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**SANITARY LANDFILL LOCAL-SCALE FLOW AND TRANSPORT
MODELING IN SUPPORT OF ALTERNATIVE CONCENTRATIONS
LIMIT DEMONSTRATIONS, SAVANNAH RIVER SITE (U)**

MASTER

Westinghouse Savannah River Company
Savannah River Site
Aiken, SC 29808

Prepared for the U. S. Department of Energy under contract No. DE-AC09-89SR18035

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1.0 INTRODUCTION

1.1 Background

The Savannah River Site (SRS) is a Department of Energy (DOE) facility located near Aiken, South Carolina which is currently operated and managed by Westinghouse Savannah River Company (WSRC) (Figure 1.1). The Sanitary Landfill (Sanitary Landfill) at the Savannah River Site (SRS) is located approximately 2000 feet Northwest of Upper Three Runs Creek (UTRC) on an approximately 70 acre site located south of Road C between the SRS B-Area and UTRC (Figure 1.2). The Sanitary Landfill has been receiving wastes since 1974 and operates as an unlined trench and fill operation. The original landfill site was 32 acres and is shown as the central shaded portion of the Sanitary Landfill in Figure 1.2. This area reached its capacity around 1987 and a Northern Expansion of 16 acres and a Southern Expansion of 22 acres were added in 1987. The Northern Expansion has not been used for waste disposal to date and the Southern Expansion is expected to reach capacity in 1992 or 1993. The waste received at the Sanitary Landfill is predominantly paper, plastics, rubber, wood, metal, cardboard, rags saturated with degreasing solvents, pesticide bags, empty cans, and asbestos in bags (Sirrine Environmental Consultants, 1991).

In 1988, evidence of elevated levels of RCRA hazardous constituents were noted in the groundwater at the Sanitary Landfill. Consequently, the Sanitary Landfill was designated a RCRA Solid Waste Management Unit (SWMU). Through an agreement between the state of South Carolina and WSRC, a Renewal for a RCRA Part B Permit Application is being prepared for submission to the South Carolina Department of Health and Environmental Control (SCDHEC) by March 31, 1993. As part of the Part B Application, WSRC wants to perform a risk-based Alternative Concentration Limit (ACL) Demonstration for all Sanitary Landfill Contaminants of Concern (COCs). In order to complete an (ACL) demonstration

for the Sanitary Landfill COCs, transport of contaminants from the Sanitary Landfill through the groundwater regime must be quantitatively evaluated.

As required under RCRA, a landfill must be closed by installing a cap (CFR 40 265.310(a)). Of particular interest to WSRC is the effect a cap would have on the local Sanitary Landfill heads and resulting flow lines. Of interest to WSRC also were the long term effects upon aquifer COC concentrations resulting from the Sanitary Landfill cap.

At the time the modeling in this report was performed, it was thought that solvent wipe rags were deposited only in the original part and the southern half of the Southern Expansion of the Sanitary Landfill (see Figure 1.2). After the groundwater modeling was contracted and completed, information regarding additional solvent wipes placement within the Southern Expansion was received (Letter, J.S. Roberts to D.E. Wilson, ESH-ENV-920360, November 20, 1992). The volume of solvent wipes placed in the landfill amounted to 2-3 wipes every three months, and is considered by SRS as not significant enough in volume to alter the outcome of the modeling. The cap proposed for the closure of the Sanitary Landfill will be completed over all areas that received the wipes.

1.2 Objectives of This Study

The objectives of this modeling study are twofold: (1) to create a local scale Sanitary Landfill flow model to study hydraulic effects resulting from capping the Sanitary Landfill; and (2) to create a Sanitary Landfill local scale transport model to support ACL Demonstrations for a RCRA Part B Permit Renewal.

To study the local hydraulics in the vicinity of the Sanitary Landfill, a steady-state calibrated two-dimensional areal finite-difference flow model was constructed. This model was used to examine flow lines, flow velocities, and to evaluate potential exposure pathways from the Sanitary Landfill to the discharge areas. This

model was also used to examine the transient hydraulic effects in the vicinity of the Sanitary Landfill resulting from various capping scenarios. This local-scale areal model is the first flow model developed specifically for the Sanitary Landfill vicinity. Beaudoin et al. (1991) developed a regional groundwater flow model for the A/M Area which included the Sanitary Landfill area. Figure 1.3 shows the Sanitary Landfill model area in relation to the larger regional A/M Area Model.

The local-scale Sanitary Landfill model differs from the regional model in several respects. First, because the local-scale model is at a smaller scale than the regional, the model can differentiate between discharge into wetlands and into UTRC. The local model also treats the Tertiary Aquifers as unconfined which is different than the regional model of the area which uses a confined formulation. Finally, the local-scale model combines layers 2 and 3 (Upper and Lower Congaree) of the Beaudoin et al. (1991) regional model into one aquifer which is consistent with the hydrostratigraphic Steed Pond Aquifer (I/IIC) after Aadland et al. (1992).

The local-scale flow model was the basis for the development of a two-dimensional vertical flow and transport model. This model was used to quantify transport of the Sanitary Landfill COCs between the Sanitary Landfill and the point of exposure (POE). The transport model is conservative in that it models maximum concentrations along the centerline of the plume neglecting lateral transverse dispersion. Relative and absolute concentrations have been calculated for each COC at the POC, the POE (nearest wetland discharge point), and UTRC. Both capped and non-capped conditions have been simulated for two different source loading scenarios. The relationship between concentrations at the point of compliance (POC) and the POE have been quantified through the use of attenuation factors.

An analytical transport model was used in conjunction with the calibrated areal flow model to estimate the lateral extent of Sanitary Landfill groundwater

contamination under conservative assumptions. The analytical model was used to delineate the wetland area and the stretch of UTRC that may potentially discharge contaminants from the Sanitary Landfill. A groundwater to surface water dilution factor was calculated based upon the volumetric flow into the potentially contaminated wetland area. UTRC COC concentrations can be calculated down gradient of the discharge area using the groundwater to surface water dilution assuming instantaneous and perfect mixing in UTRC.

Figure 1.1 Location of the Savannah River Site, South Carolina.

Figure 1.2 Site Map of the Sanitary Landfill and Vicinity

Figure 1.3 Relationship Between Local Scale Flow Model and Regional Scale Flow Model

2.0 HYDROGEOLOGIC FRAMEWORK

The hydrogeologic framework for the Sanitary Landfill area and the surrounding regions has been previously documented in several reports including those from SRS staff, outside consultants and the U.S. Geological Survey. The information contained in these reports was the basis on which the conceptual model of groundwater flow in the Sanitary Landfill area was constructed. The following section will discuss the hydrogeologic framework in the vicinity of the Sanitary Landfill. The discussion will be divided into three general topics: (1) geologic setting, (2) hydrostratigraphic framework and (3) groundwater flow.

2.1 Geologic Setting of the Area

The SRS site is located on the Aiken Plateau which lies within the Upper Atlantic Coastal Plain Province (Aadland and Bledsoe, 1990). The Atlantic Coastal Plain is underlain by a wedge of sediments which increase in thickness down dip in a southeasterly direction from the SRS towards the Atlantic Ocean. The sediments consist mainly of unconsolidated interbedded sands, silts and clays. Some gravel and carbonate deposits are also present. The unconsolidated sediments lie on a southeast-dipping crystalline basement. Although some clay stratigraphic units at the SRS are laterally extensive, most vary in thickness and typically are not extensive. These irregular clay units yield an intricate system of aquifers intermingled with discontinuous aquitards which vary greatly in overall vertical conductivity, resulting in a complex flow regime which is difficult to characterize with simplistic models.

The coastal plain sediment wedge is composed primarily of late Cretaceous and Tertiary sediments. Thin alluvial sediments of Tertiary and Quaternary age occur surficially in places within the SRS boundaries (Fallaw, 1991). These sediments overlie Paleozoic crystalline and metamorphic rocks associated with the Appalachian orogeny.

2.2 Hydrostratigraphic Framework

Several attempts have been made to divide the sediments which underlie the SRS into stratigraphic or hydrostratigraphic systems. In many of the hydrogeologic characterization studies, inconsistent terminology has been used to describe the hydrostratigraphic units at the site. One reason for the inconsistency in the terminology lies in the attempt to correlate the hydrostratigraphic nomenclature and the lithostratigraphic nomenclature by defining the hydrostratigraphic units according to their relation to the lithostratigraphic units. Aadland and Bledsoe (1990) and Aadland et al. (1992) propose a methodology which eliminates the attempted correlation between the two nomenclatures and results in a "hydrostratigraphic code" which has recently been used to describe the aquifer systems at the SRS. Although the details of this code are not described here, the nomenclature established in Aadland et al. (1992) will be used in this report. Figure 2.1 explains the hydrostratigraphic units and their correlation to the old nomenclature. The next section contains a brief description of each of the hydrostratigraphic units which are relevant to this modeling study.

2.2.1 Meyers Branch Confining System (I-II)

The deepest hydrostratigraphic unit of concern for the Sanitary Landfill local-scale flow model is the Meyers Branch Confining System (I-II). The hydrostratigraphic Meyers Branch Confining System correlates with the lithostratigraphic Ellenton Formation. As described by Bledsoe (1988), the Ellenton consists of lignitic clays and silty clays with some very fine to fine sand. The Ellenton is typically considered to be (Beaudoin et al., 1991) a primary confining bed that partially isolates the shallow aquifer system (Steed Pond Aquifer in Figure 2.1) from the deeper hydrogeologic system (Crouch Branch Aquifer in Figure 2.1). The clay layers which comprise the Ellenton range from 1 to over 20 feet in thickness in the A/M Area to the north of the Sanitary Landfill area.

Although boring logs indicate some heterogeneity in the depth and thickness of the Meyers Branch Confining System (Ellenton) clays, geologic cross-sections of the area indicate that the clays are extensive throughout the Sanitary Landfill model area and serve as a continuous confining unit. For the purpose of developing a local scale flow model of the Sanitary Landfill area, the Meyers Branch Confining System was assumed to provide hydraulic confinement between the overlying Tertiary Steed Pond Aquifer and the underlying Cretaceous Crouch Branch Aquifer. According to hydrostratigraphic cross sections interpreted by Aadland and Bledsoe (1990), the top of the clay layers of the Meyers Branch Confining System range from about 40 ft above mean sea level (AMSL) at UTRC to approximately 52 feet AMSL at the northern extent of the Sanitary Landfill flow model boundary along a section running north-northwest.

2.2.2 Steed Pond Aquifer (I/IIC)

Aadland and Bledsoe (1990) make a hydrostratigraphic distinction between the lithostratigraphic units of the Santee, Congaree, and Upper Fourmile Member of the Fishburne Formation in the southern and northern areas of the SRS. South of Upper Three Runs Creek, they report a semi-confining unit, the Gordon Confining Unit (IIA -IIB), which serves to divide the Congaree into upper and lower zones. Regionally at the SRS the Gordon Confining Unit divides the Upper Three Runs Aquifer from the Gordon Aquifer. However, north of Upper Three Runs Creek (UTRC) the more permeable beds of the upper and lower aquifer zone are considered to be in overall hydraulic communication due to the thin and intermittent character of the clay layers. The zones essentially form one aquifer with no laterally extensive confining layers (Steed Pond Aquifer). The saturated thickness of the Steed Pond Aquifer ranges from approximately 80 feet at UTRC to over 120 feet in the northern portion of the Sanitary Landfill model area.

2.2.3 Hydraulic Properties

A review of the existing database of hydraulic conductivity was conducted for the A/M Area and the Sanitary Landfill local area. The Ellenton Formation (Meyers Branch Confining System) is composed of two fining upward sequences in the A/M Area (Fallaw, 1991). The Ellenton is therefore composed primarily of clays and silts with some sand stringers occurring throughout. In the A/M Area, there have been no single-well tests in the Ellenton and just a few slug tests in sands. Because the vertical permeability of the Ellenton would be controlled by the permeability of the clays through the harmonic law of composition if the clays are laterally extensive, the vertical permeability of the Ellenton is better represented by the hydraulic conductivity of the fine-grained sediments. The Tertiary Steed Pond Aquifer in the Sanitary Landfill vicinity is composed stratigraphically of the Congaree and the Fourmile Member of the Fishburne Formation. Several pumping and slug tests have been performed in the formations over the A/M Area. In the following paragraphs, a review of the hydraulic testing results in the A/M Area and the Sanitary Landfill Area will be presented.

During the Phase II Monitoring Well construction in the A/M Area, thirteen laboratory permeability tests were performed on fine-grained sediments of the Ellenton (Geraghty and Miller, 1985). The horizontal conductivity of these samples varied from $4.5\text{E-}6$ ft/day for a clay sample to 0.21 ft/day for a sample of clayey sand. The arithmetic and geometric mean of the vertical conductivity was $7.4\text{E-}3$ ft/day and $4.8\text{E-}4$ ft/day, respectively, and anisotropy ranged from 1 to 4.2. Although the thickness of the Ellenton clays may not be constant, the overall vertical conductance of the formation can be expected to be relatively low due to the low conductivity of the intermittent clay zones.

At the Silverton Road Waste Site, the Lower Fourmile Member and the Upper Fourmile Member and the Congaree had pumping tests performed (Geraghty and Miller, 1986). These tests correlate to the hydrostratigraphic Steed

Pond Aquifer. In most wells, transmissivity was determined for drawdown and recovery. In these cases, the average between the two was used as the single monitoring well estimate. Because screen length varied among monitoring wells, the variation in transmissivity is reflective of that. Assuming that the formation produces uniformly across the screen length, hydraulic conductivities were calculated for each monitoring well. The reported transmissivities for the Lower Fourmile Member ranged from 1056 to 240 ft²/day. The hydraulic conductivities ranged from 20 to 108 ft/day with an arithmetic average of 52.2 ft/day and a geometric mean of 42.9 ft/day. Geraghty and Miller (1986) report seven pumping tests performed at the Silverton Road Waste Site in monitoring wells in the interval equivalent to the Upper Fourmile Member and the Congaree Formation. The reported transmissivities ranged from 648 to 241 ft²/day. The hydraulic conductivities ranged from 24.6 to 66.2 ft/day with a geometric mean conductivity of 43.9 ft/day and an arithmetic average of 46.8 ft/day.

