the HELP model simulations and was able to reasonably recreate the results, as discussed in Section 5.3. URS' modeling resulted in an estimate of 18.9 inches per year of leachate infiltration generated at the site. CDM conducted a similar analysis described in Section 5.2 using updated input parameters that resulted in somewhat lower estimates of infiltration rates.

Assuming that the leachate production rate can be used to approximate the infiltration to shallow groundwater, URS applied the 18.9 inches per year value to the entire 215 acre site, including areas with little or no ash fill and to the pond areas. This assumption generated a total leachate volume approximately twice that estimated by CDM. To simulate migration of the leachate in the saturated zone, URS then performed groundwater flow and transport modeling using MODFLOW and MT3D using a simple one-layer model that does not represent surface water features or groundwater-surface water interaction. Transport parameters used in the URS model assumed less adsorption of arsenic and higher dispersion coefficients than those estimated by CDM.

The data review also included the recent MACTEC report (MACTEC, 2009). MACTEC performed field work and laboratory analyses including groundwater and surface water sampling and water level measurements, soil borings of the ash fill and soil cover. This data was used extensively in CDM's analysis.

The USGS developed a regional groundwater model of the Virginia Coastal Plain area (Heywood, 2009) that provided a reference for off-site hydrological and hydrogeologic conditions used in the groundwater model development for the site described in Section 6. Other USGS reports that provided general background information used in the analyses discussed in this report include Harsh and Laczniak (1990), Laczniak and Meng (1988) and Hamilton and Larson (1988).

4.2 Hydrogeologic Data Review

4.2.1 Aquifer Performance Test Analysis

During November of 2009, CDM conducted an APT at well TW-1, as described in Section 2.3. The APT was performed with a pump capable of pumping approximately 35 gallons per minute (gpm) and was conducted over a three day period. Water level loggers were installed in seven wells (MW-3A, MW-3B, MW-3C, MW-5A, MW-5C, PZ-1, and PZ-2). Wells MW-5A and -5C were monitored for background and are located southwest of the site. **Figure 4-1** shows the locations of the pumped well (TW-

1) and the wells that were monitored. **Table 4-1** shows the well construction details for each of the APT wells installed by CDM. The following figures show the observed water levels that were recorded during the APT.

- MW-3A/MW-3B Area (**Figure 4-2**): A decline in groundwater level was observed in these two wells prior to pumping at TW-1. Following the start of APT pumping, drawdown is observed in both wells. Drawdown is more pronounced in the deeper MW-3B well (42 feet deep) versus the shallower (15 feet deep) MW-3A well.

- PZ-1/PZ-2 Area (**Figure 4-3**): A decline in groundwater level was also observed in these two wells prior to pumping at TW-1. Following the start of APT pumping, drawdown is observed in both wells. Drawdown was more significant at PZ-1 (30 feet from TW-1) than at PZ-2 (60 feet from TW-1). Drawdown was also more pronounced at PZ-1 and PZ-2 than at MW-3A and MW-3B, due to their closer proximity to the pumping well.

- MW-3C (**Figure 4-4**): A small amount of drawdown was observed at MW-3C. This well is screened in the Yorktown aquifer below the surficial aquifer where TW-1 is screened. Due to the relatively small amount of drawdown observed at MW-3C, this well was not analyzed in detail.

- MW-5A/MW-5C Area (**Figure 4-5**): Changes in groundwater level due to the APT were not observed at these two wells located over 4,000 feet from TW-1. The groundwater level at well MW-5A well shows a similar declining trend as observed at MW-3A, MW-3B, PZ-1, and PZ-2, suggesting a change in background hydrologic conditions during the test. The groundwater level at MW-5C appears to be responding to a background stress, possibly from groundwater pumping at a nearby well.

The software program AQTESOLV was used to analyze the results of the APT. AQTESOLV allows for the analysis of multiple types of APTs, including the constant rate test performed at TW-1. AQTESOLV uses the physical layout of the wells (spacing, depth, screened elevations, diameters) and specification of a pumping rate to perform the analysis. This data was based on the information previously shown in Table 4-1. The observed drawdown is also input to AQTESOLV for the software to use to estimate hydraulic properties.

The Hantoush (leaky aquifer) solution technique was used estimate the hydraulic conductivity of the surficial aquifer from the data. Use of a “leaky” type solution was selected to represent the hydraulic impact of semi-confining clay/silt layers above and below the surficial aquifer. **Figure 4-6** shows a conceptual cross-sectional view of the aquifer system as specified in AQTESOLV. The AQTESOLV estimation of horizontal hydraulic conductivity of the Columbia aquifer is shown in **Table 4-2**.

Note that two different estimates of the ratio between horizontal hydraulic conductivity (K_h) and vertical hydraulic conductivity (K_z) were assumed. This assumption had little impact on the analysis results. Additional aquifer conceptualizations (unconfined and confined) as well solution techniques (Theis, Cooper-Jacob) were also analyzed. The results of these solutions generally agreed with the results shown in Table 4-2.

The TW-1 APT analysis indicates that, in general, the horizontal hydraulic conductivity in the area of TW-1 is in the range of 50 to 70 ft/day. This range is consistent with values that are expected for a fine- to coarse-grained sand aquifer. This analysis did not provide sufficient information to assess the ratio between horizontal hydraulic conductivity and vertical hydraulic conductivity with AQTESOLV.

Section 5
Hydrologic Analysis

Specific hydrologic analyses and investigations were conducted to increase the understanding of the site hydrology and water budget and help quantify the model parameters and calibration targets for the groundwater flow modeling presented in Section 6. These analyses included:

- Review of stream flow data and hydrologic reports to help estimate the average groundwater recharge rate in the model area;
- Investigation of the influence of drainage infrastructure on regional and local groundwater flow;
- Investigation of data indicating the possible influence of the onsite golf course ponds on the shallow groundwater system;
- Numerical simulations to estimate infiltration and leachate production rates from the ash fill for development of source terms for groundwater transport modeling (Section 5.2);
- Critical review of leachate production rate estimates for the fill areas generated by HELP model simulations performed by URS (2001b); and
- Generation of an updated estimate of the likely range of average infiltration and leachate production rates using HELP model simulations with revised input parameters.

5.1 Recharge, Drainage and Ponds

Interaction between groundwater and surface water is an important feature of both local and regional hydrology. Available stream flow data was reviewed to help estimate regional average groundwater recharge in the study area. The regional USGS data for southeastern Virginia (**Figure 5-1**) show that average measured stream flow typical of the area is in the range of 0.7 to 1.1 cubic feet per second per square mile (cfsm), equivalent to 9.5 to 14.9 inches/year (in/yr). This represents a combined total of both groundwater-derived base flow and direct runoff. In stable systems base flow and recharge can be assumed to be approximately equivalent. Using base flow separation techniques, average net recharge for the Virginia coastal plain as a whole is estimated to be approximately 10 in/yr based on analysis of measured stream flow (Heywood and Pope, 2009). Actual recharge is spatially variable depending on soil properties, land slope, drainage, and land use.

The Chesapeake area is characterized by low-lying swamp lands and a high water table, and the land in the vicinity of the site is primarily used for agriculture. An extensive network of drainage ditches is plainly evident from inspection of aerial photographs, and major ditches are included in maps illustrating surface water features. Throughout the region, drainage facilities are extensively employed to manage water levels and prevent high groundwater conditions from adversely impacting agriculture and other land uses. Groundwater is discharged into these drains and this water is then conveyed by ditches to downstream courses.

A network of staff gauges has been installed in ditches on and nearby the site area. In addition, the USGS maintains a regional network of staff gauges and associated data can be accessed on-line (http://waterdata.usgs.gov/nwis).

Local influence of the ditches on groundwater flow near the site is evident from the water level data measured in the ditches and nearby groundwater monitoring wells. Hydraulic gradients toward the ditch are noted both horizontally and vertically in the observed water level data. Water levels in monitoring wells immediately adjacent to the drainage ditch to the south of the site are consistent with readings from staff gauges installed in the ditch indicating hydraulic communication between the groundwater and the surface water in the ditch.

A number of ponds were constructed onsite as golf course features by excavating in the unsaturated zone and into the saturated zone. Water level data from staff gauges sited in onsite ponds were also qualitatively examined in the context of water levels measured in nearby "A" wells. Although the MACTEC report indicated that the depths of the ponds were measured, tabulated measurement data appear to be absent; however, two measured depths are noted on cross-section figures 4 and 5 (MACTEC, 2009). These figures indicated that Pond SG-12 is 10.7 feet deep, and Pond SG-9 is 18.9 feet deep. Pond SG9 is connected to Pond SG-10 via a short canal, and thus the depth is assumed to be similar in Pond SG-10. Pond depths are not drawn to scale in the MACTEC cross-section figures. These cross-sections also suggest that Ponds SG-8 and -3 are considerably shallower (depth data are not posted), although they are also illustrated as having depths extending below the bottom of the silt/clay layer and in direct hydraulic communication with the more permeable sand zone in the surficial aquifer. Water level fluctuations within the ponds were evaluated for consistency with neighboring pond behavior (and, where available, data from shallow groundwater wells). The size of the ponds was also considered, with the larger ponds assumed to possibly be deeper than smaller ponds.

Based on this analysis, CDM concluded that the following ponds are likely to have moderate to good hydraulic connection with surficial aquifer: ponds SG-3, -9, -10, -11, -12, -16, and -17. Conversely, the following ponds are likely to have a more limited hydraulic connection with surficial aquifer: ponds SG-1, -2, -19, -6, -7, and -8. Ponds SG-3 and -16 have drainage ditches that lead from the ponds to the main site drainage ditch. The rise of the water surface level of these ponds is thus limited by an outlet structure. The water levels in these ponds varied by less than 0.2 feet, except during a dry period in July 2009, when the pond levels had receded.