As part of the A/M Area Phase IIb Monitoring well construction, seven 1 hour specific capacity pumping tests were performed (Sirriner, 1987) within the Lower Fourmile Member (Steed Pond Aquifer). The specific capacity varied from 0.92 to 3.25 gpm/ft with an average of 2.5 gpm/ft. By converting these specific capacities to transmissivities one receives a range from 246 to 1192 ft²/day. Assuming that the formation produces uniformly across the screen length, hydraulic conductivities were calculated for each monitoring well. The hydraulic conductivities ranged from 44.0 to 212.9 ft/day with an arithmetic average of 119.1 ft/day and a geometric mean of 103.6 ft/day. Likewise, four 1 hour specific capacity pumping tests were performed (Sirriner, 1987) within the Upper Fourmile Member and the Congaree (Steed Pond Aquifer) of the A/M Area. The specific capacity varied from 0.38 to 5.29 gpm/ft with an average of 2.96 gpm/ft. By converting these specific capacities to transmissivities, one receives a range from 73 to 1018 ft²/day. The hydraulic conductivities ranged from 3.7 to 212.6 ft/day with an arithmetic average of 106.8 ft/day and a geometric mean of 47.6 ft/day.

Marine and Bledsoe (1985) report on three slug tests performed in the Lower Fourmile Member (Steed Pond Aquifer) in the A/M Area. The estimated hydraulic conductivities varied from 0.9 to 66 ft/day. Although an average of three data points is of questionable use, the geometric mean of hydraulic conductivity is 10 ft/day and the arithmetic average is 27.7 ft/day. Marine and Bledsoe (1985) report on twelve slug tests performed in the Congaree and Upper Fourmile Member (Steed Pond Aquifer) of the A/M Area. The estimated hydraulic conductivities varied from 0.8 to 65.1 ft/day. The arithmetic average of hydraulic conductivity is 16.5 ft/day and the geometric mean of hydraulic conductivity is 6.5 ft/day. As part of the A/M Area Phase III monitoring well construction (Sirriner, 1990), 12 slug tests were performed in the Lower Fourmile Member (Steed Pond Aquifer). The hydraulic conductivity ranged from 1.3 to 80.5 ft/day with a geometric mean hydraulic conductivity of 24.6 ft/day and an arithmetic average of 39.9 ft/day. As part of Phase III monitoring well construction (Sirriner, 1990), eight slug tests were performed in the Congaree and the Upper Fourmile Member (Steed Pond Aquifer). The hydraulic conductivity ranged from 1.6 to 77.7 ft/day with an arithmetic average of 14.3 ft/day and of the geometric mean hydraulic conductivity of 5.2 ft/day.

The most site specific permeability measurements to the Sanitary Landfill were 26 slug tests performed in a series of new monitoring wells installed at the Existing Sanitary Landfill (Sirriner, 1992). Twenty-six slug tests were performed in the Steed Pond Aquifer as part of a new phase of monitoring well installations at the Existing Sanitary Landfill. The hydraulic conductivities derived from the tests ranged from 1 to 58 ft/day with an arithmetic average of 18 ft/day.

Table 2.1 summarizes the hydraulic conductivity estimates from all the slug and pumping tests described above from the A/M Area and the Sanitary Landfill which correlate to the Steed Pond Aquifer. The arithmetic average of hydraulic conductivity from slug tests vary from 14 to 40 ft/day. The arithmetic average of hydraulic conductivity from pumping tests vary from 47 to 148 ft/day. GeoTrans

(1992) report that the equivalent Tertiary aquifer south of UTRC (Upper Three Runs Aquifer) from the Sanitary Landfill has an average hydraulic conductivity of 0.6 ft/day in the F-Area, 7 ft/day in the H-Area, and 4.4 ft/day at the Mixed Waste Management Facility (MWMF). Pumping tests in the Y-Area yield a range in hydraulic conductivity from 22 to 62 ft/day. The regional model (Beaudoin et al., 1991) applied a hydraulic conductivity of 44 ft/day for the Tertiary Aquifer.

2.3 Groundwater Flow

The major components of groundwater flow for the Sanitary Landfill Area include (1) discharge to the wetlands along UTRC, (2) discharge to UTRC, (3) recharge from precipitation and (4) underflow from the north. These elements and their implications for the local scale flow model of the Sanitary Landfill area will be discussed in the following sections. In addition, the historical head data for the area will be reviewed along with observed horizontal and vertical hydraulic gradients in the Tertiary Aquifer at the Sanitary Landfill.

2.3.1 Wetlands

Wetlands are a very important component of the groundwater flow regime in the Sanitary Landfill area. According to USGS topographic maps, wetland regions may extend to as much as 2400 ft away from UTRC towards the Sanitary Landfill. In addition, a disconnected wetland area is mapped to the west of the Sanitary Landfill. Figure 1.2 shows the wetland boundaries as mapped in the USGS 7.5 minute quadrangle of the Sanitary Landfill area. The occurrence of this isolated wetland area and the general nature of the wetlands associated with UTRC was verified during a site visit on August 28, 1992. Observations made during the site visit indicated that the isolated wetland to the west of the Sanitary Landfill did in fact exhibit groundwater discharge in the mapped region. The foliage was notably different from the areas which did not exhibit groundwater discharge,

indicating that the water table elevation is normally very close to the ground surface in this area.

In an effort to sample the discharging groundwater, WSRC performed a seepage sampling study down gradient of the Sanitary Landfill between the Sanitary Landfill and the USGS mapped wetlands. These sampling stations are also shown on Figure 1.2. Water was discharging and ponding at each of the sampling stations located immediately south of the Sanitary Landfill. In fact, groundwater seeps were observed at the toe of the slope approximately 1000 feet south of the Sanitary Landfill southern boundary (near stations SP 262 and 263). These observations indicated that the USGS map was not an accurate portrayal of the extent of the wetlands at the time the field reconnaissance was conducted. The majority of the mapped wetland area between the toe of the slope and UTRC contained ponded water. The depth of the water varied according to variations in the ground elevation but ranged from about 1 to 12 inches. In many areas, the groundwater discharge was of sufficient magnitude such that overland flow was discernable.

2.3.2 Groundwater Recharge from Precipitation

Groundwater recharge from precipitation has been evaluated in several studies at the SRS. Parizek and Root (1986) used a water budget approach to estimate components of flow in the McQueen Branch surface water basin. The study period began on March 1, 1983 and lasted 13 months. The calculated percentage of precipitation contributing to groundwater recharge was about 29.5%. Although higher than average precipitation was measured for the study period, estimates for each of the budget components can be adjusted to reflect expected values for an "average" year. Although this approach may oversimplify the complexity of estimating the recharge component, it is helpful in evaluating the potential range of recharge. Beaudoin et al. (1991) report that daily rainfall data from the A/M area rain gauge indicated yearly rainfall amounts over the period

from 1967 to 1987 ranging from 37 to 68 in/yr and averaged 47 in/yr. Given this range and mean of precipitation, the estimated value of groundwater recharge according to the simple relationship ranges from about 11 to 20 in/yr and averages about 14 in/yr. Data from Fenimore and Hooker (1977) and Cook (1983) report that the average rainfall at the SRS from the beginning of measurement in 1952 until 1983 was 48 inches. Several modeling studies incorporating the Sanitary Landfill area have used 15 in/yr as an average recharge rate (Geraghty and Miller, 1985; S.S. Papadopoulos and Associates, 1986; Beaudoin et al., 1991). For average precipitation conditions 15 in/yr is approximately 31% of precipitation.

Figure 2.2 shows the yearly rainfall totals from 1987 to 1991 collected from the A/M area rain gauge. Table 2.2 provides monthly rainfall information from 1987 through 1991. As shown in the Figure, yearly precipitation has increased from 1988 through 1991, the average value for the five year period is about 52 in/yr. Assuming 29% of the precipitation serves to recharge the groundwater regime, a recharge value of approximately 15 in/yr is estimated.

2.3.3 Discharge to Upper Three Runs Creek

Upper Three Runs Creek acts as a regional groundwater divide for the Steed Pond Aquifer in the Sanitary Landfill model area. Groundwater from the north and south converge and discharge into UTRC. Upward hydraulic gradients near UTRC may induce a small amount of groundwater inflow from the aquifer units below the Meyers Branch Confining System. During a field trip in August 1992, one section of UTRC was observed at Stream Gauging Station C near the eastern boundary of the Sanitary Landfill local scale flow model (Figure 1.3). At this location, UTRC was approximately 30 feet wide, flowed swiftly, and apparently exceeded 1 foot in depth over 90% of the width.

Beaudoin et al. (1991) report that the gauging station at UTRC and Road A had an estimated base flow of 120-200 cfs in 1987. They estimate that the A/M

area model east of the gauging station comprises approximately 10% of the entire drainage basin. Therefore, they approximate that the A/M area contributes about 12-20 cfs to the total flow of UTRC. The USGS has recorded daily discharge at Station A and C from 1974 to present. The yearly average discharge rates for the C gage and the A gage are 195 cfs and 235 cfs, respectively, over the time period from 1974 through 1990. GeoTrans (1992) report that gains in discharge between gauging Stations A and C range from about 15-75 cfs. Based upon the yearly average flow rates, approximately 40 cfs is gained by UTRC between the C and A gages. Station C is located at the right hand side boundary of the Sanitary Landfill local scale model. Station A is located downstream of Station C approximately 24000 feet. The length of UTRC incorporated in the model is about 10000 feet. Assuming the discharge gain is linearly distributed between the gauging stations, approximately 40% of the discharge gain occurs in the portion of UTRC incorporated in the Sanitary Landfill model. Although it is uncertain what percent of flow in this reach of UTRC is contributed from northern and southern components, if we assume that 50% is contributed from the north, the amount of discharge gain expected in the Sanitary Landfill model would range from about 3-15 cfs and average 8 cfs.

2.3.4 Head Surfaces

Fifty-seven wells are currently sampled on a quarterly basis to monitor groundwater quality in the vicinity of the Sanitary Landfill. Seven of these wells (LFW 6, 7, 16, 17, 18, 19, and 20) were installed in 1981. LFW 10A, 21, 22, 23, 24, and 25 were installed in 1984, and LFW 26 through 42 were installed in late 1986. Twenty-six wells (LFW 43B through 62D) were installed in the first half of 1991 and were first sampled in the third quarter of 1991. By comparison, the Sanitary Landfill started operation in 1974 and, therefore, it is only in the relatively recent past that consistent monitoring has been performed.

Figure 2.3 shows the locations of the wells in the Sanitary Landfill network and distinguishes each well as a Point of Compliance (POC) well, background well, or monitoring well. Forty-five wells are screened in the upper portion of the Steed Pond Aquifer, eight wells are completed in the middle portion of the Steed Pond Aquifer, and four wells are screened in the bottom section of the Steed Pond Aquifer, directly above the Meyers Branch Confining System. Wells screened at the water table are designated by well names which end with a "D" or which do not end with a letter designation. The names of wells completed in the middle zone end in the letter "C" and the names of wells screened in the lower zone end in the letter "B". The primary purpose of the well network that is currently in place at the Sanitary Landfill was to serve as an early detection system to monitor groundwater quality changes very near the Sanitary Landfill which occurred as a result of contaminant migration from the unlined trenches. Although this objective has been accomplished, monitoring information from other areas within the local scale flow model boundaries is very sparse.

Contoured head surfaces for the Steed Pond Aquifer for the 4th quarter sampling event of each year from 1987 through 1991 are shown in Figures 2.4 through 2.6. Water table elevations typically decrease about 20 ft from the north side of the Sanitary Landfill to the south. In general, the average flow direction from 1987 through 1989 is South-Southeast measured relative to Plant North. For years 1990 and 1991 the flow direction is more dominantly to the south. The horizontal hydraulic gradient is on the order of 0.006 ft/ft. In the period from 1985 to 1992, the water table surface elevation fluctuated about 3 to 4 ft over the site. There does not appear to be any significant water table mounding effects in the vicinity of the Sanitary Landfill.

Figure 2.7 shows the historical record of hydraulic head levels for three wells at the Sanitary Landfill. LFW 30 is a background well located at the northern extent of the Sanitary Landfill. LFW 10A is a monitoring well located in the center of the original Sanitary Landfill area and LFW 40 is a Point of Compliance (POC) well

located south of the Sanitary Landfill Southern Expansion. The plot shows the increasing trend in groundwater level elevations between 1989 and 1991. This increase in heads (up to 4 feet) is a result of the increasing average rainfall in the A/M Area vicinity from 1987 to 1991 as depicted in Figure 2.2.

2.3.5 Vertical Gradients

As mentioned above, of the 57 screened intervals as the Sanitary Landfill, 45 are completed across the water table, 8 are completed in approximately the middle of the Tertiary aquifer (designated C), and 4 are completed at the base of the aquifer (designated B). This allows some measure of vertical gradients at the site. According to the observed heads in the well clusters during the fourth quarter of 1991, vertical gradients were downward in 8 of the 12 cluster wells. The downward gradients ranged from 0.007 to 0.069 and averaged 0.02. The average upward gradient in the four other cluster wells was 0.02 and ranged from 0.0008 to 0.071. Only well cluster LFW 57C-D exhibited a significant upward gradient. According to historical head data, it is not unusual for vertical gradients in the vicinity of the Sanitary Landfill to be on the same order as the horizontal gradients.

The head differences measured between screened intervals in well clusters range from almost none to as large as 2 feet. The average head difference is less than a foot. It is unlikely that measurement device error could fully account for these head differences. However, it is possible that head differences between screens at the same cluster could be partially explained by sampling protocol. The water level measurements are taken prior to borehole sampling with a downhole pump. It is conceivable that interferences between monitoring levels could be occurring. The vertical gradients could also be real and a result of heterogeneity and complex localized recharge and discharge patterns.

Figure 2.1 Schematic Relating Stratigraphy and Hydrostratigraphy to Groundwater Flow Models

Figure 2.2 Yearly Precipitation from 1987 through 1991 from a Rain Gauge at the A/M Area

Figure 2.3 Monitoring Well Network at the Sanitary Landfill

Figure 2.4 Head Maps from 1987 and 1988 (Fourth Quarter) Monitoring Events

Figure 2.5 Head Maps from 1989 and 1990 (Fourth Quarter) Monitoring Events

Figure 2.6 Head Map from 1991 (Fourth Quarter) Monitoring Event

Figure 2.7 Historical Water Level Elevations in Three Wells at the Sanitary Landfill

Table 2.1 Comparison of Hydraulic Conductivity Results by Test Event and Stratigraphy/Hydrostratigraphy.

HYDRAULIC CONDUCTIVITY (ft/day)				
Data Source	Congaree and Upper Fourmile Member		Lower Fourmile Member	
	Arithmetic	Geometric	Arithmetic	Geometric
Marine & Bledsoe 1985 A/M Area (slug tests)	16.5 (n = 12)	6.5	27.7 (n = 3)	10.0
Sirrine 1990 A/M Area (slug tests)	14.3 (n = 8)	5.2	40.0 (n = 12)	24.6
Geraghty & Miller 1986b A/M Area (pumping tests)	46.8 (n = 7)	43.9	52.2 (n = 7)	42.9
Sirrine 1987 A/M Area (pumping tests)	148.4 (n = 4)	66.0	119.1 (n = 7)	103.6
	Upper Steed Pond Aquifer		Lower Steed Pond Aquifer	
	Arithmetic	Geometric	Arithmetic	Geometric
Sirrine 1991 Sanitary Landfill (slug tests)	18.3 (n = 22)	10.2	18.2 (n = 4)	17.2

n = number of tests

Table 2.2 Rainfall Data for the A/M Area, Which Includes the Sanitary Landfill and Vicinity (from M-Area Groundwater Monitoring Annual Reports)

Monthly Rainfall (in/month)	YEAR					Average
	1987	1988	1989	1990	1991	
January	7.06	4.47	1.41	2.80	8.14	4.78
February	8.01	1.81	2.53	2.41	1.87	3.33
March	4.33	2.62	5.29	2.89	8.05	4.64
April	0.81	4.08	4.61	1.00	5.78	3.26
May	4.14	1.20	2.33	3.35	4.78	3.16
June	5.19	6.85	8.07	4.78	3.07	5.59
July	4.31	3.51	9.25	7.45	10.80	7.06
August	3.21	6.89	2.66	6.05	9.45	5.65
September	4.05	4.58	5.37	0.70	2.33	3.41
October	0.37	3.35	2.65	19.62	0.77	5.35
November	2.75	2.09	2.99	3.11	1.64	2.52
December	1.27	1.81	4.60	1.57	3.35	2.52
Monthly Average (in)	3.79	3.61	4.31	4.64	5.00	51.25
Yearly Total (in)	45.5	43.26	51.76	55.73	60.03	

3.0 CHARACTERIZATION OF CONTAMINANTS OF CONCERN

Dames and Moore (Atlanta) are currently preparing the Part B Closure Permit Renewal Application for the Sanitary Landfill under subcontract to WSRC. As part of that task, Dames and Moore have determined which constituents present in the groundwater at the Sanitary Landfill are to be considered Contaminants of Concern (COCs) and which may require an ACL demonstration. There are 16 potential constituents that Dames and Moore preliminarily have recommended for ACLs. These constituents are listed in Table 3.1 along with the maximum, mean, and 95th Percentile concentrations for each constituent. Thirteen of these are organics, one is radioactive, and two are metals.

3.1 Distribution of COCs

The groundwater monitoring well concentration database for the Sanitary Landfill has several limitations. The first problem is that the majority of the Sanitary Landfill monitoring wells (and all of the POC wells) are completed across the water table and are located adjacent to, or within the Sanitary Landfill boundaries. The monitoring well network at the Sanitary Landfill does allow for the detection of contaminants emanating from the landfill. The well network also provides fairly good characterization of COC concentrations very near the water table. However, the current monitoring well network does not allow full characterization of the COC plumes. There are no monitoring wells between the immediate vicinity of the Sanitary Landfill and UTRC to characterize the distribution of contaminants in the down-gradient areas. All monitoring wells are near-field sampling points which do not allow for characterization of leachate plume extent. Additionally, very few wells are present within the Sanitary Landfill boundaries and, therefore, characterization of the contamination within the Sanitary Landfill boundaries is not uniquely possible.

To add to these difficulties, sampling histories for most COCs are erratic and detection limits vary between sampling events for the same COC. At times, detection limits have exceeded the MCL for a given COC. All of these factors lead to the conclusion that the extent and magnitude of COC plumes emanating from the Sanitary Landfill cannot be uniquely characterized.

3.1.1 Groundwater Distribution of COCs

The monitoring well network at the Sanitary Landfill has been sampled for pollutants as early as 1984 and has been sampled quarterly since 1986. However, as previously mentioned, since 1986 many COCs have not been included in the chemical analyses and therefore, several gaps exist in the analytical database. Norrell et al. (1992) examined the time history and trends of several of the COCs. His analysis found that the groundwater concentrations of the constituents at the Sanitary Landfill can be characterized as being both temporally and spatially transient with a large degree of heterogeneity in concentrations. This is not surprising given the transient loading of the landfill, the heterogeneous nature of the waste, and the limited monitoring well network. Because of the transient and erratic behavior of COC concentrations within the Sanitary Landfill and because of the strongly nonuniform placement of monitoring wells within the Sanitary Landfill, a different method other than a contour plot has been selected to evaluate the distribution of the COCs around the landfill. The method is based upon first determining the maximum observed concentration value at each monitoring well for each COC. The data is then displayed through bubble plots where the area of the circle is linearly proportional to the concentration. Additionally, a second plot is prepared which posts the maximum values along with the date it was observed. Appendix A contains bubble plots and posted values for all COCs for all wells screened across the water table (level D wells) and maps of posted values are shown for level C and B wells. Although this method is not suited for evaluating a distribution of a COC at a specific time, these plots are well suited for estimation of potential source regions for a given COC within the Sanitary Landfill.

They will be used for this purpose in Chapter 5. These plots also do not require interpolation which can be misleading considering the potential of a strongly heterogeneous source.

Areal distributions of some COCs in the upper water table zone indicate the presence of contaminant plumes which emanate from the Sanitary Landfill. Analyses of quarterly monitoring data from the Sanitary Landfill monitoring network indicates that several organic compounds, metals, and radioactive constituents such as tritium, gross alpha, and radium have been present at levels exceeding the primary drinking water standards. Due to the prevailing southerly direction of groundwater flow, these constituents and others have been transported past the southern extent of the Sanitary Landfill. Although it is not clear how far these contaminants have been transported, preliminary data from seepage samples indicate that only a minor amount, if any, of these COCs are detectable near the wetlands. The horizontal and vertical extent of any "plume" which may be emanating from the Sanitary Landfill is uncertain due to the lack of monitoring locations down gradient of the Sanitary Landfill in the horizontal or vertical dimensions.

The estimated vertical extent of the COCs in the vicinity of the landfill is based on the monitoring data from seven well clusters located immediately to the west and south of the Sanitary Landfill. The database is limited because the wells are new and were first sampled in the third quarter of 1991. Four of the clusters have wells screened in the lower part of the Steed Pond Aquifer, and seven wells are screened in the middle section of the Steed Pond Aquifer. Analytical data from water samples collected from the four wells screened in the lower part of the Steed Pond Aquifer do not provide evidence of any COCs above MCLs. Based on information available at this time, it appears that the COCs are not present in the deeper portions (Level B) of the aquifer directly below the Sanitary Landfill.

Analytical data from seven wells screened in the middle portion of the Steed Pond Aquifer indicate that some of the COCs are present at levels above MCLS. TCE has been detected above the MCL in three wells south of the Sanitary Landfill. Several other organics have been detected, such as 1,1-dichloroethane which was detected above the proposed MCL in LFW 59C and LFW 61C in the first quarter of 1992. However, the COCs do not appear to be as widely distributed and are typically present at much lower concentrations than in the upper portions of the saturated zone.

3.1.2 Seepage Sampling for COCs

Three seepage/wetland sampling events have occurred down gradient of the Sanitary Landfill in an effort to characterize contamination. A preliminary sampling event was used as a screening event. Only pH, specific conductance, and tritium levels were measured. This event was completed in February and April 1991. A second sampling event was performed during September and October 1991. These samples were analyzed for a full suite of contaminants including inorganics, organics, and radioactives. A third sampling event similar in scope to the second was performed during January and February 1992. As of the date of this report, the radioactive analytical results were not available for the third sampling event. A "Work in Progress" summary of the wetland sampling results from Dames and Moore dated July 6, 1992 was reviewed for this report.

The June, 1992 DRAFT Summary Report on the "Ecological Characterization of Upper Three Runs Creek" prepared by Metcalf and Eddy, Inc. was also reviewed for this report. Groundwater samples were collected from ten sampling stations in an effort to characterize potential contamination in the wetlands associated with UTRC. The sampling locations are shown on Figure 1.2 along with the wetland survey boundary. The gross alpha exceeded standards in 5 of 10 stations sampled with a maximum of 45 pCi/mL (± 12 pCi/mL) at SP-262. Tritium did not exceed the MCL at any stations; however, it did reach a concentration of

7.62 and 7.8 pCi/mL (± 0.5 pCi/mL) in sampling events 1 and 2 at SP-258. Strom and Kaback (1992) report that tritium in rainfall can be expected to range from 1.6 to 4.8 pCi/mL. The tritium concentrations at SP-258 may be attributable to tritium emanating from the Sanitary Landfill. From a statistical analysis, Dames and Moore (1992) found that the difference in tritium concentrations at the wetlands and the Sanitary Landfill background wells were not statistically significant.

Toluene was detected in the fall survey at SP-263 with a concentration of 40 ug/L and of 20 ug/L in the SP-254D duplicate sample. A value of 2 ug/L was reported for toluene at SP-263 in the spring sampling event. Dames and Moore (1992) found that toluene concentrations at the wetlands were statistically significantly higher than those measured from Sanitary Landfill background wells. Possible trace contamination at SP-263 by several chlorinated solvents such as 1,1,1-trichloroethane, 1,1-dichloroethane, and trichloroethene was also suggested by the data.

The results from the wetland sampling indicate that if the Sanitary Landfill contaminants have arrived at the wetlands they have arrived at trace concentrations. This statement should be tempered by the fact that the data base is limited historically and all concentrations are very near detection limits or background levels. Consequently, all conclusions based upon the data base are uncertain. However, there is some evidence for tritium occurring at SP-258 and also organic contaminants occurring at SP-263. Due to the proximity of the wetlands to the landfill and the sand and clayey sand aquifer matrix, it is not inconsistent to believe that the contaminants would have migrated to the wetlands by 1991. In fact, the regional A/M Area model predicts that the advective transport time between the center of the original landfill and the wetlands is approximately 3 to 4 years. If this is true, the environment would appear to be providing significant attenuation due to degradation, dilution from recharge, and dispersion.

Table 3.1 List of COCs with Selected Statistics from the Baseline Risk Assessment (Dames and Moore, 1992)

Constituent	MCL (ug/L)	Cmax (ug/L)	Cmean (ug/L)	Upper 95% Confidence Interval (ug/L)	Cmax/MCL
<u>Chlorinated Aliphatics</u>					
1,1,1-Trichloroethane	200	249.0	5.61	7.10	1.25
Trans1,2-Dichloroethene	70	754.0	35.12	45.32	10.77
Chloroethylene	2	400.0	6.54	10.20	200.00
Dichlorodifluoromethane	nr	75.0	36.50	57.40	
Tetrachloroethylene	5	564.0	9.97	14.59	112.80
Trichloroethylene	5	44.0	4.36	4.96	8.80
Trichlorofluoromethane	nr	580.0	29.25	36.70	
Dichloromethane	nr	103.0	9.34	10.89	
<u>Chlorobenzenes</u>					
1,4-Dichlorobenzene	75	99.0	14.70	19.43	1.32
Chlorobenzene	100	52.0	3.05	3.59	0.52
<u>Aromatics</u>					
Benzene	5	8.0	2.20	2.34	1.60
Ethylbenzene	700	52.6	4.62	5.46	0.08
Xylenes	10,000	28.0	6.04	7.80	0.00

Table 3.1 List of COCs with Selected Statistics from the Baseline Risk Assessment (Dames and Moore, 1992)

Constituent	MCL (ug/L)	Cmax (ug/L)	Cmean (ug/L)	Upper 95% Confidence Interval (ug/L)	Cmax/MCL
Metals					
Arsenic	50	38.0	2.47	2.91	0.76
Lead	50	50.0	3.32	3.69	1.00
Constituent	MCL (pCi/mL)	Cmax (pCi/mL)	Cmean (pCi/mL)	Upper 95% Confidence Interval (pCi/mL)	Cmax/MCL
Radioactive					
Tritium	20	101.0	7.58	8.46	5.05

MCL = Primary Drinking Water Standard, Maximum Concentration Limit
 nr = not reported

4.0 GROUNDWATER FLOW AND TRANSPORT MODELS FOR THE SANITARY LANDFILL AREA

Two separate models were constructed to evaluate flow conditions and transport of Contaminants of Concern (COCs) from the Sanitary Landfill to the Point of Exposure (POE). A two-dimensional areal model was developed as a tool to assess groundwater flow conditions on a local scale in the Sanitary Landfill area. This model was used to assess general flow directions and water level elevations, evaluate flow dynamics of the wetlands and UTRC, estimate groundwater travel times via particle tracking methods and investigate the effects of landfill capping.

A two-dimensional vertical cross-section flow and transport model was developed to address contaminant transport in the vertical dimension and to provide a means to simulate the complex degradation, sorption and biotransformation processes which affect transport of several COCs detected in groundwater near the Sanitary Landfill. Although the cross-section and areal models were developed for different purposes, both are based on the same conceptual model and therefore share fundamental characteristics. The model implementations are reviewed and compared in the following sections.

4.1 Conceptual Flow Model

The conceptual flow model developed for the Sanitary Landfill area was based on the geologic and hydrologic information presented in Section 2.0. The conceptual model combines the major elements of the complex hydrogeology into a framework which can be incorporated into a mathematical description of the flow system. In order to achieve this goal, it was necessary to simplify the hydrogeologic framework, however, the major characteristics of the flow system were maintained.

In both Sanitary Landfill local scale flow models, groundwater flow conditions in the Steed Pond Aquifer (Aadland et al., 1992) were assumed to be unconfined (see Figure 2.1). This formulation allows for proper treatment of groundwater elevations near UTRC and the associated wetlands as well as head changes due to landfill capping scenarios. In addition, the Tertiary aquifer (Steed Pond Aquifer) is treated as a single hydrostratigraphic unit. Modeling the Tertiary aquifer (Steed Pond Aquifer) as a single aquifer is consistent with the most current hydrostratigraphic cross sections constructed through the area, which suggest that very little confining clays exist in the Steed Pond Aquifer. Also, by not modeling the Cretaceous aquifers underlying the Meyers Branch Confining System (the Ellenton), we are assuming that cross-formational flow across the confining zone is a minor contribution to the Tertiary flow system. The A/M regional model (Beaudoin et al., 1991) models the Tertiary aquifer in the vicinity of the Sanitary Landfill as two aquifers representing the Upper and Lower Congaree (model layers 2 and 3, respectively) consistent with older stratigraphic terminology. Their results show that vertical flow from layer 3 to 2 is approximately 14 times as large as from layer 4 to 3 in the immediate vicinity of UTRC. This supports the assumption that in the vicinity of the Sanitary Landfill the dominant flow system can be modeled by a single-layer two-dimensional unconfined system. This is also consistent with most General Separations Area local-scale flow models where the Cretaceous is not modeled (GeoTrans, 1992).