5.2 Leachate Production Rates (HELP Model)

CDM was tasked with evaluating a project-specific, Integrated Pathway Model (URS, 2001b). A baseline component of the input data for the Integrated Pathway Model was a HELP Model simulation of the site resulting in an estimate of infiltration rates through the ash fill in the unsaturated zone. The HELP model is a quasi-two dimensional, deterministic model (Schroeder, 1994) developed by the EPA to help landfill designers estimate the magnitudes of components of a landfill's water budget and the amount of leachate produced by the landfill. The HELP model determines runoff, evapotranspiration, percolation, and lateral drainage to obtain water budgets. CDM performed a review of the HELP model simulation conducted by URS for the Integrated Pathway Model.

Table 3.4 in Section 3.2.1 of the URS report documenting the Integrated Pathway Model (URS, 2001b) listed the assumptions used for the model input data. Three layers were simulated at the Battlefield Golf Course site: Layer 1 represents the soil cover; Layer 2 represents the fly ash fill; and Layer 3 represents the underlying silt/clay layer. As a starting point, CDM attempted to recreate the URS HELP model run and simulate the results reported. Several inconsistencies were noted between input files included in the report appendix and tabulated data in the body of the report, including the depth of the ash fill, and the hydraulic conductivity value used for the silt/clay layer at the base of the fill. Using the corrected values gleaned from the model output files in the report appendix, CDM was able to recreate the URS HELP model simulation and obtain the results reported by URS of 18.8 in/yr of recharge. This was the value used by URS as an initial estimate of the local rate of recharge entering the groundwater flow model.

CDM conducted several more simulations to evaluate the sensitivity of the model to various assumptions. These included:

- Omitting the representation of a low-hydraulic conductivity silt/clay layer (Layer 3) at the base of the ash fill. This simulation yielded an infiltration estimate of 18.84 in/yr, indicating that the HELP model estimate of infiltration is insensitive to hydraulic conductivity assignments of Layer 3. In the URS HELP model simulations (URS, 2001b), the silt/clay layer (Layer 3) underlying the ash fill was assigned a value of 8.2E-07 centimeters per second (cm/s), or 0.0023 ft/d, based on data from boring B1B.

- A review of fly ash samples (URS, 2001b, and MACTEC, 2009) yielded a geometric mean saturated hydraulic conductivity value of 6.4E-06 cm/s. Using this site-specific value, a 20.3 in/yr rate of recharge was calculated by the HELP Model.

- Increasing the simulated thickness of the soil cover on the landfill (Layer 1) from 6 inches to 18 inches to improve model agreement with soil boring data reported by MACTEC (2009) yielded an annual infiltration rate of 19.9 in/yr.

- Increasing both the simulated thickness of the soil cover (Layer 1) and the evaporative zone depth from 6 to 10 inches yielded an annual mean infiltration rate of 15.8 in/yr. Increasing the evaporative zone depth from 6 to 18 inches yielded a rate of 12.7 in/yr. The recommended range of evaporative depths provided in HELP model documentation is approximately 10 to 42 inches (Schroeder, et. al., 1994) for southeastern Virginia. It was assumed that at the site, the root zone would not exceed the depth of the soil cover.

- The top cover material applied above the ash fill was described by MACTEC (2009) as "brown to dark brown and gray, stiff to firm, clay and silt soils." A sensitivity simulation, whereby the hydraulic conductivity value originally cited in the URS report (2001b) of 8.2E-07 cm/s for onsite silt/clays tested from boring B1B (URS, 2001b) was applied to the soil cover (Layer 1), and the simulated soil cover thickness and evaporative zone depths were both set at 18 inches, yielded an annual mean infiltration rate of 7.15 in/yr.

- Applying the hydraulic conductivity value used in the groundwater model layer representing the silt/clay, 0.05 ft/d (1.76E-5 cm/s) to the soil cover (Layer 1) generated an infiltration rate of 7.55 in/yr.

The HELP model simulations did not appear to be sensitive to adjustments in parameters describing the silt/clay layer underlying the ash fill (Layer 3), but were found to be sensitive to the hydraulic conductivity, depth, and evaporative depth assignments in the soil cover (Layer 1), all of which reduced model estimates of infiltration through the landfill areas. As a result of this analysis, the estimated infiltration through the emplaced fly ash at the golf course is reasonably expected in the range of approximately 7.5 – 15.8 in/yr. The estimate of infiltration (leachate production) through the unsaturated zone in areas of ash fill was an important parameter in calculating mass loading rates for the transport model, as described in Section 7.2.1.

Section 6
Groundwater Flow Model

A groundwater flow model was developed to evaluate groundwater flow patterns in the site area and to provide a basis for contaminant transport modeling.

6.1 Model Code

The DYNFLOW modeling code was used to develop the project groundwater flow model. The flexibility of DYNFLOW's finite element structure makes it easy to conform the model geometry to streams, ditches, ponds and other hydrologic features. The DYNSYSTEM groundwater modeling software includes DYNFLOW (single-phase groundwater flow), and DYNTRACK (solute transport). DYNFLOW is a fully three-dimensional, finite element groundwater flow model. This model has been developed over the past 25 years by CDM engineering staff, and is in general use for large scale basin modeling projects and site specific remedial design investigations. It has been applied to over 200 ground water modeling studies in the United States and has been reviewed and tested by the International Ground Water Modeling Center (IGWMC) (van der Heijde 1985, 2000). The code has been extensively tested and documented by CDM and is commercially available for purchase.

DYNFLOW accepts various types of boundary conditions on the groundwater flow system including:

- Specified head boundaries (where the piezometric head is known, such as at rivers, lakes, ocean, or other points of known head)
- Specified flux boundaries (such as rainfall infiltration, well pumping, and no-flow "streamline" boundaries)
- Rising water boundaries; these are hybrid boundaries (specified head or specified flux boundary) depending on the system status at any given time. Generally used at the ground surface to simulate streams, wetlands, and other areas of ground water discharge.
- Head-dependent flux (3rd type) boundaries including "River," "Drain," and "General Head" boundary conditions. Third-type boundaries can be used to represent drainage to local streams or surface water bodies if the piezometric head in a phreatic aquifer rises to the elevation of topmost model level, representing the streambed or land surface. Rising water fluxes at the conditional model boundary at the land surface elevation represent discharges of groundwater to surface water.

DYNFLOW uses a finite-element grid mesh built with a large number of tetrahedral elements. These elements are triangular in plan view, and give a wide flexibility in grid variation over the area of study. An identical grid is used for each level (surface) of the model, but the thickness of each model layer (the vertical distance between levels in the model) may vary at each point in the grid. Linear interpolation functions are applied in hydraulic computations within each element.

DYNFLOW can treat phreatic (unconfined), confined or mixed conditions, with the phreatic surface at each plan view node location occurring in any model layer, or moving between layers in a transient case. As such, model layers are not explicitly classified as “confined,” “unconfined,” etc. The phreatic surface defines the current model upper limit, and adjustments to the model grid geometry are made accordingly.

DYNFLOW is the core of an integrated set of modeling codes (DYNSYSTEM) that can simulate solute transport, non-aqueous phase liquid (NAPL) flow and density-driven aqueous-phase flow such as seawater intrusion. A graphical user interface, DYNPLOT, provides model building capabilities and rapid graphical displays of model inputs, simulation results, field data, and physical and geographical features.

6.2 Model Domain and Computational Grid

The model domain and computational grid are shown on **Figure 6-1**. The model domain has been extended to natural hydrologic boundaries at a considerable distance, 2 to 14 miles, from the site so that simulated groundwater flow near the site is not constrained by assumed model boundary conditions. The model extends to the Intracoastal Waterway to the north, the North Landing River and Currituck Sound to the east, Northwest River to the south and southwest, and to swampland and unnamed tributaries to the Northwest River to the west.

The finite element grid is comprised in plan view of 15,620 triangular elements defined by 7,932 node points at the vertices of the triangles. Aquifer and confining unit hydraulic properties are specified by element and layer. Fluxes, piezometric heads and layer top and bottom elevations are specified or computed at nodes and levels (layer top and bottom boundaries). Nodal spacing ranges from approximately 80 feet on site to 2,000 feet near the model boundaries. Nodal spacing was further refined to 15 feet in the vicinity of APT well, TW-1, for the purpose of simulating the aquifer performance test conducted by CDM in November 2009. The computational grid in the site area is shown in **Figure 6-2**.

6.3 Hydrogeologic Layers and Properties

The model includes the surficial aquifer, the Yorktown aquifer and the Yorktown confining zone that overlies the Yorktown aquifer and underlies the surficial aquifer. The Yorktown aquifer, the bottom layer of the model, is underlain by the St. Mary’s confining unit. The St. Mary’s confining unit is a low permeability layer with a thickness greater than 500 feet in the Battlefield USGS model domain (Heywood and Pope, 2009). Hence, hydraulic interaction between the Yorktown aquifer and deeper aquifers is insignificant for the purpose of this study.

In addition, the model explicitly incorporates an approximately five-foot thick layer of relatively low permeability silt and clay at the unimproved land surface at and near the site, as identified in the soil borings. The surficial aquifer is subdivided into four computational model layers to better represent the vertical component of flow and transport in that aquifer. Model layers are numbered from bottom to top in DYNFLOW. The model layers are summarized in **Table 6-1**.

The model layering is illustrated on **Figures 6-3** and **6-4**, which are cross-section plots showing model layering with the boring logs superimposed. Cross-section A-A′ shown in Figure 6-3 is an east-west cross-section along the southern perimeter of the golf course. Cross-section B-B′ on Figure 6-4 is a north-south cross-section, approximately through the middle of the golf course.