Recharge is assumed to be constant over the entire model area. UTRC acts as a regional sink for the Tertiary aquifers north of UTRC in the vicinity of the Sanitary Landfill. In the local-scale models, UTRC is assumed to be a groundwater divide and, therefore, all water flowing southward through the Sanitary Landfill vicinity exits through UTRC or the associated wetlands. The flow dynamics of the wetland areas is a very complex process. During the site visit on August 28, 1992, the wetlands region between UTRC and the Sanitary Landfill was examined. The wetlands area was typically saturated to the ground surface and, in many areas, standing water was present. In some areas, groundwater seeping to the surface

would visibly flow toward UTRC in small rivulets. Because of the large areas of ponded groundwater between the Sanitary Landfill and UTRC, evapotranspiration also plays an important role in the overall loss of groundwater in addition to overland flow. In summary, it is apparent from the site visit and historical information that the wetlands area adjacent to UTRC typically serves as an important groundwater discharge area.

4.2 Two-Dimensional Areal Flow Model

4.2.1 Code Selection

The code MODFLOW was selected for simulation of groundwater flow in the Sanitary Landfill area. MODFLOW is a three-dimensional flow model which incorporates a block-centered finite-difference approach to solving the groundwater flow equations. Hydrogeologic layers can be simulated as confined, unconfined or a combination of both. External stresses such as wells, areal recharge, evapotranspiration, drains and streams can also be simulated. For this study, the finite-difference flow equations were solved using a pre-conditioned conjugate-gradient method. The code is written in FORTRAN 77 and has been well documented (McDonald and Harbaugh, 1988). The code is in the public domain, has been demonstrated to be well accepted by regulatory agencies, and has been extensively used to simulate groundwater flow under field conditions. MODFLOW was selected for the areal flow model in concurrence with WSRC because it was sufficient to address the issues of the study and because it could easily be used with MODPATH, the particle tracking code designed specifically for use with MODFLOW. In addition, the use of MODFLOW is consistent with other groundwater flow models at the SRS, including the A/M Regional Flow Model (Beaudoin et al., 1991).

The data used for constructing the flow model using the MODFLOW (McDonald and Harbaugh, 1988) code was gathered from various reports. The

hydrogeologic framework for the Sanitary Landfill area is discussed in Section 2.2. In the following section, details of the areal model construction and development and calibration data are presented. The steady-state calibrated MODFLOW data set for the areal flow model is included in hard copy in Appendix B of this report.

4.2.2 Model Grid and Boundary Conditions

The first concern in designing the finite-difference grid was to provide a fine enough grid spacing to adequately represent the wetland regions and the Sanitary Landfill area. Because this local-scale flow model was used to evaluate landfill capping scenarios, the mesh needed to be fine enough to accurately portray the landfill cover. In observing these criteria, the finite-difference mesh was constructed using a uniform grid spacing of 100 feet. This grid spacing is 10 times more dense than the A/M regional scale model. The local-scale model area is 10,000 by 10,000 feet, which yields a 100 x 100 grid with a total of 10,000 grid blocks. Because only one aquifer is considered in this model, only one layer of grid blocks is used. The thickness of each grid block is variable and is determined by the difference between the specified base elevation of the grid block and the elevation of the water table in that grid block.

As shown in Figure 1.3, the flow model grid was situated to take advantage of natural boundary conditions in the Sanitary Landfill area. The lower boundary is located along UTRC and its associated wetlands. The grid blocks below UTRC were considered inactive because it was not useful or necessary to model groundwater flow on the south side of UTRC. The upper right-hand side of the grid is located along wetlands associated with the flood drainage from Tim's Branch. Situating the grid boundaries in the wetlands provides an upper bound for the water level elevation in that area if we assume that the depth of water ponded on the surface is limited by the ground surface elevation. The left-hand side of the grid is situated along a line which is essentially perpendicular to the potentiometric contours from the calibrated A/M regional flow model (Beaudoin et

al., 1991). If the boundary was situated exactly perpendicular to the potentiometric lines, the left-hand border could be considered a no-flow boundary. Because some variation in flow direction does exist along the boundary, the boundary condition along the left-hand side was formulated to allow groundwater movement into or out of the model. The northern model boundary was simulated with prescribed head boundary conditions.

Estimates of aquifer base elevations were based on a hydrostratigraphic cross-section through the Sanitary Landfill area. The elevation of the aquifer base at UTRC is about 40 feet AMSL. The base of the aquifer slopes gently upward to the northwest with a maximum elevation of about 52 feet AMSL at the northern end of the model. As mentioned in Section 2.2.3, the clay layer (Meyers Branch Confining System) which underlies the Steed Pond Aquifer serves to substantially limit flow between the aquifers due to its relatively low vertical hydraulic conductivity. Analysis of the flow terms from the A/M regional model indicates that upward flow through the confining unit below the Steed Pond Aquifer is about 15 times smaller than the upward flow from the Steed Pond Aquifer to UTRC. The difference is even larger in other zones of the model domain. Therefore, the assumption that groundwater does not flow from lower aquifers through the Ellenton clays is justified for the objectives of this modeling study and the base of the system is assumed to be a no-flow boundary.

4.2.3 Hydraulic Parameterization

The wetlands associated with UTRC and Tim's Branch were simulated through the use of drains. The drain boundary condition provides a mechanism to regulate the upper elevation of the water table in the wetland regions. To incorporate drain conditions, a grid block is first specified to contain a drain. The grid block is then given a conductance value which indicates the grid blocks ability to transmit water to the drain. In addition, the elevation of the drain is provided. The drain condition can be implemented so that the groundwater elevation in a

grid block does not rise above a prescribed level. The depth of surface water in the wetland region of the model is limited because of surface drainage to UTRC, therefore the elevations of each drain in the Sanitary Landfill flow model were set approximately equal to the ground elevation. In addition, relatively high conductance values were specified for grid blocks containing drains so that the water level in the grid block would be maintained at the elevation of the drain. In this way, groundwater elevations would not be much higher than the ground surface elevation. Figure 4.1 illustrates the final spatial distribution of drain cells in the model.

Discharge to UTRC was simulated using general head boundary (GHB) conditions. Grid blocks that are defined as general head boundary conditions may serve as either sources or sinks depending on the direction of the hydraulic gradient. UTRC serves as a discharge point for groundwater. The GHB condition allows the user to specify the approximate head in the grid block. The estimate of the head in the grid blocks along UTRC were estimated according to the approximate ground surface elevation taken from topographic maps (3302 Series) and the approximate depth of water in the creek. Conductance terms in the GHB cells were adjusted during model calibration.

The prescribed head boundary conditions on the northern boundary of the model were initially based upon (1) calibrated heads for this area from the A/M regional flow model, (2) water elevations at Well P-30 near the B-Area, and (3) extrapolation of water levels from the Sanitary Landfill Area Northward. These heads were also adjusted during the calibration process because the regional model was calibrated to head data from a different time period.

As reported in Section 2.3.2, the potential range of groundwater recharge from infiltration of precipitation is 11 to 20 in/yr. Because the local scale model and the regional A/M flow model were calibrated with different sets of data, recharge was considered a variable in the calibration process. Recharge was

applied uniformly throughout the local model because no site-specific information was available on the distribution of this parameter.

With the boundary conditions specified, only two hydraulic parameters influenced flow significantly and were therefore considered calibration parameters. These were the hydraulic conductivity and the specific storage. For the steady-state model, the only explicit hydraulic parameter which influenced the flow model was the hydraulic conductivity of the Steed Pond Aquifer. As discussed in Section 2.2.3, field tests within the A/M Area yield values of hydraulic conductivity which range from 14 to 148 ft/day. An arithmetic average for the different sets of data is 32 ft/day. For calibration purposes, hydraulic conductivity was allowed to vary over the observed range. The only other hydraulic parameter needed to complete the analysis was the specific yield of the Steed Pond Aquifer. This parameter was needed for the transient model to evaluate the nature of different capping scenarios (discussed in Section 6).

4.2.4 Model Calibration

Model calibration refers to the process of estimating the best combination of model parameters which will adequately reproduce observed conditions. These parameters which are adjusted until the performance measure is satisfied can be referred to as calibration parameters. Several calibration parameters exist in the present model. These include:

- 1) hydraulic conductivity of the aquifer unit
- 2) groundwater recharge from precipitation
- 3) conductance and drain elevation of drain grid blocks
- 4) conductance and head elevation of GHB cells
- 5) prescribed head elevations along northern boundary

The following sections describe the approach to model calibration as well as the performance measures, the calibration criteria, and the calibration results. The two-dimensional areal flow model was calibrated to steady-state conditions.

4.2.4.1 Calibration Approach

The first step in the calibration of a groundwater flow model is to define the performance measures and to define the calibration criteria or calibration target. If a model predicts the performance measure to a degree that will satisfy the calibration target, the model may be considered calibrated. The present flow model calibration was calibrated to steady-state conditions which are assumed to be representative for groundwater elevations measured in monitoring wells at the Sanitary Landfill for the fourth quarter of 1991. However, the set of calibration criteria chosen is in fact dependent upon time as we have seen in Figure 2.7. Thus, the performance measure is a single realization of a transient process. During calibration, the model parameters were modified within specified ranges until the performance measure compared favorably with the observed measurements. This deterministic approach to calibration is very simple, but justified for this model because of the relatively simple nature of the model geometry and boundary conditions.

The primary performance measure for this model was the observed groundwater elevations in the monitoring wells at the Sanitary Landfill. Forty-four wells near the Sanitary Landfill were used to calibrate the flow model. Although fifty-seven wells exist at the Sanitary Landfill, only 44 wells are screened across the water table (or within a few feet of the water table). Water level elevations from the fourth quarter of 1991 were used as the performance measure. Table 4.1 lists the wells which were used as calibration points for the Sanitary Landfill model. Also listed are the well coordinates and the fourth quarter 1991 water level elevations, and the model grid block numbers corresponding to the well locations. Unfortunately, this model does not allow simple comparison of observed stream

flow measurements to simulated discharge values from UTRC. This comparison would be difficult because the model covers only a small portion of the total discharge region for UTRC and because the gaging stations are not positioned at the locations which permit proper measurement of stream discharge at the limits of the model area. For these reasons, streamflow is not considered a primary performance measure for the model.

In order to quantify when the performance measure has been satisfactorily reproduced, a calibration target must be defined. To define the calibration target for the heads required an error analysis on the head values. The measurement of head is performed with electric tape water level measuring devices. These types of instruments generally are quite accurate and may provide measurement error on the order of ± 0.02 feet (Woesner and Anderson, 1992). In addition, elevation surveys commonly accumulate errors which average around ± 0.1 feet (Woesner and Anderson, 1992). These two types of error can be classified as measurement errors. A third type of error is associated with the heads used for the performance measure. This error type can be classified as a scale error. Because most of the heads measured at the Sanitary Landfill are measured at the top of the water table, these heads are taken to be the model performance measure. However, it is known from monitoring well clusters that the heads in the water table aquifer can potentially vary by as much as one foot between the top and bottom of the aquifer. By only using the uppermost head measurements, and assuming they are applicable for the full saturated thickness, a scale error is introduced. We assume that this error is ± 1.0 feet. Therefore, the total potential error for the head measurements is on the order of ± 1.12 feet. The criteria was used as a guide during the calibration process.

One other measure to quantify calibration is to require that the head residuals (the difference in observed and simulated heads) be normally-distributed about a zero mean. It is also desirable to ensure that the residuals possess no significant directional trend. This would occur if a portion of the model consistently

over or under predicted the heads. Both of these requirements were addressed in the calibration process.

Because limited information was available concerning the distribution of hydraulic properties in the Sanitary Landfill area, it was assumed that the hydraulic parameters of the model should be initially homogeneous. Heterogeneity in hydraulic properties would be incorporated only if the model required it for calibration. Therefore, adjustments to hydraulic properties in the model during calibration did not take into account any potential spatial variability of the hydraulic properties unless calibration could not be obtained via homogeneous properties. The hydraulic conductivity of the aquifer unit was assumed to be uniform throughout the model domain. Likewise, the spatial distribution of groundwater recharge was also maintained at a uniform rate across the model domain throughout the model calibration process.

Initial conductance terms of the GHB and drain cells were based on the hydraulic conductivity of the aquifer unit and the geometry of the cells. Some cell conductances were adjusted in order to gain a better agreement with observed water level elevations within the model domain.

4.2.4.2 Calibration Procedures and Results

As stated in the previous section, initial calibration simulations were completed using uniformly distributed conductivity. The drain cells (used to simulate wetland effects) in the original model were not initially placed as close to the Sanitary Landfill as shown in Figure 4.1. However, calibration simulations indicated that without drain cells in these areas, water levels were too high and the calibration criteria at the southern extent of the Sanitary Landfill could not be met. The extension of the drain cells up gradient of the USGS mapped limits was consistent with field observations. The extension of the drain cells into the low area at the lower left hand side of the Sanitary Landfill was also necessary to attain

an adequate match of water levels in that area. It was also necessary to utilize drain cells along the upper right hand side of the model in order to maintain water levels at ground surface in that wetland area. Water level elevations and conductance terms for the drain cells were both adjusted during the calibration simulations.

General head boundary cells used to simulate discharge to UTRC were assigned a relatively large conductance value. Water level elevation in the GHB cells were assigned by estimating the streambed elevation and adjusting for the assumed one foot of surface water in the creek. Assuming all other conditions are identical, water level elevations in the GHB cells used to simulate UTRC would increase if the conductance terms for the cells were significantly decreased. Therefore, relatively high conductance values were maintained throughout the calibration simulations because it was assumed that the water levels near UTRC could be estimated with a relatively high degree of certainty.

GHB cells incorporated along the left-hand side and the right-hand side (between UTRC and the wetland area to the north) were initially assigned water level elevations and conductances which were estimated from the regional model. Adjustments were then made to reflect changes in the water level conditions due to the different period over which the calibration criteria were based. Prescribed head cells on the northern boundary of the model were initially estimated from the regional model but were increased during the calibration process to reflect increased water levels over the entire Sanitary Landfill area between model calibration periods.

Many simulations were completed to determine the range and combination of parameters which would result in an adequately calibrated model. The final hydraulic conductivity value estimated for the aquifer unit was 35 feet/day and the groundwater recharge value was 15 inches/year. The input files for the MODFLOW steady-state calibrated data set are presented in Appendix B. This

data includes the conductivity and heads values assigned to the general head boundaries and the drains cells. The sensitivity of the heads with respect to the calibration parameters will be discussed in Section 4.3.6.

Figure 4.2 shows the steady-state heads from the calibrated model. The head contours indicate that the low-lying area southwest of the lower left corner of the Sanitary Landfill (simulated by the drains) does affect the flow of shallow groundwater. This fact was verified during a field trip to the Sanitary Landfill site in August, 1992. The wetlands region in the upper right hand corner of the model also serves as a groundwater discharge area.

Steady-state head residuals are shown in Figure 4.3. Table 4.2 lists the residuals for each well. The minimum, maximum and mean head residual values are -1.9, 1.9, and -0.09 feet, respectively. The standard deviation of the head residuals is 0.82 feet. Heads were consistently over-predicted to the west of the Sanitary Landfill and under-predicted to the east. Because the great majority of the residuals were smaller than the error associated with the performance measure (calibration target), we did not feel it was warranted to add highly uncertain changes to the transmissivity field, thereby compounding uncertainty. Figure 4.4 illustrates the distribution of the head residuals around the median value of -0.1 feet. This figure confirms that the heads are normally distributed with a mean value near zero. This figure also shows that most head residuals are within the ± 1.12 foot calibration target specified in Section 4.2.4.1. Figure 4.5 shows the scattergram of simulated and observed heads. The higher heads are located at the northern end of the Sanitary Landfill and the lower heads at the south. The scattergram illustrates that there is not a directional trend in the errors of the simulated heads from north to south.

4.2.5 Water Budget Analysis

An inspection of the overall water budget for the model is sometimes helpful in assessing the major components affecting the flow system. Figure 4.6 illustrates the magnitude of all sources and sinks for the Sanitary Landfill model. The numbers presented in the illustration are in cubic feet per second. Each value represents the total sum of inflow (+) or outflow (-) from each boundary of the flow model. One interesting result of this analysis is that the sum of the discharge from the drain cells in the wetland area is very close to the amount of recharge over the entire model area. In addition, the lateral inflow from the prescribed head boundaries on the northern boundary is very near the magnitude of discharge from UTRC. The total discharge to UTRC and the associated wetlands is 4.7 cfs. This falls with the expected range UTRC flow gain of 3-15 cfs as described in Section 2.3.3. A more thorough discussion of the water budget with respect to capping scenarios is presented in Section 6.0.

4.2.6 Sensitivity Analysis

There is always some amount of uncertainty associated with the input parameters for flow models. Usually, many combinations of input parameters within a simplified mathematical model can provide a solution which meets the calibration criteria. Each one of these combinations is part of a "non-unique" set of potential solutions for the prescribed performance measure and calibration criteria. As the number and complexity of the performance measures increase, the set of potential parameter combinations which can satisfy the calibration criteria is reduced. If the set of performance measures is relatively limited (in our case being limited to one set of heads), the uncertainty in the calibrated parameter set will be increased. Therefore, it should be understood that there is uncertainty in the calibrated model. This section will describe the method employed to evaluate that uncertainty through a sensitivity analysis.

In this flow model, the parameters that are potentially the most uncertain are:

- 1) magnitude and distribution of the aquifer hydraulic conductivity
- 2) magnitude and distribution of the recharge to the groundwater system.

And possibly to a lesser degree, uncertainty is present in:

- 3) groundwater level estimates along UTRC and the conductance and head elevations used in the GHB cells to represent this component of the flow system
- 4) groundwater level in the wetland areas and the conductance and head elevations used in the drain cells

A number of simulations were performed in order to evaluate the sensitivity of the model results to changes in model input. It was assumed that the uncertainty in hydraulic conductivity and recharge were the most important to evaluate for this flow model. Several sensitivity simulations were performed calibrating heads with a range of recharge values from 10 to 20 in/yr. It was expected that a large combination of hydraulic conductivity and recharge pairs could satisfactorily calibrate the model. Figure 4.7 illustrates this point by a number of sensitivity simulations which come to this conclusion. The line on the graph indicates the combination of recharge and conductivity values which provided an adequate fit the calibration criteria. This graph clearly illustrates the non-unique nature of the calibrated model. Although some of the non-unique characteristics of the model could be reduced if more constraints could be applied to model output, much of the non-uniqueness is a result of the simplicity of the flow model. Flow through the system is steady and fairly uni-dimensional and, therefore, changes in hydraulic conductivity can be offset by modifications to the recharge.

From Figure 4.7 we can see that if we consider recharge uncertain but ranging from a low of 10 in\yr to a high of 20 in\yr, we can see that the corresponding uncertainty in hydraulic conductivity ranges from approximately 23 ft/day to 47 ft/day. This range of uncertainty in hydraulic conductivity implies that there is at least a factor of two uncertainty in flow velocities through the aquifer and in corresponding travel times. However, this range is far less than the observed range of hydraulic conductivities measured (14 to 143 ft/day).

In summary, there is some degree of uncertainty associated with the results of the calibrated flow model. However, this uncertainty does not restrict the use of the model results to accomplish the objectives of this analysis. The uncertainty in model parameters is overshadowed by the insight gained from the site-specific, quantitative evaluation of the flow dynamics of the Sanitary Landfill area.

4.3 Two-Dimensional Vertical Flow and Transport Model

In order to evaluate the potential importance of vertical flow in the vicinity of the Sanitary Landfill, INTERA constructed a vertical cross-section flow and transport model to aid in understanding the transport phenomena between the Sanitary Landfill and the POE. Specifically, a vertical cross-sectional model was built to examine horizontal and vertical flow components at the Sanitary Landfill and wetlands and to better understand the relationship between the Sanitary Landfill and discharge in the wetlands.

Given the large amount of uncertainties in the Sanitary Landfill database, an analytical approach to transport was first suggested for the Sanitary Landfill vicinity. However, an analytical modeling approach was found to not be satisfactory because of the following considerations: (1) transport of daughter product COCs through degradation cannot be easily incorporated; (2) dilution resulting from recharge is not considered; and (3) spatial and temporal variability of source are not easily incorporated. Initially, it was felt that not incorporating degradation into

the simulations would be conservative. However, degradation products are present at the Sanitary Landfill and it may be considered non-conservative not to account for vinyl chloride as part of a biotransformation pathway due to its high toxicity. Upon WSRC suggestion, degradation processes were incorporated into the calculations. Therefore, a cross-sectional flow model constructed with SWIFT-II was selected to perform the transport calculations. A cross-sectional model is more appropriate to yield estimates of peak concentrations in the groundwater system as compared to single-layer areal model which would yield vertically averaged concentrations. A full three-dimensional grid to model transport was considered unnecessary; selection of cross-sectional model is conservative since it does not include dispersion effects normal to the section.

4.3.1 Code Selection

SWIFT-II was selected to simulate flow and transport at the Sanitary Landfill because it provides the flexibility to model complex transport phenomena, including first-order chain decay. SWIFT-II is a block-centered finite-difference flow and transport code (Reeves et al., 1986). SWIFT-II solves the coupled equations for transient flow and transport and can also be used to examine the effects of variable density, heat transport, and brine or radionuclide transport. The code handles both fractured or non-fractured media. The fracture formulation is either discrete or a continuum approach (double porosity). The code is in the public domain, has been demonstrated to be well accepted by regulatory agencies, and has been extensively used to simulate groundwater flow and transport under field conditions.

4.3.2 Model Grid and Boundary Conditions

The cross-section flow and transport model was developed from the same conceptual model as the areal model. Figure 4.8 shows the location of the cross-section model within the Sanitary Landfill area and in relation to the areal flow

model. The cross-section model was aligned parallel to the Sanitary Landfill which is essentially parallel to the groundwater flow direction toward UTRC. The up-gradient side of the model was located 8000 feet from UTRC while the opposite side of the model was situated at UTRC. The top of the Meyers Branch Confining System served as the base of the model and was assumed to be a no-flow boundary. This assumption is consistent with the areal model as is the assumption that UTRC serves as a no-flow boundary. The aquifer is assumed to be homogeneous, unconfined and at steady-state conditions.

Figure 4.9 illustrates the model domain and finite difference grid for the cross-section model. The horizontal spacing of the grid blocks is equal to 100 feet across the entire model. The vertical grid spacing varies in relation to the expected height of the water table. The finite-difference grid consists of 80 blocks in the horizontal direction and 10 blocks in the vertical direction. As in the areal model, the base of the aquifer ranges from 40 feet AMSL at UTRC to about 52 feet AMSL at the left hand side of the model.

A prescribed head boundary condition was used to describe the lateral inflow on the left-hand side of the model. The prescribed head value at the top of the left side of the model was taken directly from the calibrated areal model. This prescribed head boundary condition was assumed to be hydrostatic. In other words, there was no vertical variation in the total potentiometric head along the left side of the model, which means that flow into the model was horizontal. The wetlands associated with UTRC were simulated using a special well option within the SWIFT code. The option allows the user to prescribe the water level elevation within a grid block containing "prescribed pressure" wells. The code then calculates what volume of fluid if any to be removed from the grid block in order to maintain the prescribed water level. If the water level falls below the prescribed level, the well will not remove any fluid from the grid block. In this fashion, the prescribed pressure wells act identical to drain cells within the MODFLOW code used to simulate flow in the two-dimensional areal model. In the cross-section

model, the water level elevation prescribed in the "prescribed pressure" wells used to simulate the wetlands was set equal to the manually estimated ground surface elevation within that grid block. It is assumed that the error associated with the estimate of ground-surface elevation is about ± 1.0 foot. The ground surface elevations were taken from the 3300 Series Topographic maps of the SRS.

4.3.3 Hydraulic Parameterization

As stated in Section 4, one of the purposes of the cross-section model was to delineate the degree of vertical dilution and dispersion within the aquifer between the Sanitary Landfill and the POE. Therefore, the cross-section model was patterned after the areal model as closely as possible in an effort to avoid variations between the models. Because the model parameters and boundary conditions for the cross sectional model closely mimic those of the areal model, the discussion of the origin of the parameters is not repeated here. For a more complete discussion, refer to Section 4.2.3.

The hydraulic properties used in the cross-section model are identical to those used in the areal model. The only explicitly defined hydraulic property that influences the steady-state flow model is the hydraulic conductivity of the aquifer unit, which was assigned a value of 35 feet/day. Several other physical parameters are required to perform transport calculations, however the discussion of these properties will be presented in Section 5. As in the areal model, the groundwater recharge value was assumed to be 15 in/yr.

4.3.4 Model Calibration

Because the areal model and the cross-sectional transport model have the same conceptual model, the model parameters and boundary conditions are identical. Again the 1991 fourth quarter heads were used as the performance measure. Because the cross-sectional model has no actual observation points

which fall exactly on the model grid, the universal kriging code AKRIP (Kafritsas and Bras, 1981) was used to estimate a calibration surface for the water table at an evenly spaced interval along the cross-section traverse through the Sanitary Landfill vicinity. In the regions to the south, water level measurements recorded as part of the Phase 3 Seepage Sampling Survey (Haselow, 1991) were used to constrain the head surface. In areas to the north of the Sanitary Landfill, heads from the areal model were used for a prescribed head boundary condition. The cross-sectional model required no calibration because the same hydraulic conductivity and recharge values as were determined from the calibrated areal model were implemented. The hydraulic conductivity and recharge used are 35 ft/day and 15 in/yr, respectively. Figure 4.10 shows the simulated or calibrated head surface relative to the observed kriged heads.

Although the cross-sectional model can be considered hydraulically calibrated consistent with the areal flow model, the cross-sectional model is not calibrated with respect to transport. Several uncertainties make calibration of the transport model not uniquely possible. The primary ones are: (1) the plumes cannot be fully characterized with the current sampling results; (2) the source inventory is unknown; and (3) no far-field sampling points provide firm evidence of contamination with a Sanitary Landfill COC.

All monitoring wells at the Sanitary Landfill are near-field wells located within or immediately adjacent to a large heterogeneous contaminant source. As a result, near field representations of the contaminant plumes such as contour plots probably represent superposed plumes emanating from multiple sources. For example, one cannot determine whether TCE concentrations detected south of the Southern Expansion of the Sanitary Landfill emanate from the original Sanitary Landfill or the Southern Expansion. If concentrations were monitored several source dimensions down gradient, the source heterogeneity would become less important. This is because the size and shape of the plume are less dependent on the size and shape of the source as the distance from the source increases.

Most simple deterministic transport conceptualizations and associated models assume that a representative elementary volume of the transport medium has been established. This typically occurs some number of source widths down gradient from a contaminant source.

Figure 4.1 Finite-Difference Grid and Boundary Conditions for Areal Flow Model

Figure 4.2 Head Contours from Calibrated Steady-State Areal Model

Figure 4.3 Head Residuals from Calibrated Steady-State Areal Model

Figure 4.4 Histogram of Head Residuals from the Steady-State Calibrated Model

Figure 4.5 Scattergram of Simulated versus Observed Heads for the Steady-State Calibrated Model

Note: Discharge units are in ft^3/sec

Figure 4.6 Water Balance Summary for the Steady-State Calibrated Areal Model

Figure 4.7 Relationship Between Hydraulic Conductivity and Recharge for the Calibrated Flow Model

Figure 4.8 Location of Cross-Section Model within the Sanitary Landfill Area

Figure 4.9 Cross-Section Model Finite-Difference Grid

Figure 4.10 Calibrated Water Level Elevation in Cross-Section Model

Table 4.1 Measured water level elevations for fourth quarter 1991 used as model calibration targets.

SLF Well No.	SRS Easting (feet)	SRS Northing (feet)	Measured Water Elevation (ft AMSL)	Modflow Column (J)	Modflow Row (I)
6	45241.2	84537.8	155.41	63	40
8	45415.3	84032.6	150.86	65	45
10A	45935.6	84369.6	153.89	70	42
16	45852.6	84748.9	157.34	69	38
17	45607.3	84602.8	156.42	67	39
18	45459.4	84577.3	156.02	65	40
19	45135.4	84817.2	156.56	62	37
20	45582.9	85262.6	160.80	66	33
21	46149.4	84178.3	151.82	72	44
22	46325.2	84223.6	152.40	74	43
23	46456.1	84251.3	152.64	75	43
24	46520.8	84544.2	155.31	76	40
25	46425.7	84967.2	158.29	75	36
26	45633.8	85654.6	163.15	67	29
27	45596.1	85839.1	164.02	66	27
28	45555.3	86079.6	165.34	66	25
29	45503.3	86372.7	166.90	66	22
30	45170.9	86318.4	167.24	62	22
31	44869.0	86262.2	166.80	59	23

Table 4.1 Measured water level elevations for fourth quarter 1991 used as
(Cont.) model calibration targets.