The top of the model represents the land surface. The distribution of land surface elevations was taken from the National Elevation Dataset (USGS) except at the Battlefield golf course and nearby drainage ditches. The land surface at the golf course, shown in **Figure 6-5**, was assigned based on design contours for the site (MJM_Golf_Documents), since as-built topography was not available. The elevations along the drainage ditch immediately west and south of the golf course, were assigned based on interpolation of available staff gage data as described in Section 6.4.3.

The top of the surficial aquifer is represented by the land surface, except in the vicinity of the site where a surficial silt-clay layer is explicitly represented. The top of the computational model is automatically located at the water table in DYNFLOW. The elevation of the bottom of the surficial silt-clay layer (and top of the surficial aquifer) was interpolated from soil boring logs, as illustrated in cross-section Figures 6-3 and 6-4. The spatial distributions of (1) the bottom elevation of the surficial aquifer, (2) the Yorktown confining zone, and (3) the Yorktown aquifer were assigned to the model based on interpolation of data from site and regional borings. These elevation distributions are shown in **Figures 6-6** through **6-8**. The bottom of the model domain is defined by the bottom of the Yorktown aquifer.

Table 6-2 lists the model hydraulic property assignments. These assignments were based primarily on model calibration, as described below in Section 6.5. The K_h and K_v, specific storativity (S_s) and specific yield (S_y) shown in Table 6-2 are within the expected range of values for these hydrogeologic units presented by Heywood and Pope (2009).

A range of hydraulic conductivity values is shown for some of the stratigraphic units. The lower value is based on the APT calibration, as described in Section 6.5.1. The higher value is based on an alternative model developed during the model calibration and sensitivity analysis as described in Section 6.5.2 that represents higher groundwater flow rates in the aquifer system. Note that in the APT calibration, different hydraulic conductivity values were assigned to the upper half and lower half of the surficial aquifer.

The S_s and specific yield S_y values were taken from the USGS Coastal Virginia regional SEAWAT groundwater model. These parameters do not affect the steady-state simulations used for the transport modeling or steady-state calibration. The transient aquifer performance test simulation was somewhat sensitive to the S_s value assigned to the surficial aquifer.

6.4 Boundary Conditions

Boundary conditions were specified for the model perimeter, model top and model bottom. These boundary conditions include rivers and streams, drainage ditches, onsite ponds, recharge, evapotranspiration, and groundwater withdrawals.

6.4.1 Model Perimeter

Discharge to a river or stream is represented along almost the entire model perimeter in the surficial aquifer, thus providing a natural boundary condition. A specified fixed head boundary condition was assigned to model perimeter nodes in the Yorktown aquifer (layer 1). The specified head values were interpolated from the initial Yorktown aquifer heads assigned in the USGS Coastal Virginia regional SEAWAT model.

A no-flow boundary condition is applied to the bottom of the model. As noted above, vertical flow between the Yorktown aquifer and the underlying St. Mary's confining unit is assumed to be very small compared with the flow in the Yorktown aquifer.

The top of the model, computationally, is the water table. A drain boundary condition, or conditional rising water boundary condition described in Section 6.4.2, was assigned to the top of the model. The computed water table level is free to rise and fall depending on hydrologic and hydraulic conditions, except that it is constrained to not rise above the land surface. Recharge and evapotranspiration fluxes are applied at the water table as described below.

6.4.2 Rivers and Streams

Groundwater discharge to rivers and streams is represented using conditional "rising water" boundary conditions. A rising water node is a "free" node, with specified recharge or discharge and computed head, unless the computed water table tends to rise to or above the land surface. In that case, a specified head boundary condition is invoked with the head fixed to the land surface elevation and discharge from groundwater to surface water is automatically computed. In this way, groundwater discharge is automatically simulated at low points in the topography coincident with streams or wetlands. This is analogous to a drain boundary condition with the drain level set at the land surface with negligible hydraulic resistance between the groundwater and surface water. When assigning land surface elevations to nodes, care was taken to ensure that local low points in the DEM topography were incorporated into the model land surface elevation assignments. Because of a generally shallow water table within the model domain, no significant outflow from streams to groundwater is expected.

6.4.3 Agricultural Drainage

The land use of more than half of the model area is agricultural. This can be seen on Figure 6-9, which shows land use within the model domain based on the National Land Cover Database (NLCD) land cover/use maps for 2001, downloaded from http://www.mrlc.gov/. A dense surface drain network in the agricultural areas can be seen in aerial photographic images. Assuming that surface and sub-surface drainage systems have been constructed in the agricultural land, a drain boundary condition was assigned to all nodes within agricultural areas shown in Figure 6-9. Due to a lack of available design/construction data for the agricultural drainage network, the drain elevation was set to be 3 feet below land surface. A high conductance value (50,000 square ft/d) was assigned, resulting in little computed head loss (hydraulic resistance) between the groundwater and drain.

Special attention was focused on the representation of significant drainage ditches near the golf course. These are shown in dark blue in Figure 6-10. In particular, the drain that runs along the western and southern perimeter of the golf course significantly affects simulated groundwater flow from the golf course. These ditches were represented using the drain boundary condition. Model nodes were specifically placed along the alignment of these ditches. The surface water elevation assignments for the ditch that runs along the western, southern and eastern golf course perimeter were based on the available surface water staff gage data. The elevations along the drainage ditch approximately 3,200 feet south of the golf course, were estimated based on available data at a single staff gage (SG-15), DEM land surface elevations, and the elevations of the drain along the south perimeter of the golf course.

6.4.4 Onsite/Golf Course Ponds

The locations and identifiers of ponds on the golf course are shown on Figure 6-11. Following the convention of MACTEC (2009), the ponds are identified by the number of the staff gage installed for a given pond.

Two of these ponds, SG-3 and -16, discharge to surface drainage ditches and are assumed to behave essentially as groundwater drains. The range of measured staff gage water level readings for these ponds is less than 0.5 feet and 0.8 feet for SG-3 and -16, respectively. Drain boundary conditions were assigned to all nodes associated with these ponds, with the drain water level assigned equal to the average of measured staff gage water level values for these ponds, except for ponds SG-3 and -16, a drain boundary condition was not assigned to the pond nodes.

Ponds SG-9, -10 and -12 are indicated by MACTEC (2009) to be deep enough that the pond bottoms are in direct contact with the surficial aquifer with no intervening silt-clay layer. Although depth data is not available for ponds SG-3, -11, -16 and -17, they were also assumed to be hydraulically well connected with the surficial aquifer based on the close similarity of measured pond levels to heads measured in nearby monitoring wells. For model elements associated with these ponds, the relatively low hydraulic conductivity associated with the surficial silt-clay layer was not assigned to the top model layer. Instead, a very high horizontal hydraulic conductivity of 1,000 ft/d was assigned to account for the negligible resistance to flow within the pond. This results in a relatively flat simulated water table corresponding to the pond surface. The assignment of the 1,000 ft/d "pond" hydraulic property set can be seen in cross-section Figure 6-4.

6.4.5 Recharge and Evapotranspiration

A specified groundwater recharge flux was applied at the water table. Conceptually, groundwater recharge is the remaining precipitation after subtracting runoff and evapotranspiration from the vadose zone, land surface and vegetation surface. Infiltration from irrigation return flow, septic tanks and leaking water pipes can also contribute to groundwater recharge.

For the Virginia coastal plain as a whole, average net recharge is estimated to be approximately 10 in/year based on analysis of measured stream flow using base flow separation techniques (Heywood and Pope, 2009). A HELP model analysis conducted for this study described in Section 5.2 indicated a range of recharge rate, depending on surface soil conditions, of 7.5 to 15.8 in/yr for the site.

As described in Section 6.5, two alternative groundwater models were developed. One model incorporates aquifer hydraulic properties resulting from analysis of the APT conducted by CDM in 2009. Based on model calibration using this set of hydraulic properties, an average recharge rate of 3.1 in/yr is specified for the entire model domain. Since this recharge rate is lower than expected, a second model was developed that incorporates higher values of hydraulic conductivity and recharge. Based on model calibration using this set of hydraulic properties, an average recharge rate of 10.1 in/yr is specified for the entire model domain, except that a recharge rate of 16 in/yr is assigned to the Battlefield golf course area based on the upper limit recharge rate estimated by HELP model analysis.

For the onsite pond areas, a net recharge of 22 in/yr was specified, which is simply the difference between average precipitation of 46 in/yr, multiplied by 1.2 to account for runoff into the ponds from surrounding areas, and an average evaporation of 32 in/yr. Because the pond areas are limited, the model simulations were not very sensitive to the pond recharge assignment.

Where the water table is sufficiently close to the land surface, an evapotranspiration flux may be subtracted directly from the water table. Evapotranspiration from the water table can be significant in this region of Virginia, because there is a relatively shallow water table at many locations. In the model, evapotranspiration from the water table is computed as a function of the depth of the water table below land surface. With the water table at the land surface, computed evapotranspiration is at the specified maximum value of 32 in/yr, based on studies conducted by the Virginia State Climatology Office, as reported by Heywood and Pope (2009). The computed evapotranspiration decreases linearly with depth of the water table below the land surface to a value of zero at a specified root extinction depth.

Extinction depth is a function of crop or vegetative cover, and also soil type and land use. A uniform extinction depth of 3 feet was assigned to the entire model area. However, the evapotranspiration computations are not invoked at the agriculture land use nodes in the model. This is because the drain boundary conditions assigned to agricultural area nodes prevent the water table from rising to an elevation less than 3 feet below the land surface. In effect, the drainage system is assumed to prevent groundwater from saturating the root zone of the crops.

6.4.6 Groundwater Withdrawals

Approximately 63 known residential wells in the site vicinity are used for water supply. The well depths for 17 if these wells are known and the wells were assigned to these depths in the groundwater flow model. The remaining 46 residential wells that do not have available well depth data were simulated as pumping from the surficial aquifer as a conservative measure. All residential wells were assumed to pump continuously at the average residential water usage rate of 0.45 gpm based on typical City of Chesapeake water use rates. This non-intensive, dispersed pumping exerts a negligible overall effect on the groundwater flow field in the vicinity of the site, which is dominated by recharge and discharge to drainage ditches.

The USGS eastern Virginia regional model includes no municipal or industrial pumping from the surficial aquifer within the model domain. The USGS model includes two wells pumping a total of 180 gpm from the Yorktown-Eastover aquifer within the project groundwater model domain. This Yorktown aquifer pumping is assigned to the project groundwater model in the same location as assigned in the USGS model shown on Figure 6-12.

6.5 Flow Model Calibration and Sensitivity

The flow model was calibrated using:

- The results of the Columbia aquifer performance test conducted by CDM in 2009; and
- Comparison of the average measured heads in monitoring wells to model computed heads for steady state simulations representing average hydrologic conditions.

6.5.1 Aquifer Performance Test Transient Calibration

CDM conducted an APT in November 2009 to help define appropriate hydraulic parameters of the surficial aquifer. Discussion of the APT analysis is detailed in Section 4.2.1. Groundwater potentiometric surface response to the pumping monitored in wells MW-3A and MW-3B and piezometers PZ-1 and PZ-2 was analyzed. The relative location of these wells is shown on Figure 6-13. TW-1 was designed to nearly fully penetrate the entire thickness of the surficial aquifer. Well MW-3A monitors the upper surficial aquifer; Well MW-3B, PZ-1 and PZ-2 monitor the lower surficial aquifer. The distances of the monitoring wells from the test pumping well are listed in Table 4-2.

A traditional analysis of the aquifer performance test results using type-curve fitting analytical methods was performed. The computations and curve fitting were done using the AQTESOLV program as described in Section 4.2.1. The results using the Hantush leaky aquifer solution are summarized in Table 4-2. They indicate a surficial aquifer K_h in the 50 to 80 ft/day range. The analysis results were not sensitive to the assumed vertical hydraulic conductivity in the surficial aquifer.

A more comprehensive analysis of the APT results was conducted using the numerical groundwater flow model. The numerical model is not as limited to idealized conditions as the analytical models are. In particular, the numerical model explicitly accounts for vertical flow and gradients and interactions with overlying and underlying layers.

Numerous trial transient simulations using different hydraulic parameters were made with the objective of achieving reasonable agreement between simulated and measured drawdown patterns. Figure 6-14 shows measured and simulated time-drawdown plots at the key monitoring wells for this aquifer performance test. The distribution of simulated drawdown at the end of the pumping period is shown in Figure 6-15. The hydraulic properties listed in Table 6-2 (lower value of ranges) were applied in this simulation.

As indicated in Table 6-2, a relatively lower K_h was applied to the upper half of the surficial aquifer to achieve this result. The APT simulation was sensitive primarily to K_h and K_v of the surficial aquifer, and secondarily to S_s of the surficial aquifer, K_v of the upper silt-clay layer, and K_h/K_v) of the Yorktown confining zone.

6.5.2 Steady State Calibration

The steady state calibration was initially conducted using the surficial aquifer, Yorktown confining zone, and surficial silt-clay layer properties resulting from the APT calibration. Recharge, evapotranspiration, and Yorktown aquifer K_h and K_v were adjusted to provide reasonable agreement between simulated and measured head. The hydraulic properties listed above in Table 6-2 (lower value of ranges) were applied in this simulation. The calibrated recharge rate was 3.1 in/year. This model is called the "Aquifer Performance Test Model" because it incorporates the hydraulic properties developed by the APT calibration.

The calibration results are summarized in **Table 6-3**. Overall, the mean difference between simulated and measured head (termed the residual) was 0.048 feet, with a standard deviation of 1.721 feet. The spatial distribution of simulated head near the site and calibration residuals is shown in Figures 6-16 through 6-18. Figure 6-16 shows simulated head contours near the top of the surficial aquifer, along with color coded symbols at monitoring wells screened in the corresponding vertical interval (A wells) indicating the calibration residual at that location. Figure 6-17 shows simulated head contours and residuals near the bottom of the surficial aquifer (B wells). Figure 6-18 shows simulated head contours and residuals near the top of the Yorktown aquifer (C wells). **Table 6-3** lists the calibration monitoring wells with the average measured head. The steady state calibration target was based on heads measured in monitoring wells on and near the site during 2008 and 2009.

6.5.3 Model Sensitivity

The steady state flow model calibration is very sensitive to the recharge and K values specified. However, the steady state calibration was not unique, in that a similar distribution of simulated head could be achieved using a higher recharge rate in combination with higher K values. Calibration using the APT calibration hydraulic properties required assignment of a lower than expected recharge rate. Because the extent of the APT analysis was limited to a small area of the aquifer within 300 feet of well TW-1, the K values indicated by the APT may not be fully representative of the aquifer as a whole.

Therefore, an alternative model was developed and calibrated (steady state) in which higher recharge rates and higher K values were assigned. For this alternative model, the K_h values in the surficial and Yorktown aquifer were assigned to be 100 ft/d, at the upper limit of the reasonable range of published values for these aquifers presented by the USGS (Heywood and Pope, 2009). The upper value of parameter ranges shown in Table 6-2 were used, except for within approximately 700-1000 feet of APT TW-1 where K values developed for the APT model were assigned. This model is called the "High Flow Model" because the simulated rate of recharge and flow in the aquifers is greater than for the APT Model.

The High Flow Model incorporates a recharge rate in the expected range and resulted in better calibration statistics than the aquifer performance test calibration model. It also provides a basis for more conservative contaminant transport simulations, because higher groundwater flow rates and velocities are simulated. Also, because the higher recharge and higher K_h in the Yorktown aquifer will induce higher flows in that aquifer, the potential for downward flow (and transport) to the Yorktown aquifer is increased.

The calibrated recharge rate for this model was 10.1 in/yr. The calibration results are summarized in **Table 6-4** for the High Flow Model. Overall, the mean difference between simulated and measured head was 0.139 feet, with a standard deviation of 1.158 feet.

The spatial distribution of simulated head near the site for the calibrated steady-state High Flow Model and calibration residuals is shown in Figures 6-19 through 6-21. Figure 6-19 shows simulated head contours near the top of the surficial aquifer, along with color coded symbols at monitoring wells (A wells) indicating the calibration residual at that location. Figure 6-20 shows simulated head contours and residuals near the bottom of the surficial aquifer (B wells). Figure 6-21 shows simulated head contours and residuals near the top of the Yorktown aquifer (C wells).

6.6 Simulated Groundwater Flow Field and Water Budget

Figure 6-22 shows simulated upper surficial aquifer flow direction arrows and head contours (High Flow Model). The flow simulation results indicate that flow in the upper surficial aquifer from beneath the site and surrounding area converges toward the drainage ditch that runs along the south perimeter of the site.

An east-west cross-section A-A′ along the south perimeter of the site is shown in Figure 6-23. Simulated head contours in the surficial aquifer for the High Flow Model are shown, along with average measured head posted at monitoring well locations. A slight upward gradient is indicated by both the measured heads and simulated contours. Simulated head contours and average measured head values are shown for north-south cross-section B-B′ in Figure 6-24. On this figure, a mix of upward and downward head gradients in the surficial aquifer beneath the site are indicated.

The overall water budgets for the APT Model and High Flow Model steady state calibration simulations are summarized in Table 6-5. Positive values indicate flux into the groundwater model domain, negative fluxes indicate flux discharging from the groundwater model domain. The greater simulated flow rates in the High Flow Model are evident in Table 6-5.

Section 7
Transport Model

Groundwater transport model simulations were performed to investigate the potential for constituents from the fly ash at the site to migrate in groundwater and impact downgradient receptors, either in the short term, or over a period of up to 200 years in the future.

The simulated steady-state groundwater flow fields developed as described in Section 6.5 were used to project potential future constituent transport in groundwater resulting from the site. The transport model was not calibrated because the currently available groundwater quality data does not provide a basis for calibration. The lack of calibration based on field observation of an established plume increases the range of the most-probable results for the transport simulations in groundwater. For the simulations, a reasonable range of source loading rates and transport properties was simulated based on data available prior to golf course construction (URS, 2001b) and site-specific post-construction data (MACTEC, 2009), literature, and past experience.

CDM considered the ten constituents identified in Section 3 as being potentially above the baseline in selecting the constituents for transport purposes. In addition to these ten constituents, CDM also considered arsenic (**Table 7-1**). The criteria that CDM considered in selecting the constituents for transport include their presence in the fly ash, leachability tests performed on the fly ash, available regulatory standards/criteria, and mobility.

Nitrate was selected for transport modeling because it is currently above the baseline concentrations in shallow onsite groundwater; it has a Federal Maximum Contaminant Level (MCL), and is present in the fly ash. Because nitrate is highly mobile, it will be present in groundwater in a relatively short period of time following fly ash emplacement. However, this high mobility will also cause nitrate to be depleted from the fly ash in a relatively short time frame. As a result, a constituent with lower mobility was considered to represent constituents that will continue to be leached for longer periods of time into the future. Arsenic was selected for long-term leaching scenario because it has a lower mobility; it has a low MCL, and is present in the fly ash and leachate samples. Arsenic is a high toxicity metal of great concern as a drinking water constituent.

The recent upper surficial aquifer A well data yielded 21% of samples with arsenic concentrations above the MCL of 10 ug/L. It is noteworthy that background arsenic concentrations can be greater than the MCL in ambient groundwater. Furthermore, arsenic was found in MACTEC's fly ash analyses, and in TCLP results on stabilized ash samples as used in the fill (URS, 2001b).