SLF Well No.	SRS Easting (feet)	SRS Northing (feet)	Measured Water Elevation (ft AMSL)	Modflow Column (J)	Modflow Row (I)
	44935.9	85836.8	164.56	60	27
33	44973.0	85633.8	163.18	60	29
34	45016.9	85409.5	161.77	61	31
35	45378.8	85237.4	160.52	64	33
36	45582.3	83535.5	146.44	66	50
37	45667.7	83113.2	143.06	67	54
38	46018.5	83172.3	143.85	71	54
39	46218.5	83213.1	144.09	73	53
40	46395.1	83248.8	143.64	74	53
41	46626.9	83304.9	145.55	77	52
42	46532.9	83776.2	147.73	76	48
43D	45244.5	86443.2	167.94	63	21
44D	45022.6	84524.4	155.49	61	40
45D	45142.0	84217.8	152.75	62	43
46D	45162.8	84054.0	151.35	62	45
47D	45167.9	83859.3	149.48	62	47
48D	45443.7	83877.5	149.34	65	47
55D	45189.3	83601.3	146.98	62	49
56D	45306.6	83398.0	145.49	64	52

Table 4.1 Measured water level elevations for fourth quarter 1991 used as
(Cont.) model calibration targets.

SLF Well No.	SRS Easting (feet)	SRS Northing (feet)	Measured Water Elevation (ft AMSL)	Modflow Column (J)	Modflow Row (I)
57D	45417.4	83190.2	143.89	65	54
58D	45700.2	82940.6	141.93	68	56
59D	46056.1	83000.1	142.94	71	55
60D	45722.3	82531.5	138.31	68	60
61D	46471.1	83089.1	143.42	75	55
62D	45922.9	82991.6	143.11	70	56

Table 4.2 Head Residuals from the Steady-State Calibrated Model

SLF Well No.	SRS Easting (feet)	SRS Northing (feet)	Head Residual (feet)
43D	45244.5	86443.2	-0.2
29	45503.3	86372.7	-0.1
30	45170.9	86318.4	0.0
31	44869.0	86262.2	0.2
28	45555.3	86079.6	-0.3
27	45596.1	85839.1	-0.1
32	44935.9	85836.8	0.1
26	45633.8	85654.6	-0.6
33	44973.0	85633.8	0.3
34	45016.9	85409.5	0.3
20	45582.9	85262.6	-0.6
35	45378.8	85237.4	0.0
25	46425.7	84967.2	-1.2
19	45135.4	84817.2	1.5
16	45852.6	84748.9	-0.9
17	45607.3	84602.8	-0.4
18	45459.4	84577.3	-0.5
24	46520.8	84544.2	-1.0
6	45241.2	84537.8	0.4
44D	45022.6	84524.4	0.6
10	45935.6	84369.6	-0.4
23	46456.1	84251.3	-0.4
22	46325.2	84223.6	-0.1
45D	45142.0	84217.8	0.9
21	46149.4	84178.3	0.0

**Table 4.2 Head Residuals from the Steady-State Calibrated Model
(Cont.)**

SLF Well No.	SRS Easting (feet)	SRS Northing (feet)	Head Residual (feet)
46D	45162.8	84054.0	0.7
	45415.3	84032.6	0.8
48D	45443.7	83877.5	0.7
47D	45167.9	83859.3	0.9
42	46532.9	83776.2	0.6
55D	45189.3	83601.3	1.6
36	45582.3	83535.5	0.8
56D	45306.6	83398.0	0.0
41	46626.9	83304.9	-0.6
40	46395.1	83248.8	0.5
39	46218.5	83213.1	0.0
57D	45417.4	83190.2	-0.6
38	46018.5	83172.3	-0.7
37	45667.7	83113.2	0.2
61D	46471.1	83089.1	-1.1
59D	46056.1	83000.1	-0.8
62D	45922.9	82991.6	-2.0
58D	45700.2	82940.6	-0.9
60D	45722.3	82531.5	-1.9

5.0 TRANSPORT PARAMETERIZATION

Table 3.1 lists the sixteen contaminants of concern. The list of COCs include two metals, one radioactive constituent, and thirteen organic constituents for transport calculations. Of the thirteen organic constituents in the COC list, many are known to undergo biotic or abiotic transformation in groundwater. In addition, most of these constituents are known to sorb to organic matter or mineral surfaces resulting in retardation of the contaminant. Tritium is not retarded during transport but does undergo first order decay. Lead and manganese are generally highly retarded due to sorption. However, in the presence of organic or inorganic colloids, lead and manganese can sorb to these and effectively move unretarded through the aquifer (i.e., facilitated colloids transport). In the following section the parameters controlling transport will be assigned values for the transport calculations. These parameters are: (1) effective porosity, (2) bulk density, (3) dispersivity, (4) free-water diffusion coefficient, (5) distribution coefficient, and (6) half life associated with first-order decay or degradation. The values used for these parameters are taken from the literature in most cases and references are provided.

5.1 Effective Porosity and Bulk Density

The effective porosity and bulk density values used in transport calculations were adopted from Looney et al. (1987). In this Environmental Information Document from the Savannah River Laboratory, estimates and recommendations are made regarding physical and geochemical parameters governing transport at the Savannah River Plant. In this document, an effective porosity of 0.2 is recommended for environmental assessment calculations. Likewise, Looney et al. (1987) recommend that a bulk density of 1.6 gm/cm^3 (100 lb/ft^3) be used for transport and assessment calculations.

5.2 Dispersivity and Free Water Diffusion Coefficient

The two dispersivities required for the transport model are longitudinal dispersivity and transverse dispersivity. The magnitude of longitudinal dispersivity versus scale of transport was recently the subject of a survey article of field-scale dispersion in aquifers (Gelhar et al., 1992). In this article a large percentage of all available studies on tracer tests and other field-scale dispersion studies were reviewed and in many cases re-analyzed. They presented a x-y plot of longitudinal dispersivity versus transport scale. For the Sanitary Landfill transport problem, the scale of transport is approximately 2000 feet from the centroid of the Sanitary Landfill to the first discharge blocks. Based upon the data plotted in Gelhar et al., (1992), an appropriate longitudinal dispersivity would be approximately 100 feet. This value was adopted for this study. This study also looked at the magnitude of transverse dispersivities relative to longitudinal dispersivities. The basic conclusion was that the y direction transverse dispersivity was generally one order of magnitude less and the z-direction transverse dispersivity was generally two orders of magnitude less than the longitudinal dispersivity. Based upon this, the z-direction transverse dispersivity used was chosen as 1 foot. This low value is consistent with the lack of contamination present at the lowest monitored zones (B screen levels).

Because diffusion is a part of the overall dispersion coefficient, an estimate of free-water diffusion coefficient must be made. Because of the high groundwater velocities in the vicinity of the Sanitary Landfill (500 ft/yr), the transport is advection dominated and consequently the distribution of contaminants will be largely insensitive to diffusion. Because most of the COCs are organic contaminants, the free-water diffusion coefficient was based on an average of the free-water diffusion coefficients for vinyl chloride, tetrachloroethylene, trichloroethylene, and pentachlorophenol. The range in free-water diffusion coefficient for these contaminants is from 1.1×10^{-3} to 5×10^{-4} ft²/day (11.3×10^{-6} to 5.4×10^{-6} cm²/sec). The average value was 7.4×10^{-4} ft²/day (8×10^{-6} cm²/sec).

To illustrate the dominance of mechanical advective mixing over diffusive mixing, an average mechanical dispersion coefficient was calculated for comparison to the average diffusion coefficient. The mechanical dispersion coefficient can be defined as the product of groundwater velocity and dispersivity. This yields a dispersion coefficient of $5 \times 10^4 \text{ ft}^2/\text{day}$. An effective molecular diffusion coefficient is equal to the product of the free-water diffusion coefficient ($7.4 \times 10^{-4} \text{ ft}^2/\text{day}$) and the media tortuosity (assume 0.15). This yields an average diffusion coefficient of $1 \times 10^{-5} \text{ ft}^2/\text{day}$.

5.3 Distribution Coefficients

The distribution coefficient (K_d) is equal to the mass of solute on the solid phase per unit mass of solid phase divided by the concentration of the solute in the solution. The use of the distribution coefficient here assumes that sorption is both fast and reversible and that the isotherm (the relationship between the sorbed and solute concentration) is linear. Distribution coefficients are usually measured in the laboratory under controlled conditions in batch studies or column studies. Because distribution coefficients are very site specific, their uncertainty is inherent when measurements have not been made with site-specific constituents, aquifer materials, and geochemical constraints. Based upon a lack of site-specific data at the Sanitary Landfill, distribution coefficients were estimated from the literature and empirical relationships. The retardation factor is a ratio of the groundwater velocity to the advective velocity of a given constituent and is a function of distribution coefficient (K_d), the bulk density of the aquifer (ρ_b), and the effective porosity of the aquifer (ϕ):

$$R_f = 1 + \frac{\rho_b K_d}{\phi}$$

The higher the K_d , the higher the retardation coefficient and consequently the slower the constituent advective velocity.

5.3.1 Organic COC Distribution Coefficients

For the organic COCs, the K_d is equal to the product of the fraction organic carbon (foc) present in the aquifer and the partition coefficient between organic carbon (K_{oc}) and water. Unfortunately K_{oc} is seldom known and must be estimated based upon experimentally derived regression equations describing the relationship between the octanol/water partition coefficient (K_{ow}) and K_{oc} . Values of K_{ow} for organic constituent are readily available in the literature. Schwille (1988) reports K_{oc} values for some organics which are included as COCs. For these constituents, the product of K_{oc} as reported by Schwille (1988) and the foc will define the distribution coefficient. Some COCs do not have a value of K_{oc} defined in Schwille (1988).

For these COCs, the K_{oc} will be estimated from the octanol-water partition coefficient using the Schwartzbach and Westfall (1981) formula as presented below:

$$\log K_{oc} = 0.72 \log K_{ow} + 0.49$$

The values of K_{ow} were taken from Looney et al. (1987) where reported and, otherwise, from other literature sources (Howard, 1989; Howard, 1990; Montgomery and Welkom, 1990) for those remaining constituents. The Schwartzbach and Westfall relationship is based upon experiments with chlorobenzenes and toluene and should be considered applicable to the organics present at the Sanitary Landfill. Looney et al. (1987) reported a regression equation similar in form to the one above; however, their relationship generally yields much lower estimates of K_{oc} and subsequently retardation. The Looney et

al. (1987) equation is based upon the behavior of chlorophenolic compounds in an alkaline aquifer and is not considered appropriate for the Sanitary Landfill. Table 5.1 reports the values of K_{oc} and K_{ow} used to estimate organic COC distribution coefficients.

Because the distribution coefficient is dependent upon the fraction organic content of the aquifer, site-specific estimates are desirable. A recent study by McMahon et al. (1990) reports foc measurements for Tertiary sands and clayey sands near Aiken, SC. His results indicate that foc would vary from a high of 80 to a low of 0.02 percent by weight. With such a range in potential foc values, the distribution coefficient can also vary greatly. Looney et al. (1987) proposes that a foc value of 1 percent by weight be used for the Savannah River Site. Through consultation with WSRC (Looney and Haselow, 1992), the smallest foc value of 0.02 percent by weight was chosen to be used.

Because the distribution coefficient of organics increases with increasing foc, WSRC considers it conservative to use the smallest foc value reported by McMahon et al. (1990). Table 5.1 reports K_d and R_f factors for the organic constituents of concern assuming a foc of 0.02 percent by weight. The retardation factor assumes an effective porosity of 0.2 and a bulk density of 1.6 gm/cm^3 after Looney et al. (1987). The estimates based upon the lowest value of foc are considered conservative as they result in the lowest estimates of retardation factor. For the conservative case (foc = 0.02 % by weight), the retardation factors range from 1.01 to 2.39.

5.3.2 Inorganic COC Distribution Coefficients

Tritium is generally considered a conservative species with a K_d of 0.0 and a R_f of 1.0. Looney et al. (1987) does report a K_d for tritium of 10^{-3} which results in a retardation factor which insignificantly deviates from unity. Therefore a K_d of 0.0 was adopted for tritium.

Looney et al. (1987) reports a range of K_d for lead to be from 10^0 to 10^4 with a proposed value of 10^2 . A K_d greater than 1 is generally considered immobile so we can see that one would not expect lead to transport away from the Sanitary Landfill over the time frame of the ACL demonstration. However, as Looney et al. (1987) point out, several factors can enhance the mobility (i.e., decrease the K_d) of a constituent and result in facilitated transport of the constituent. Metals such as lead can have a high affinity for organic or inorganic colloids which act as mobile sorbents thereby decreasing the effective retardation of the sorbed constituent. The complexation of metal ions to organic colloids is governed by several factors such as the specific ion and the pH and ionic strength of the water. Looney et al. (1987) proposed a method for dealing with facilitated transport by determining a facilitated transport fraction (FTF) and applying a low K_d (i.e., 0.001) to this fraction of the inventory of the given constituent. The method used by Looney et al. (1987) to determine the FTF cannot be applied to the Sanitary Landfill because there is great uncertainty in the original constituent inventories. For the sake of conservatism it was assumed that there is a sufficient fraction of lead available which could leach from the landfill over the entire period of the 50 year calculations. For conservatism it was also assumed that lead has an effective K_d of 0.001 mL/g. For comparison lead was also transported with an effective K_d of 100 mL/g.

For an estimate of the distribution coefficient for arsenic, we first looked at selected analytical results from the Sanitary Landfill to characterize the geochemical environment present at the Sanitary Landfill. Results of chemical analyses (Norrell et al., 1992) of monitoring wells LFW 29 and LFW 10A were evaluated using MINTEQ2 (Allison et al., 1991), an EPA chemical equilibrium speciation program, to assess the Sanitary Landfill geochemical environment. From an assessment of the geochemical conditions combined with soil gas survey results, the potential for microbial degradation of organic compounds was estimated. Well LFW 29 is a background well and is located at the northeastern corner of the Northern Expansion of the Sanitary Landfill (please see Figure 2.3). Well LFW 10A is located

near the middle of the southern one half of the original landfill. This well is one of the most contaminated wells associated with the Sanitary Landfill. The water compositions in these two wells were assumed to represent natural and contaminated waters, respectively. Table 5.2 summarizes several of the major differences between them.