Groundwater transport of nitrate was also simulated. Nitrate transport in groundwater is not subject to significant adsorption/retardation. In the transport simulations, it therefore functions as a conservative tracer of flow pathways from the beneath the footprint of the fly ash into shallow groundwater. If generated in sufficient quantity from the ash, nitrate may also function as a tracer in the field.

7.1 Model Code

DYNTRACK is the companion solute transport code to DYNFLOW. DYNTRACK has been developed over the past 20 years by CDM engineering staff. DYNTRACK has also been reviewed and tested by the IGWMC (van der Heijde 1985). It has been accepted by the US EPA for use, and has been used in several litigation cases.

DYNTRACK is a fully three-dimensional particle tracking and solute transport code. In simple particle-tracking mode, DYNTRACK simulates the mean advective flow path of dissolved solutes using 3-dimensional flow fields developed by DYNFLOW. In full transport mode, the code simulates the advection, dispersion, adsorption, and decay processes controlling solute transport in groundwater.

DYNTRACK uses a Lagrangian approach to approximate the solution of the partial differential equation of transport. This process uses a random walk method to track a statistically significant number of particles, wherein each particle is advected with the mean velocity within a grid element and then randomly dispersed according to specified dispersion parameters.

In DYNTRACK, a solute source can be represented as an instantaneous input of solute mass (represented by a fixed number of particles), as a continuous source on which particles are input at a constant rate, or as a specified concentration at a node. The concentration within a particular zone of interest is represented by the total number of particles that are present within the zone multiplied by their associated solute mass, divided by the volume of water within the zone. DYNTRACK also has the capability to simulate first order decay, nonlinear equilibrium sorption and non-equilibrium sorption (or kinetics).

7.2 Input Parameters

7.2.1 Source Representation

CDM's field investigation attempted to collect samples from leachate wells completed within the fly ash to allow direct laboratory measurements of constituents in leachate to represent the source water quality. However, insufficient leachate was found in the fly ash at the three boring locations where the leachate wells were planned for installation. CDM assumes that the water levels in the fly ash were low during the investigation because insufficient infiltration had accumulated in the fly ash since the fly ash was emplaced. As a result, the source representation required use of the available leaching data for the fly ash and simulations of leaching based on geochemical data.

Figure 7-1 shows the estimated area of fly ash, inferred by CDM using an incomplete map of areas of fly ash obtained from the URS hydrogeologic report (URS, 2001a) and an early map of planned topographic contours of the golf course (March 2002 site plan) , downloaded from the City of Chesapeake website. As-built information for the golf course was not available. CDM assumed that no fly ash was placed in low-lying areas or ponds. The estimated footprint of the areas receiving fly ash totals approximately 92.4 acres. The estimated total mass of stabilized ash used to construct the golf course was 1.5 million tons, projected by URS (2001b) and cited by MACTEC (2009).

Precipitation and irrigation water infiltrating into the landfill from the ground surface and not lost to evapotranspiration is assumed to percolate in a primarily vertical direction through the fly ash and underlying soil until it reaches the saturated groundwater zone. In the groundwater transport model, the total simulated arsenic and nitrate mass were applied to groundwater evenly over the entire estimated area of fly ash shown in Figure 7-1, at a vertical depth equivalent to the approximate base of the fly ash (top of natural surficial silt/clay layer). For mass loading purposes, CDM used an arsenic value of 43 mg/kg arsenic in fly ash. This value was calculated from the average concentration of 59 mg/kg from most recent investigations less the 95% confidence interval of the mean of 16 mg/kg. The source loading rate (total mass flux) applied to the transport model was calculated as the product of the volumetric rate of infiltration (groundwater recharge) through the fly ash and the estimated constituent concentration in the infiltrating water (leachate) when it reaches the groundwater table. Various combinations of infiltration rates and concentrations were simulated. Development of source loading rates is described in more detail below.

Over time, it is recognized that the source of constituents in the landfill will become depleted. It is assumed that the arsenic concentrations in the leachate will exhibit first-order decay. An initial arsenic source loading rate, ranging from approximately 30 to 878 grams/day, based on source loading calculations and available data, was applied in the arsenic transport simulations as shown in **Table 7-2**. Source loading rates applied in the nitrate simulations are also shown in Table 7-2. The estimated mass loading rates to groundwater and half-life values representing the rate of source decay in Table 7-2 were calculated as described below, based on available data.

Infiltration rates through the fly ash, representing leachate production, were estimated based on HELP model results discussed in Section 5.3. Values of 15.8 in/yr and 7.5 in/yr are representative of the upper and lower ends of the range of reasonable leachate production rates generated by the HELP model, respectively, and are in general agreement with the local groundwater infiltration rates in the calibrated groundwater models. Based on the groundwater flow model simulations, the lower end of this range may more reasonably estimate true annual infiltration and leachate production rates, as the HELP model was developed for the purpose of conservatively estimating leachate production rates for landfill design. Source loading estimates for the transport model simulations were generated using both infiltration values.

It was generally assumed that the relationship between a concentration of a constituent in the fly ash and its concentration in leachate produced by the fly ash is represented by a linear relationship, expressed by the partitioning coefficient (K_d). Instantaneous equilibrium between the liquid and solid phases is assumed.

The leachate concentration multiplied by the infiltration rate was then used to generate an annual source loading rate estimate to groundwater beneath areas of emplaced fly ash. For arsenic, concentrations were assumed to be constant over each year. Using a similar approach for nitrate, concentrations were assumed to be constant over just a ten-day period, because it is quite soluble in water. Over time, as the quantity of available arsenic or nitrate in the landfill is depleted, concentrations of the constituent in the leachate will likewise decrease.

In the transport model, the parameter K_d is used to calculate the source loading rate, and, indirectly, the rate of source depletion, as described in this section, as well as the retardation factor (R) as described in Section 7.2.2.

The initial estimate of available constituent mass for transport into groundwater as leachate was calculated in two different ways for each chemical investigated for the transport model analysis, arsenic and nitrate, based on available data, to obtain a reasonable range of source loading rates for the transport model simulations. These values are summarized in Table 7-2.

The two methods employed for the arsenic simulations were leachate loading based on TCLP results from fly ash samples and leachate loading based on total constituent concentrations in fly ash. Because the TCLP data represent leaching based on a very short period of time, these results represent the lower bounds of the effects of the fly ash on leachate water quality that can be reasonable expected. Under the actual site conditions, the fly ash will be in contact with the leachate water for a much longer period of time and the constituent concentrations will likewise increase. As a result, the second method was employed to provide and upper bound on the reasonable constituent concentrations in leachate. The upper bound method based on the total arsenic concentrations in fly ash are believed to be most representative of actual conditions. Because of nitrate’s high degree of solubility, the lower bound of the leachate water quality was represented by assuming that slight nitrate retardation would occur during leaching. The upper bound assumed that all nitrate would be leached from the fly ash in the first flush of leachate water. The differences in results between the two methods were relatively small and the resulting range is believed to be representative of the actual conditions.

<u>Arsenic Method #1: Leachate loading from TCLP results</u>
Toxicity Characteristic Leaching Procedure (TCLP) concentrations were used to estimate the total mass of leachable constituents in the fly ash (Method #1). This method was also employed by URS (2001b), who performed TCLP analyses on ash amended with various amounts (1%, 3%, and 5%) of both cement kiln dust and lime kiln dust. MACTEC (2009) indicated that the fly ash was generated by amending raw

fly ash with approximately 2% of an unspecified combination of cement kiln dust and lime kiln dust. Thus, the mass of leachable arsenic in the fly ash was estimated using the average TCLP analysis results for the fly ash amended with 1% and 3% of both cement and lime kiln dusts as reported by URS (2001b) in Table 2.5 of their report: average = 0.23 mg Arsenic/L (equivalent to 230 ug/L). In Method #1, this TCLP concentration is used to estimate the total mass of mobile or leachable arsenic in the fly ash.

To perform the standard TCLP analysis, a sample of the treated fly ash was combined with 20 times its mass in acidic aqueous leaching fluid, agitated by end-over-end rotation of the testing vessel for 18 hours, filtered, and the resultant fluid analyzed for the chemicals of concern. To calculate an equivalent mass of leachable arsenic in the fly ash at the site, used to determine source loading rates for the transport model, the average of the measured arsenic TCLP values was multiplied by 20 to estimate the concentration of mobile arsenic in the fly ash: 4.6 mg Arsenic/kg stabilized fly ash. Thus, it was assumed that the total mass of the analyte in TCLP leachate represents the total mass available to be leached from the original sample under natural conditions, with time.

K_d values for arsenic have been reported in the literature (e.g., EPA, 1996 and EPA, 2004); they are typically variable, but most lie between 20 and 30 liters/kilogram (L/kg). The smaller average value, 20 L/kg, was selected for the transport model calculations to simulate a conservative situation with maximum arsenic concentrations in the leachate. Simulations of a K_d of 30 L/kg were performed as sensitivity runs. Source loading rates calculated using Arsenic Method #1 ranged from approximately 30 to 96 milligrams per day (mg/d), as summarized in Table 7-2.

<u>Arsenic Method #2: Leachate loading from total concentration in fly ash</u>
MACTEC (2009) tested four fly ash samples collected from three borings within the site. The average of the four arsenic analytical results was 42.7 mg Arsenic/kg of fly ash. For the purpose of generating a conservative upper limit, Method #2 assumed that all of the arsenic mass present in the fly ash would be available to be leached into groundwater, a situation that would be very unlikely in reality. Method #2 then proceeded in the same manner as Method #1 to derive an estimated initial source loading rate using as a basis the assumed K_d relationship and the infiltration rate. Source loading rates estimated using Arsenic Method #2 were approximately one order of magnitude higher (more conservative) than Arsenic Method #1, based on the higher estimate of initial arsenic mass in the fly ash.