The data indicates that the pH, ionic strength, and concentrations of Fe and TOC increase dramatically within the boundaries of the landfill. In addition, a soil gas survey of the Sanitary Landfill area revealed considerable methane production within the Sanitary Landfill boundaries (Pirkle and Masdea, 1990). The methane production below the landfill suggests that microbially-catalyzed transformation is occurring. Methanogenic conditions imply that concentrations of dissolved oxygen, nitrate, and sulfate should be non-detectable; however, nitrate and sulfate concentrations are reported to be approximately 3 and 4 mg/L, respectively. This suggests that LFW 10A samples may have been exposed to the atmosphere, improperly preserved or analyzed, or that there are discrete microenvironments with different redox characteristics within the subsurface. The increase in pH may result from the reduction of carbon dioxide to methane and/or dissolution of basic materials in the waste form. The increase in ionic strength and alkalinity suggests that the latter may have caused most of the rise in pH.

The charge balance error in equilibrium calculations for LFW 10A may have occurred because the Fe was assumed to be Fe(II). Fe(II) is much more soluble than Fe(III) at pH 6.5. However, the potential complexation of Fe(II) or Fe(III) with the dissolved organic matter was not incorporated in the equilibrium calculations. If the Fe is assumed to be complexed to dissolved organic matter or in the form of iron oxide colloids, the charge balance error reduces to less than 5 percent. These types of organic and inorganic colloids are often encountered where high dissolved organic carbon concentrations and changing water chemistries occur (McCarthy and Zachara, 1989). To obtain a more accurate representation of the

chemistry in this groundwater, these colloids could be characterized using ultrafiltration techniques.

Arsenic (III) is predicted to occur predominantly in the H_3AsO_3 (aq) form according to MINTEQA2 when dissolved. This should not complex strongly with organic matter. Wiley (1989) measured distribution coefficients for As (V) and As (III) in different pH soils. They measured over the pH range of 6.5 and the soils had a mean grain size of 0.2 mm which is consistent with the Sanitary Landfill borings (Serrine, 1992). From their results, a distribution coefficient of 20 mL/g at a pH of 6.5 for As (III) was interpolated. This corresponds to a retardation coefficient of 161. With such a high value, one would not expect to see As in the saturated zone at elevated concentrations. Because a site specific distribution coefficient for arsenic is not known, a conservative distribution coefficient of 0.001 mL/g was used. This is consistent with lead.

5.4 Dominant Degradation Processes and Associated Half-Life Values

Several of the COCS listed in Table 3.1 are halogenated aliphatic and aromatic compounds which are known to transform or degrade under certain geochemical and microbiological conditions through both biotic and abiotic transformations (Vogel et al., 1987). In the following section, transformation rates will be estimated for applicable COCs. A great deal of study is ongoing regarding the biotic and abiotic transformation of organic constituents and the potential for in-situ bioremediation is great. However, a great deal of uncertainty exists as to pathways, rates of transfer, and controls on biotransformation. Studies both at laboratory and field scales result in a large variation in results and many times contradictory results through comparison between studies (Barker and Wilson, 1992). The degradation model applied in these transport calculations assumes that the transformation processes described are first order decay reactions which are analogous to a radionuclide decay chain.

After the water chemistry was examined to determine the relevant pH and geochemical environment, the literature was reviewed for biodegradation half-life values of selected halogenated ethenes, ethanes, methanes, benzenes, and other aromatics under methanogenic conditions and reducing conditions.

Halogenated ethenes and ethanes are generally reductively dehalogenated step-wise under methanogenic conditions in a chain-decay process where the concentration of a lower member of the chain is dependent upon the member concentration one level above it. Figures 5.1 and 5.2 present the preferred pathways and estimated half-life values. In general, these compounds dehalogenate more slowly as the number of halogens decreases (Vogel et al., 1987). Most of the half-life values given are derived from published laboratory or field studies which simulate representative groundwater aquifers under methanogenic conditions. However, a value for vinyl chloride (VC) was not found in the literature for these conditions and was estimated. The estimate was based upon the half-life values observed for the other compounds in the chains and the anticipated increase in half life down the chain as is suggested in the literature (Vogel et al., 1987). The half life for vinyl chloride of 1000 days falls within the range of groundwater degradation half lives reported in Howard et al. (1991) of 56 to 2875 days. The 1000 day estimate is conservatively high relative to the anaerobic half life range reported in Howard et al. (1991) of 112 to 720 days.

The biotransformation of 1,1,1-trichloroethane under methanogenic conditions occurs very quickly under methanogenic conditions with a reported half life of 1 day or less (Vogel and McCarthy, 1987). However, with such a short half life one would not expect to observe concentrations of TCA in the groundwater at the Sanitary Landfill. Howard et al. (1992) reports that in groundwater under unacclimated non-methanogenic conditions that TCA has a half life that varies from 140 to 546 days. In an effort to be conservative, a half life of 343 days was adopted which is the middle of the reported groundwater range. This half life is

probably large and conservative in magnitude given the TCA concentration history at the Sanitary Landfill monitoring well network.

The biotransformation of tetrachloroethene (PCE) under methanogenic conditions involves step-wise reductive dechlorination to vinyl chloride (VC) and then mineralization to carbon dioxide (Vogel and McCarthy, 1985). Similarly, 1,1,1-trichloroethane (1,1,1-TCA) reductively dechlorinates step wise to chloroethane (CA) which mineralizes to carbon dioxide (Vogel and McCarthy, 1987). Although 1,1,1-TCA has been shown to degrade to 1,1-dichloroethane (1,1-DCA) abiotically, the half-life values for this reaction (approximately 1.9 years) and other elimination reactions are reported to be much longer than the biotransformation reactions under methanogenic conditions (Vogel and McCarthy, 1987). These calculations assume that the biotic pathway predominates at the Sanitary Landfill and the abiotic pathway is ignored.

Estimated half-life values for all the organic COCs are included in Table 5.3. No reaction paths are suggested for the last eight compounds -- the halogenated methanes, 1,4-dichlorobenzene, dichlorobenzene, benzene, ethylbenzene, and xylenes. The only two compounds among them which are likely to be involved in the same chain are dichlorobenzene and benzene, but this connection has not been well established in the literature for methanogenic conditions. The chlorofluorocarbons (CFCs) will most likely dechlorinate before they defluorinate, so the dichloromethane concentrations should not be affected by CFC degradation (Bouwer and Wright, 1988). Barker and Wilson (1992) report that chlorobenzene is not expected to biotransform under methanogenic conditions. Out of conservatism, chlorobenzene will be considered stable for the transport calculations.

5.5 Source Characterization

The Sanitary Landfill became operational in 1974 as a trench and fill landfill. By 1987 the original landfill was full and two expansion areas were created to the north and south. The Southern Expansion has been used since that time and is projected to fill up by 1993. The Northern Expansion has received no waste to date. The capacity of the landfill (original and Southern Expansion) has been estimated to be 799 acre-ft (WSRC internal memorandum).

5.5.1 Spatial and Temporal Nature of Source

The exact loading history of the Sanitary Landfill is not known because records of where and when old trenches were open were not kept for the greater part of the landfill history. From figures and memos provided by WSRC, a loading history has been created. The original landfill was active from 1974 until 1987. It is assumed that the Southern Expansion was loaded between 1988 and 1993. To quantify the temporal and spatial loading history in a simple manner, the original landfill and Southern Expansion were divided into quadrants and each quadrant was assigned a time interval over which loading is believed to have occurred. Figure 5.3 shows the Sanitary Landfill with the quadrants depicted as well as the loading time intervals. From the time periods one can derive the general direction of trenching in the landfill over its history. This interpretation of loading at the Sanitary Landfill is the basis used later in Section 5.5.3 to derive source loading functions.

5.5.2 Waste Types

The Sanitary Landfill receives a variety of solid waste from SRS construction sites, shops, offices, and cafeterias. The types of materials reported to have been disposed in the landfill are paper, plastics, rubber, wood, metal, cardboard, rags saturated with degreasers, empty containers, food wastes, pesticide bags, and asbestos in bags (Sirrione, 1992). The Sanitary Landfill is not supposed to receive

any radioactive wastes. However, it has been noted that tritium would not be detected by on-site scanners (B. Looney, personal comm.). This is substantiated by the presence of tritium in the groundwater at the site. Gross alpha and gross beta are also apparently elevated at the landfill suggesting sources of nuclides. Groundwater sampling does not currently analyze for potential nuclides.

Solvent rags containing industrial degreasers such as TCA and PCE have been deposited at the Sanitary Landfill over two known areas. These areas are the original landfill area and the southern half of the Southern Expansion (see Figure 1.2). Degreaser use at the A/M Area has been extensive with 3.7 million pounds of TCE used from 1952 until 1970, 8.7 million pounds of PCE used from 1962 until 1979, and 0.7 million pounds of TCA used from 1979 until 1982 (Christensen and Brendell, 1982). Although the exact type of degreaser associated with the solvent rags is not known, one possibility is that these rags were permeated with PCE and TCA given the chronology above. As previously discussed, dilute concentrations of degreasers have been detected in the groundwater at the site.

PM&A Consulting Engineers conducted a waste characterization over 25 weeks in 1990. Their findings are reported in Table 5.4. Over the course of their study they recorded an average waste deposition rate of 22.3×10^6 pounds/year. Unfortunately the amount of each waste type is based upon volume and not mass. For this reason, there is significant uncertainty in estimates of COC inventories at the Sanitary Landfill.

5.5.3 Source Loading and Implementation

The three types of information needed to approximate source loading were: (1) where in the landfill do we see evidence for a given COC emanating?; (2) when do the various portions of the landfill load (i.e., when do they start acting as a source for a given COC)?; and (3) what is the source inventory? In order to

approximate the spatial and temporal loading of the landfill, the original landfill and the Southern Expansion were divided into four quadrants each (see Figure 5.3). These quadrants are numbered 1 through 8. SWIFT II grid block numbers associated with each quadrant also listed in the figure. The grid blocks are numbered left to right in layers starting in the upper left corner of the grid for the cross-sectional flow and transport model illustrated in Figure 4.9.

To determine which quadrants of the landfill act as a source for a given COC, all POC wells and other wells interior to the Sanitary Landfill were associated with a given quadrant. Then using the bubble plots in Appendix A, all wells at the Sanitary Landfill were checked to see if a given COC was ever present at concentrations over one half of the Maximum Concentration Limit (MCL). For those constituents not having a primary drinking water standard (see Table 3.1), risk based surface water and organism standards provided by Dames and Moore were used. To determine these values, Dames and Moore calculated the threshold concentration for surface water for ingestion of water and eating of organisms based upon a risk of 10^{-6} . The standards for dichlorodifluoromethane, trichlorofluoromethane, and dichloromethane are 5.67, 5.67, and 4.65 ug/L, respectively. Because these values are so low, all potential source blocks must be considered sources. Therefore, for these 3 COCs the best-estimate source scenario is equivalent to the maximum source scenario. Using this approach a list of quadrants were tabulated for each COC which could conservatively be considered a best estimate source area. For the organic constituents, the soil gas survey was used to corroborate the areas identified with the method described above. For the maximum source scenario, all quadrants were considered source areas for all COCs with the exception of the degreaser solvents tetrachloroethylene and 1,1,1-trichloroethane. For these two COCs, quadrants 5 and 8 were not considered as source areas because these areas were not used for solvent rag deposition. There are no monitoring wells within quadrants 5 and 8, therefore the assumption that these quadrants are not rag source areas is uncertain. Table 5.4

lists the quadrants considered as source areas by COC for both the conservative best-estimate and the maximum source scenarios.

The quadrant system was also used to identify when individual transport model grid blocks were started as source contaminant source blocks. The sanitary landfill has been in operation since 1974 and is currently still active. The Sanitary Landfill has been receiving wastes since 1974 and operates as an unlined trench and fill operation. The original landfill site was 32 acres. This area reached its capacity around 1987 and a Northern Expansion of 16 acres and a Southern Expansion of 22 acres was added in 1987. The Northern Expansion has not been used for waste disposal to date and the Southern Expansion is expected to reach capacity by the end of 1993 to 1994. The exact location of each trench and its time of operation was not documented. However, some rough details are known regarding the landfill loading.

The eight quadrants making up the active Sanitary Landfill are shown in Figure 5.3. Also shown in parenthesis are the years over which each quadrant of the landfill was active. Each quadrant has a number of SWIFT-II grid block cells which are associated with it. Each SWIFT-II source block was associated with a year in which it became active which was based upon the active years of the quadrant and the direction of trenching in the given quadrant. Figure 3.3 lists the SWIFT-II grid block numbers associated with each quadrant. Table 5.6 lists the year each SWIFT-II grid block is activated as a source. For example, a COC with only quadrant 3 active as a source area for the conservative best-estimate source scenario would have blocks 32, 34, and 36 act as model source blocks turning on in 1987, 1986, and 1986, respectively.

So far the discussion has focused on the rationale for determining which model grid blocks act as source blocks and when they become active for a given COC. Assumptions regarding a given COC source inventory and how long each

source block can remain active as a source will be discussed in the following paragraphs.

Little is known regarding the potential inventory of the COCs in the Sanitary Landfill. Several studies of the Sanitary Landfill waste stream have been performed. The latest, and perhaps the most extensive study was performed by PM&A Consulting Engineers and the results are summarized in Table 5.4. Over a 25 week study period, the Sanitary Landfill received an estimated 10,726,848 lbs. of waste. This could be extrapolated to a yearly deposition rate of 2.23×10^7 lbs./year. Assuming this rate would be good from 1974 until 1993, this would mean that a total of 4.24×10^8 lbs. of waste could potentially be deposited in the Sanitary Landfill by closure.

Table 5.4 lists the relative percentages of each waste type by volume. Although this type of information is useful, it does not provide a great deal of insight into potential COC inventories. Because we do not know a COC inventory, we can not conservatively estimate when a given COC may stop emanating from the Sanitary Landfill. Because of this, the inventories are conservatively assumed to be sufficient to be constant throughout the period of institutional control (i.e., the year 2025 which is 30 years after the proposed capping in 1995). As an extra conservatism, the input concentration will be the highest observed point of compliance (POC) concentration (Table 3.1). Therefore, once a source block is turned on, it produces Sanitary Landfill effluent at a concentration equal to the maximum observed POC concentration for the entire simulation period.

In the implementation of SWIFT II, the actual input concentration is a relative concentration (C/C_0) set equal to unity. The simulation results from SWIFT II are in terms of relative concentrations. The results are simply denormalized to absolute concentrations in a procedure which will be fully described in Section 7.4.