<u>Nitrate Method #1: Leachate loading based on K_d</u>
Nitrate is negatively charged, and thus, unlike arsenic and most positively charged metals, can move relatively unaffected by adsorption through the mostly slightly negatively charged soil particles. Nitrate Method #1 assumed slight adsorption. An R of 1.1 was selected, and the corresponding K_d was back-calculated based on the retardation equation below.

$$R = 1 + \{(\rho_b/n) \times K_d\}$$

Where

ρ_b is the soil bulk density, 1.65 mg/cm^3; and
n is the soil porosity, 0.382, based on fly ash boring B1B data reported by MACTEC (2009).

In this case, the equivalent value of K_d for nitrate was calculated to be 0.023 L/kg. The average "nitrate as nitrogen" concentration in the fly ash from three soil borings was read from the measurements in Table 15 (MACTEC, 2009), and converted to a nitrate concentration of 14.5 mg/kg. Due to its high degree of solubility, the entire nitrate mass in the fly ash was assumed to be leachable. Just as described above in Methods #1 and #2 for arsenic, K_d was used to apportion nitrate from the fly ash to the leachate; except that constant concentrations and source loading rates were assumed for 10-day time increments rather than 1-year increments, due to its high solubility. An estimated initial source loading rate of 122 - 258 kilograms per day (kg/day) to the groundwater beneath the footprint of the fly ash was estimated using this method, depending on the infiltration rate used. The source loading rate decays exponentially with this method, because the leachate concentration is directly dependent on the total mass available in the solid phase. This method does not assume an upper limit for the nitrate concentration in the leachate.

<u>Nitrate Method #2: Leachate loading based on all nitrate mobilized in the first pore volume</u>
Due to the high solubility assumed with Nitrate Method #1, CDM applied a second method for comparison. Nitrate Method #2 assumed that the entire nitrate in the fly ash was flushed out in a volume of water equivalent to the first pore volume of leachate at a constant concentration by direct infiltration of recharge. Thus, the concentration of nitrate in the first pore volume was the total estimated mass of nitrate in the solid fly ash, divided by the total pore volume. The porosity value measured in soil boring B1B (MACTEC, 2009) of 0.382 was used. The number of days required for this first pore volume to flush entirely through the fly ash was calculated by dividing one pore volume by the product of the area of the solid ash fill and the estimated infiltration rates provided by the HELP Model. After the first pore volume passes through the source, it is completely exhausted, so that its nitrate concentration was assumed to go to zero. Estimated constant source loading rates using Nitrate Method #2 ranged from approximately 25.8 kg/day for 2.1 years (767 days), to 12.2 kg/day for 4.4 years (1,615 days).

Although for arsenic and other metals, the annual amount of decay in the source material is expected to be quite low, over a long period of time such as several hundred years, the source depletion may become significant. Where depletion of the constituent source was simulated, the rate of depletion was estimated by calculating the product of the leachate concentration and the leachate infiltration volume over a given time period (one year for arsenic and 10 days for nitrate) to estimate the overall constituent mass reduction in the fly ash during that same time period. The soil/water partitioning coefficient, K_d, was used to estimate a constant equilibrium concentration in the leachate based on the concentration of constituent mass in the fill. Leachate was assumed to reach steady-state concentrations between soil and water based on a linear relationship expressed by K_d. This is a reasonable assumption because the rate of leachate movement through the fly ash is slow.

The estimated mass in the leachate resulting from this calculation for the initial time period was then subtracted from the mass in the solid fly ash, resulting in a new mass of constituent in the fill during the subsequent time period; these calculations were performed in an iterative fashion. The resulting mass reduction was plotted versus time, and the coefficient of decay, λ, was determined by a fit to an exponential function, as follows.

$$Cs(t) = Cs(t_0) * e^{-\lambda t}$$

Initial mass loading rates and coefficients of decay for various combinations of K_d, infiltration (leachate production) rates, and initial mass of constituent in the fly ash were calculated using this method as summarized in Table 7-2.

As expected, the coefficients of source decay are faster for the higher infiltration rates, because more leachate is in contact with the fill in any given time period. Therefore, these simulations represent a conservatively high estimate of the rate of leachate transport into groundwater, as well as of source decay. The lower infiltration rates generate less leachate, and the constituents are more slowly released into underlying groundwater. The source loading rates are most sensitive to the assumption of the available mass that can be leached out of the fly ash.

7.2.2 Transport Parameters

Model parameters used in the transport computations are summarized in **Table 7-3** and include:

- Effective porosity – Advective velocity is inversely proportional to the effective porosity of the aquifer material. Effective porosity is typically less than total porosity because most of the groundwater flow will typically occur in a subset of the soil pores. This is especially the case for heterogeneous soils, and also for vertical transport for confining layers where much of the groundwater flow may occur in discontinuities in the silt/clay layer. Computationally, changes of effective porosity specifications have the same effect on simulation results as changes to adsorption/retardation. Since the effect of adsorption/retardation on arsenic transport simulation results is much greater than the effect of effective porosity, a single effective porosity value (0.20), rather than a range, was assigned in the arsenic simulations.

- Adsorption/retardation – Adsorption to the soil of solutes, most notably charged species such as arsenic, tends to slow the migration of the solute relative to the advective velocity of the groundwater. Arsenic adsorption is often quantified by adsorption isotherm equations, non-linear relationships which typically relate the degree of interaction with the solid matrix based on its content of iron oxides and iron oxy-hydroxides. Since this level of detail regarding the fly ash and native soils was not available, the simpler K_d approach was used, providing a more empirical description of arsenic adsorption, which is in fact often linear over the relatively small concentration range applicable here. Retardation factors were estimated from K_d values, a bulk density of the soil of 1.65, and a measured porosity of 0.382 in the fly ash using the standard groundwater retardation equation. Transport of nitrate, a soluble anion, is not expected to be significantly retarded in groundwater.

- Dispersivity – This parameter controls the variability of transport velocity about the mean rate. Dispersion computations, based on Fick's Law, cause the simulated plume to spread somewhat in both longitudinal (parallel to direction of flow) and transverse directions. The dispersivity values assigned are in the range commonly used for this type of transport modeling. The simulation results are not very sensitive to dispersivity parameters.

- Source decay, λ - First order decay of the source concentrations was estimated for various combinations of K_d values, leachate infiltration rates (recharge), and initial available mass (source loading), as described above. These values were converted to a half-life and are summarized in Table 7-2.

- Constituent decay in the aquifer was not simulated for either arsenic or nitrate. Arsenic is a metal and will not degrade or otherwise decay; nitrate is assumed to be stable in groundwater.

Table 7-3 shows the range of transport parameters used in this study. Unless otherwise specified, these values were applied to all transport simulations.

7.3 Transport Simulations

Transport simulations were run using the combinations of source terms, source decay, and transport parameters indicated in Tables 7-2 and 7-3 for a period of 200 years. Particles representing constituent mass were introduced at model nodes located within the areas of estimated fly ash emplacement at an elevation corresponding to the approximate top of the silt/clay layer noted beneath the site. Where this was above the simulated water table, the mass was allowed to migrate downward into groundwater under a unit hydraulic gradient until the water table was reached. The High Flow Calibrated Model provided the groundwater flow field used as the basis of the transport simulations. This model was selected because it represents a conservative estimate of the water flux underneath the site, and would generate reasonable worst-case conditions of constituent migration with time.

The particle transport paths are derived from the groundwater flow model and are representative of the site-wide flow field. Small-scale variations in the actual particle flow paths in actuality can vary from those simulated by the flow model because small-scale influences on the migration paths are not conventionally characterized for the purposes of a site-wide model. Small-scale influences can be caused by features such as small areas of differing K and small areas where the potentiometric surfaces differ from those included in the model. However, the overall simulation results and the site-wide transport model are valid.

Transport simulation results are illustrated at 5, 20, and 200-year intervals for arsenic simulations in **Figures 7-2** to **7-13**. These figures show calculated concentrations in the top of the surficial aquifer, beneath the silt/clay layer. The arsenic simulations indicate that predicted concentrations are most sensitive to the initial estimate of leachable mass in the fly ash. In addition, it takes approximately 20 years for the arsenic to migrate vertically downward through the silt/clay layer to the upper surficial aquifer. This vertical flow rate would be accelerated by preferential flow pathways in areas where the silt/clay layer is compromised.

Once in the surficial aquifer, the flow field beneath the site is significantly influenced by the ditches. The transport simulations confirm that arsenic migration is likely to be generally toward the ditches, where it would be discharged to surface water. Little to no arsenic migrates down into deeper portions of the surficial aquifer or to off-site locations in the simulations.

A time history of simulated nitrate and arsenic concentrations in the upper surficial aquifer is shown for three locations in **Figure 7-14**. This figure shows that location 3, downgradient of the southern drainage ditch, is not impacted by contaminants in the model simulations.

Subsequent to arsenic mass leaching through the silt/clay layer and reaching the upper surficial aquifer, arsenic concentrations off-site are not expected to be impacted by the fill. After 200 years, the arsenic concentrations beneath the site footprint are estimated to be 0.01 to 0.10 mg/L, or conservatively estimating the total amount of leachable arsenic in the fill, as high as 1-2 mg/L in the upper 15 feet of the surficial aquifer, according to model simulations.

Nitrate simulations, which represent the maximum distance that constituents from the site would be expected to travel since the retardation of this chemical is negligible, show a similar pattern, except that the source of nitrates is expected to deplete much more rapidly than for arsenic or other metals that are highly retarded (adsorbed to soils).

Nitrate simulations show that there is potential for mass to travel deeper into the surficial aquifer with time beneath the footprint of the site, although the same basic pattern is observed of ultimate discharge to the surface drainage system in the transport model, such that mass does not appear likely to migrate off-site. The nitrate simulations can be considered representative behavior of a conservative tracer in groundwater. Their transport is much more rapid in the groundwater than arsenic. Mass that migrated deeper beneath the footprint of the fill did not migrate beyond the influence of the surface drainage ditches in the simulations.

7.4 Sensitivity Analysis

A series of sensitivity simulations was run varying K_d, retardation, source decay, and initial source strengths, as described in Section 7.2. The nitrate simulations represent an analysis of the sensitivity of the transport model to the very high retardation factors estimated for metals.

The transport model was found to be very sensitive to assumptions about both the initial quantity of leachable mass in the fly ash and the adsorption characteristics of the material, as shown by the differences between the transport model results for arsenic and nitrate. Available information that would allow for more precise calculation of these parameters, and/or documentation on this is sparse.

The groundwater flow field was found to greatly limit offsite migration. When the water table is below the level of the ditches such as following heavy rainfall, groundwater could migrate underneath the ditches on a temporary basis. However the simulations indicate that the ditches exert a sufficiently strong hydraulic force on the aquifer on both upgradient and downgradient sides of the ditches that any leachate that may migrate beneath the ditches during periods of high ditch water levels will be pulled back and discharge to the ditch when the water levels recover to normal.

Assumptions about the solubility and leachability of the nitrate into groundwater dominated the nitrate simulations. The fly ash is likely to release nitrate into the groundwater more slowly than simulated in this model. However, while the arsenic source is likely to continue for a period of 200 years or more, appreciable leachable nitrate is expected to be depleted within a few decades at the most conservative estimate.

Because the expected rate of mass transport is low for arsenic simulations relative to depletion of the constituent source due to its high adsorption to soils, the transport model is not particularly sensitive to the rate of infiltration through the fly ash or the source decay, but is sensitive, within the site area, to the total quantity of mobile arsenic that is assumed.

Section 8
Conclusions

Amended fly ash was used as fill in the development of the Battlefield Golf Course. This study was performed to assess the current and likely future water quality in groundwater beneath the site and in offsite locations to determine whether groundwater is currently, or could become, adversely impacted by constituents originating from the fly ash. CDM collected existing data for the site, completed a hydrogeologic investigation, and developed a groundwater model to assess current and future water quality. This report is strictly focused on groundwater and the conclusions provided herein do not address potential issues associated with constituents in surface water, soil, or air. Nor does this report address ecological or human health toxicological issues that may be associated with the site.

8.1 Current Water Quality Conditions

CDM identified ten constituents in shallow groundwater onsite that are elevated as compared to the baseline data set. These constituents include aluminum, ammonia, iron, magnesium, manganese, nickel, nitrate, nitrite, sulfate, and zinc. With the exception of nitrite, all of these constituents have been shown to be present in the fly ash and were detected by leaching tests performed on the fly ash. Analyses for nitrite have not been performed on the fly ash. It is likely that these constituents are elevated because of the site. However, none of these constituents presents current groundwater plumes that can be reasonably mapped by concentration and they have apparently irregular spatial distribution patterns. This is likely because sufficient time has not elapsed following fill emplacement for the water quality effects to become fully recognizable. Onsite monitoring wells MW-5A and -8A were qualitatively identified as high-concentration outliers based on aluminum and nickel and the cause for these high concentrations should be evaluated.

8.2 Groundwater Flow

CDM developed a 3-dimensional numerical groundwater flow model to investigate local groundwater flow patterns and to enable a more sophisticated basis for an analysis of potential offsite constituent migration. The data used to support the flow model development, primary assumptions, and model results are summarized below. A complete description of the model and associated assumptions are included in Section 6.

To develop an assessment of the local hydrology, and generate appropriate input parameters for the numerical groundwater flow model, CDM performed several qualitative and quantitative analyses, including an APT analyzed with AQTESOLV and further evaluated with the groundwater model. The evaluation suggests that the hydraulic conductivity of the surficial aquifer is in the range of approximately 30 to 70 ft/day.

The groundwater flow model was developed to simulate saturated groundwater flow beneath the site and surrounding area. The groundwater flow model includes four hydrogeologic units, listed from top to bottom:

- Surficial silt-clay semi-confining layer approximately 5-feet thick that underlies the fill at the site;
- Surficial Aquifer – fine- to coarse-grained sand and gravel interbedded with fine-grained sediments approximately 50- to 60-feet thick in the site vicinity;
- Yorktown Confining Zone – heterogeneous semi-confining zone approximately 20- to 30-feet thick in the site vicinity that separates the underlying Yorktown aquifer from the overlying surficial aquifer; and
- Yorktown Aquifer - heterogeneous unit approximately 40- to -60 feet thick composed of sand with interbedded silt/clay.

The groundwater flow model boundaries selected include the Intracoastal Waterway to the north, the North Landing River and Currituck Sound to the east, the Northwest River to the south and southwest, and swampland and unnamed tributaries to the Northwest River to the west.

Approximately 63 known residential wells in the site vicinity are used for water supply. The well depths for 17 if these wells are known and the wells were assigned to these depths in the groundwater flow model. The remaining 46 residential wells that do not have available well depth data were simulated as pumping from the surficial aquifer as a conservative measure. All residential wells were assumed to pump continuously at the average residential water usage rate of 0.45 gpm based on typical City of Chesapeake water use rates.

The groundwater model was calibrated using groundwater heads measured in monitoring wells in 2008 and 2009 and an APT conducted by CDM in 2009. The groundwater flow model sensitivity to recharge and the hydraulic parameter assignments for K_h and K_v was assessed and the model was found to be sensitive to these two variables. As a result, two alternative flow models were developed incorporating the range of parameter value assignments considered appropriate for the site.

The relatively low hydraulic conductivity of 30 to 70 ft/day derived from the APT required the use of a recharge rate of 3.1 in/yr that was lower than expected based on regional values. A higher hydraulic conductivity of 100 ft/day was required to achieve calibration at the more typical recharge rate of 10.1 in/yr. Therefore, the higher-flow USGS model assumptions were simulated in addition to the APT results. This resulted in two simulated flow fields for the groundwater flow model that represent the most likely range of possible groundwater flow rates beneath the site and the surrounding area.

The regional groundwater flow in the surficial aquifer is toward the southeast and the Atlantic Ocean with localized variations in this flow direction being caused by surface water features. Numerous local surface water features on and near the site complicate patterns of groundwater flow in the shallow surficial aquifer, including onsite ponds, and a network of drainage ditches, including one that is located along the southern and southeastern border of the site.

Based on water level elevations, CDM assumed that onsite ponds SG-3, -9, -10, -11, -12, -16, and -17 have good connection with the shallow groundwater and onsite ponds SG-1, -2, -19, -6, -7, and -8 have poor connection. Furthermore, because SG-3 and SG-16 discharge to surface drainage ditches, they are assumed to behave essentially as groundwater drains.

Onsite, groundwater flow model simulations in the surficial aquifer, as well as groundwater and surface water data from the network of monitoring wells, indicate that groundwater from beneath the footprint of the site area primarily discharges to the ditches along the southern and southeastern site boundary. Local pumping from residential supply wells does not appear to have an appreciable impact on local groundwater flows, except in their immediate vicinity.

In summary, the groundwater flow model was found to be most sensitive to the recharge rates and hydraulic conductivity assumptions with regard to flow model calibration and the range of values considered acceptable by CDM were used to effectively bracket the range of flows. The pumping effects from the residential wells on the groundwater flow field beneath the site were found to be negligible. This is primarily because the groundwater flowing beneath the site primarily discharges to the intervening drainage ditch that is south and southeast of the site boundary and this discharge occurs for both the APT flow model and the high flow model. Should the hydraulics of this drainage ditch change in the future this discharge could change. Features such as a downstream dam or siltation and ditch infilling at the site would likely decrease the groundwater discharge from the site.

The geographic locations of the individual residential wells within the properties were not available for inclusion in the model. As a result, CDM used the approximate center of the properties for each of the residential wells. The results of the overall site-wide model are not highly sensitive to the location on each individual property. However, if a residential well were located very close to the ditch, it is possible that a portion of the groundwater pumped from the well could be derived from beneath the ditch and in the future that groundwater may contain constituents derived from the fly ash.

8.3 Future Water Quality Conditions

Future water quality conditions beneath the site and in offsite locations were assessed with the aid of a numerical transport model. The results of this groundwater modeling effort indicate that water quality in the surficial aquifer beneath the fly ash fill will be affected by the site. In order to assess the future magnitude of the water quality affects, the model variables and assumptions that are sensitive to the model results were considered for ranges of reasonable values. Representing a range of values was appropriate because the currently available data did not provide a basis for transport model calibration. CDM concludes that sufficient time has not passed since fly ash emplacement for a definable plume to develop that would provide a basis for transport model calibration. While not calibrating the groundwater model for observed contaminant transport could increase the range of the most-probable results simulated for contaminant transport, not calibrating the model does not invalidate the transport model. CDM concludes that the future groundwater quality will likely fall within the approximate range simulated by the model. However, predicting a constituent concentration at a given location for a future date is not a reasonable expectation for the model, primarily because contaminant transport calibration data do not currently exist.

The groundwater flow model formed the basis for simulations of groundwater transport simulations. The groundwater transport model requires inputs for the mass of constituents being added to the groundwater. The EPA HELP model was used to develop the water infiltration rates through the fly ash fill and these infiltration rates were used to develop the mass loading inputs. Arsenic and nitrate were selected as the constituents for the transport model. These constituents were selected on the basis of being present in the fly ash, being present in leachate from tests performed on the fly ash, being regulated constituents having drinking water MCLs, and their mobility in groundwater.

Arsenic has a low drinking water MCL of 10 ug/L and was present in fly ash leachate samples. It should be noted however that the ambient background concentration of arsenic in groundwater can also exceed the MCL. Arsenic typically has a moderate mobility and would be expected to leach from the fly ash for a longer period of time than nitrate. Nitrate does not have a drinking water MCL but has a state standard of 5,000 ug/L and was present in fly ash leachate samples. Nitrate has a high mobility and would be expected to leach from the fly ash relatively quickly as compared to the other constituents in the fly ash. Simulations using these two constituents should effectively represent the range of effects on groundwater that can be expected from the fly ash.

Data from TCLP leaching tests performed on samples of fly ash and samples from borings in the emplaced fly ash at the site suggest the potential for groundwater contaminants in the fly ash to leach into precipitation and irrigation water that infiltrates from the surface and percolates through the fly ash fill to the saturated groundwater system. Potential differences may exist between the TCLP leaching data and the actual leaching at the site. However, CDM conservatively used the mass leached from fly ash by the TCLP test to represent the total leachable mass from the fly ash for one scenario. CDM reviewed the chemical data from laboratory analysis of samples of fly ash, as well as pre-construction analyses used in design and planning to assess potential future water quality impacts resulting from the use of the fly ash in golf course development. For mass loading purposes, CDM used an arsenic value of 43 mg/kg arsenic in fly ash. This value is calculated from the average concentration of 59 mg/kg from most recent investigations less the 95% confidence interval of the mean of 16 mg/kg.

The estimated footprint of the areas receiving fly ash totals approximately 92.4 acres. The estimated total mass of stabilized ash used to construct the golf course was 1.5 million tons, projected by URS (2001b) and cited by MACTEC (2009). CDM conservatively assumed that no fly ash was placed in low-lying areas or ponds. In addition, CDM assumed that fly ash was not placed along the site boundaries such as in the southwest corner. No current data are available defining the precise locations of fly ash fill. The transport simulations were performed for 5 years to simulate current conditions and for 20 years and 200 years to simulate future conditions.

The conclusions for future water quality represent static site-wide conditions that are assumed to not change in the future. Examples of site conditions that could change in the future and cause these conclusions to change include items such as deterioration of the soil cover over the fly ash, low plant transpiration rates over the fly ash, and changes in the hydrology of the drainage ditches surrounding the golf course.

8.3.1 HELP Model Infiltration Rates

A range of estimated infiltration rates to groundwater through fly ash fill was developed based on the layer properties for the soil cover, fly ash, and underlying soil. Eight scenarios were evaluated for infiltration rate determination and evaluated for sensitivity. In these scenarios, the soil cover thickness was evaluated at 6 and 18 inches and the evaporative zone depth was evaluated at 6, 10, and 18 inches. The hydraulic conductivity of the soil cover was evaluated at three values as well. The next lower layer in the HELP model was the fly ash fill and this layer was included with a single hydraulic conductivity value of 5×10^{-5} cm/sec and a variable thickness based on available information regarding the thickness across the site. The lower layer beneath the fly ash fill was included as a relatively low permeability layer having conductivity between 8.27×10^{-7} cm/sec and 6.4×10^{-7} cm/sec. This layer was excluded in one scenario because it may be discontinuous. These data input ranges for the HELP Model were derived from the previous model prepared by URS and additional information obtained by CDM.

The resulting infiltration rates ranged from approximately 7 to 20 in/yr. The model results were most sensitive to those elements of the model related to the soil cover parameters, including hydraulic conductivity, thickness, and depth of the evaporative zone. Based on the HELP Model results, a range of infiltration from 7.5 to 15.8 in/yr appears reasonable and this range excludes the infiltration rate calculated with the lower layer being absent. Assuming that the lower layer is present at all locations is a conservative assumption because water levels indicate that it may be absent beneath certain onsite ponds. It should be noted that these infiltration results do not include infiltration associated with irrigation of the golf course. Therefore, it is reasonable to assume that the fly ash fill is exposed to infiltration rates that are higher than those representing the reasonable range. Should the soil cover become eroded and thinned in the future or if the vegetative cover becomes stressed, the infiltration rates will likely increase.

8.3.2 Arsenic Migration

A range of estimated arsenic to groundwater mass loading rates over time for the transport model simulations was developed based on the following assumptions: It was assumed that the arsenic concentrations in the leachate will exhibit first-order decay and the relationship between the arsenic concentration in the fly ash and its concentration in leachate is linear, and expressed by K_d. A range of K_d values was evaluated for arsenic. Instantaneous equilibrium between the liquid and solid phases is assumed. The simulated steady-state groundwater flow field and the estimated arsenic loading rate were used to simulate potential future transport of arsenic in the groundwater over a period of 200 years. The groundwater transport simulations represent advection of arsenic with flowing groundwater, adsorption of arsenic to the soil, and dispersion of the arsenic plume. Because arsenic has a strong tendency to adsorb to soil particles, its transport in the subsurface is much less rapid than the groundwater velocity. Field data are not available to calibrate the transport model, so a range of transport parameters and loading rates was simulated.

After a simulated period of approximately 20 years, the model results indicated very minor impacts to the surficial aquifer were observed. Because the arsenic depletion rate in the fly ash is expected to be low, the model predicts that water quality impacts in shallow groundwater beneath the site would persist for 200 years or more. Simulated arsenic concentrations in the upper surficial aquifer at the conclusion of the 200 year arsenic transport simulations were 0.1 mg/L to 2 mg/L, with the higher concentrations reflective of a conservatively high estimate of mass leaching into the aquifer. The model did not indicate that appreciable arsenic from the site will migrate offsite beyond the ditch during that period. The model indicated that arsenic would ultimately discharge to the drainage ditch just south and east of the golf course. There was no arsenic migration to the Yorktown aquifer in the simulation result.

8.3.2 Nitrate Migration

Transport of nitrate is not significantly retarded, and it is expected to migrate through the aquifer at approximately the same rate as groundwater. It was assumed that the nitrate concentrations in the leachate exhibit first-order decay and the relationship between the nitrate concentration in the fly ash and its concentration in leachate is linear, and expressed by K_d. CDM also conducted simulations of nitrate transport from the fly ash in the groundwater. Nitrate was simulated as a conservative substance, without significant adsorption or degradation, and thus migrated much faster than the arsenic.

The nitrate transport simulations indicated that the impact to the upper surficial aquifer beneath the site is possible within the first 5 years following equilibration of the water infiltration and leaching process. The maximum simulated nitrate concentration in groundwater is 50 to 500 mg/L. Because nitrate is much more soluble than arsenic, the source mass in the fly ash was estimated to be largely depleted from the source within approximately 20 years. The model results show that nearly the entire simulated nitrate mass had discharged to the drainage ditch south of the golf course within 20 years.

Hydraulic connections between the onsite ponds and the underlying surficial aquifer would provide a preferred pathway through the 5-foot thick silt/clay layer at the base of the fly ash for constituents in the fly ash leachate to migrate into shallow groundwater. Due to the characteristics of the local groundwater flow field, the model simulations suggest that preferred pathways, if present, are not expected to impact water quality at offsite receptors, although the direct connection of the ponds to the surficial aquifer would decrease the length of time required for leachate constituents to be observed in shallow groundwater directly beneath the site.

8.4 Future Land Use Considerations

The groundwater flow and transport model simulations of future conditions can become invalid if the assumptions related to leaching from the fly ash and the site-wide hydrology change because of future land use changes. Examples of the type of changes that could occur and some of the effects of these changes are depicted on **Figure 8-1** and described below.

One prominent feature that effects the conclusions of the model centers on the drainage ditch located to the south and southeast of the site. Based on the investigation data, this ditch serves as the receiving water body for groundwater that passes beneath the site and therefore limits the extent of groundwater migration to the south. The current ditch channel appears to be sufficiently deep to allow groundwater to discharge efficiently to the ditch and to flow unrestricted downstream based on the observed potentiometric surface data. These conditions must be maintained or improved into the future in order for the ditch to remain a limiting factor on groundwater migration to the south.

Examples of possible future changes in the ditch include infilling of the ditch by sediments and blockages/restrictions of the ditch flow by debris or constructed features. Infilling of the ditch channel could occur from sediments being transported from upstream and settling in the ditch near the site. Construction activities could also contribute to ditch infilling. The ditch must also be capable of supporting downstream flow into the future to remain effective. Flow restrictions are possible from man-made dams, undersized culverts, or debris deposited during storm events or by beavers. A period of severe drought could also lower the water table surface in the surficial aquifer and cause groundwater to no longer discharge to the ditch. Under this circumstance, groundwater beneath the site would continue to migrate beyond the ditch to offsite locations.

The infiltration rates through the soil cover over the fly ash could also be increased in the future, which will increase the rate of constituent loading to the groundwater system and cause the constituent concentrations to increase quicker and to higher concentrations than those simulated by the model. Decreases in the soil cover thickness to less than two feet can cause an increase in the infiltration rate. The soil cover thickness could be decreased by erosion and construction/landscaping activities on the golf course. The model also assumes that the infiltration rate is offset in part by evapotranspiration. Should the vegetation growing in the soil cover over the fly ash become stressed or vegetation with low transpiration rates be used, more precipitation can infiltrate through the soil cover and increase the leaching rate. Because the turf is assumed to require irrigation, this will increase the infiltration rate as well, although the vegetation will benefit. As a result, a balance should be maintained between irrigation requirements, using turf species with appropriate transpiration qualities, and keeping the vegetation healthy. Since April 2008, bare areas of soil cover and eroded soil areas have been observed on the golf course.

Section 9
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