With respect to source inventory questions, there are some bounding calculations which can be made. The assumption is that the maximum POC concentration will be input over the simulation period. In Table 5.7 the inventory necessary for inputting the maximum observed POC concentration for each COC is calculated assuming the total Sanitary Landfill Area and an infiltration rate of 15 in/yr. If we input the maximum observed POC concentration for all COCs, this would imply a total COC inventory of 30,000 lbs. If we assume that the potential total Sanitary Landfill waste mass deposited is 4.24×10^8 lbs, then the maximum transport calculation inventory is 0.007% by weight of the potential waste mass.

Since the tetrachloroethylene (PCE) source is thought to be a result of spent solvent rag deposition in the Sanitary Landfill, a potential PCE source inventory can be estimated. The "rag deposition area" (see Figure 1.2) is approximately 1.8×10^6 ft². Assuming the deposition of 1 inch of solvent rags over the entire rag deposition area, inventory of PCE can be estimated assuming a PCE saturation and porosity. Assuming a range in saturation from 0.2 to 0.01 and a range in rag porosity from 0.1 to 0.5, the variation in PCE inventory ranges from 1.5×10^4 lbs to 1.5×10^6 lbs PCE. The mass required to input the observed POC maximum concentration over 50 years (Table 5.7) is approximately 5500 lbs. Based upon this simple analysis, an assumed inventory of PCE of 5500 lbs is well below the potential inventory and, therefore, non-depletion of the source mass is a reasonable assumption.

The assumption to input the maximum observed POC concentration for each COC is considered conservative. Table 5.7 compares the total COC inventory required for inputting the average versus the maximum POC concentration. The difference in inventory is a factor of 18. It is considered likely that the average observed POC concentration represents a more probable long term release concentration as compared to the maximum observed concentration. Therefore, the source input assumptions for the Sanitary Landfill transport modeling must be considered conservative from a long term perspective. However, based upon

source uncertainties, the proposed use of the maximum observed COC concentration is conservative from a regulatory perspective.

	<u>$t_{1/2}$</u>	<u>References</u>
<div style="display: flex; align-items: center;"> <div style="border: 1px solid black; padding: 2px 10px; margin-right: 5px;">PCE</div> <div style="margin-right: 5px;">↓</div> <div style="margin-right: 5px;">B</div> </div>		
<div style="display: flex; align-items: center;"> <div style="border: 1px solid black; padding: 2px 10px; margin-right: 5px;">TCE</div> <div style="margin-right: 5px;">↓</div> <div style="margin-right: 5px;">B</div> </div>	70 days	Suflita et al. (1988)
<div style="display: flex; align-items: center;"> <div style="border: 1px solid black; padding: 2px 10px; margin-right: 5px;">DCE</div> <div style="margin-right: 5px;">↓</div> <div style="margin-right: 5px;">B</div> </div>	120 days	Barrio-Lage et al. (1988)
<div style="display: flex; align-items: center;"> <div style="border: 1px solid black; padding: 2px 10px; margin-right: 5px;">VC</div> <div style="margin-right: 5px;">↓</div> <div style="margin-right: 5px;">B</div> </div>	200 days	Barrio-Lage et al. (1986) Wilson et al. (1986)
<div style="display: flex; align-items: center;"> <div style="border: 1px solid black; padding: 2px 10px; margin-right: 5px;">CO₂</div> <div style="margin-right: 5px;">↓</div> <div style="margin-right: 5px;">B</div> </div>	1000 days	estimated

B = Biotic transformation (Vogel and McCarty, 1987; Vogel et al, 1987)

$t_{1/2}$ = Half Life

Figure 5.1 Transformation of PCE/TCE Under Methanogenic Conditions

B = Biotic transformation

A = Abiotic transformation to ethanol which is quickly transformed
biotically to carbon dioxide (Vogel and McCarty, 1987)

$t_{1/2}$ = Half Life

Figure 5.2 Transformation of 1,1,1-TCA Under Methanogenic Conditions

SANITARY LANDFILL

(years) Time Interval that the Landfill Area was Assumed Active

<u>Quadrant</u>	<u>Associated SWIFT-II Grid Blocks</u>
1	27, 29, 31
2	33, 35, 37
3	32, 34, 36
4	28, 30
5	39, 41
6	43, 45, 47
7	44, 46
8	38, 40, 42

Figure 5.3 Estimated Loading History for the Sanitary Landfill by Quadrant

Table 5.1 Sorption Parameters for Organic COCs

Constituent	Koc Schwille (1988)	log Kow from Literature	Literature Citation	Kd (mL/gm)	Rf
Chlorinated Aliphatics					
1,1,1-Trichloroethane	152			0.030	1.24
Tetrachloroethylene	324			0.065	1.52
Trichloroethylene	126			0.025	1.20
Trans 1,2-Dichloroethylene	59			0.012	1.09
Chloroethylene	NR	0.9	1	0.003	1.02
Dichlorodifluoromethane	NR	2.16	4	0.022	1.18
Trichlorofluoromethane	159			0.032	1.25
Dichloromethane	8.8			0.002	1.01
Chlorobenzenes					
Chlorobenzene	330	2.84	2	0.066	1.53
1,4-Dichlorobenzene	NR	3.4	1	0.173	2.39
Aromatics					
Benzene	NR	2	1	0.017	1.14
Ethylbenzene	NR	3.15	2	0.115	1.92
Xylenes	NR	3.12	3	0.109	1.87

Citation 1: Looney et al. (1987)

Citation 2: Howard (1989)

Citation 3: Howard (1990)

Citation 4: Montgomery and Welkom (1990)

NR = Not Reported

Koc = Organic Carbon and Water
Partition Coefficient

Kow = Octanol and Water
Partition Coefficient

Kd = Distribution Coefficient

Table 5.2 Water Analyses from Monitoring Wells LFW-29 and LFW-10A, Sampled First Quarter, 1991.

	LFW 29	LFW 10A
pH	4.9	6.5
Fe	0.0223	98.1
TOC (mg L ⁻¹)	1.0	43.6
Alkalinity (mg L ⁻¹)	0	214
Charge Balance Error	< 5 %	37.5 %
Ionic Strength (M)	0.00018	0.0043

Table 5.3 Biotransformation Half-Life Values for Organic COCs under Methanogenic Conditions.

Organic Compound	Half-Life (days)	Reference
Tetrachloroethylene	70	Suflita et al. (1988)
Trichloroethylene	120	Barrio-Lage et al. (1988)
Trans-1,2-Dichloroethylene	200	Barrio-Lage et al. (1986) Wilson et al. (1986)
Vinyl Chloride	1000	estimated
1,1,1-Trichloroethane	343	Howard et al. (1987)
Trichlorofluoromethane	730	Howard et al. (1991)
Dichlorodifluoromethane	730	Howard et al. (1991)
Dichloromethane	112	Howard et al. (1991)
1,4-Dichlorobenzene	730	Howard et al. (1991)
Chlorobenzene	NA	Barker and Wilson (1992)
Benzene	1100	Barker and Wilson (1992)
Ethylbenzene	1380	Barker and Wilson (1992)
Xylenes	475	Barker and Wilson (1992)

Table 5.4 Relative Percentage of Waste Type by Volume and Cumulative Waste Weight, Sanitary Landfill, for a 25 Week Period in 1990 (After PM&A Consulting Engineers, 1990).

	Percent by Volume
Paper	20.38
Cardboard	28.59
Lumber	0.17
Aluminum	2.25
Other Metals	5.28
Glass	0.74
Plastic	21.62
Cafeteria	7.34
Other	13.61

Cumulative Weight = 10,726,848 lbs

Table 5.5 Active Source Areas Defined by Quadrant for the Best-Estimate and Maximum Source Scenarios

Constituent	Conservative Best-Estimate Source Scenario	Maximum Source Scenario
Chlorinated Aliphatics		
1,1,1-Trichloroethane	2,3,6,7	1 ,2,3,4,6,7
Tetrachloroethylene	2,3,6,7	1 ,2,3,4,6,7
Trichloroethylene	2,3,6,7	1 ,2,3,4,6,7
Trans 1,2-Dichloroethylene	2,3,6,7	1 ,2,3,4,6,7
Chloroethylene	2,3,6,7	1 ,2,3,4,6,7
Dichlorodifluoromethane	1 through 8	1 through 8
Trichlorofluoromethane	1 through 8	1 through 8
Dichloromethane	1 through 8	1 through 8
Chlorobenzenes		
Chlorobenzene	2	1 through 8
1,4-Dichlorobenzene	2,5	1 through 8
Aromatics		
Benzene	2,3,5	1 through 8
Ethylbenzene	2,3	1 through 8
Xylenes	2,3	1 through 8
Metals		
Arsenic	1,2	1 through 8
Lead	2,3,4	1 through 8
Radioactive		
Tritium	2,3	1 through 8

Table 5.6 Timing of Source Loading by SWIFT-II Grid Block Number.

SWIFT Grid Number	Year Source Begins
27	1975
28	1988
29	1977
30	1987
31	1979
32	1987
33	1981
34	1986
35	1983
36	1986
37	1985
38	1994
39	1993
40	1993
41	1992
42	1992
43	1988
44	1991
45	1989
46	1990
47	1989

Table 5.7 Assumed Inventory Based upon Inputting Concentration Over Entire Sanitary Landfill Area, Infiltration Rate = 15 in/yr.

Contaminant	Mean Conc ug/L	Max Conc ug/L	Mass Required (lbs)	
			Mean Conc. 50 YRS Total SLF	MAX Conc. 50YRS Total SLF
1,1,1-Trichloroethane	5.6	249.0	54.7	2426.7
Tetrachlorethylene	10.0	564.0	97.2	5496.5
Trichloroethylene	4.4	44.3	42.9	431.7
Trans1,2-Dichloroethane	35.1	754.0	342.1	7348.2
Chloroethylene	6.5	400.0	63.3	3898.3
Dichlorodifluoromethane	36.5	74.6	355.7	727.0
Trichlorofluoromethane	29.3	580.0	285.5	5652.5
Dichloromethane	9.3	103.0	90.6	1003.8
1,4-Dichlorobenzene	14.7	99.0	143.3	964.8
Chlorobenzene	3.1	52.0	30.2	506.8
Benzene	2.2	8.0	21.4	78.0
Ethylbenzene	4.6	52.6	44.8	512.6
Xylenes	6.0	28.0	58.5	272.9
Arsenic	2.5	37.6	24.4	366.4
Lead	3.3	40.0	32.2	389.8
Sum =			1686.8	30076.1

6.0 EVALUATION OF CAPPING SCENARIOS

The local-scale modeling in the Sanitary Landfill vicinity has many objectives, one of which is to quantify the hydraulic responses in the vicinity of the Sanitary Landfill resulting from various capping scenarios. WSRC will install a RCRA geosynthetic cap at the Sanitary Landfill as part of the landfill closure. The proposed RCRA cap to be constructed at the Sanitary Landfill may cover an area as large as 52 acres. The cap will consist of a foundation layer, a gas vent layer, a bentonite mat overlain by a flexible membrane liner (FML), a drainage layer, a geotextile filter, and a vegetative layer including 6 feet of topsoil. The cap, including the foundation layer, will exceed 5 feet in thickness over the existing trench area and will taper off with a 1 to 4 slope past the existing trench area.

Because the water table is shallow in the vicinity of the Sanitary Landfill, it is thought that the bottom of the waste trenches may have in the past been saturated by groundwater in northern areas of the landfill in times of above average precipitation. Modeling was performed for various capping scenarios to examine the net groundwater head decrease expected to occur under the Sanitary Landfill. These calculations were performed to assist WSRC closure staff in determining the future potential of water levels rising up to the bottom of the waste trenches under capped conditions.

The steady-state calibrated areal flow model was used as the basis for modeling the areal head changes resulting from capping. The steady-state calibrated heads were the initial conditions for the capping modeling. The cross-sectional flow and transport model was used to examine the effects of capping on constituent concentrations down gradient from the landfill. The results of these analyses are presented in the following sections.

6.1 Hydraulic Response to Capping

Two different capping scenarios were proposed by WSRC to be evaluated. These two scenarios differ in the area covered by the cap. Scenario 1 considers capping of the original landfill area and the entire Southern Expansion. This area corresponds to the entire area which is known to have received waste at the Sanitary Landfill. Scenario 2 considers capping only those areas which are estimated to have contained solvent-saturated rags in the area commonly regarded as the solvent rag deposition area. Figure 1.2 shows the area commonly regarded as the solvent rag deposition area prior to November 20, 1992. It was discovered that a small amount of rag deposition did occur in the northern half of the Southern Expansion (letter, Roberts to Wilson. ESH-ENV-92-360, November 20, 1992). This discovery was too late for the modeling contained in this report. Because the deposition in the northern half of the Southern Expansion is thought to be minimal, it is expected that the neglect of their presence would not alter model conclusions (memorandum, T.G. Jordon to A. Suer, February 16, 1993).

For each cap scenario listed above, a set of four simulations were completed based upon a range of cap efficiencies which were 100%, 95%, 90% and 85%. These cap efficiencies were provided by WSRC for consideration. The cap efficiency is defined as the percent of total precipitation which infiltrates through the Sanitary Landfill cap. Infiltration rates through the caps are based upon the long term SRS site average precipitation of 48 in/yr (Fenimore and Hooker, 1977). Therefore, cap efficiencies of 100%, 95%, 90%, and 85% correspond to cap infiltration rates of 0, 2.4, 4.8, and 7.2 in/yr, respectively.

6.1.1 Areal Flow Model

The steady-state calibrated areal flow model was used to examine the areal effects upon the hydraulic heads and particle travel times caused by the proposed capping scenarios. The calibrated model (see Section 4.2.4) was used as the

initial condition for the transient modeling of cap effects on heads. A transient model requires specification of a storage term for the aquifer unit. The specific yield is the storage term for an unconfined aquifer and is generally equal to the porosity of the aquifer unit. For these calculations the specific yield was assumed to be equal to the medium porosity of 0.2.

Cap conditions are simulated by reducing recharge over the capped portion of the Sanitary Landfill by the appropriate amount based upon the modeled efficiency. The remainder of the model area still receives the recharge rate of 15 in/year as determined by the areal steady-state calibrated flow model. Assuming that the precipitation averages 48 in/year, the natural land cover in the Sanitary Landfill local-scale model area has an efficiency of 65% without a cap.

It should be noted that "capping" the landfill (by reducing the volumetric recharge in the model) will instantaneously affect water levels in the numerical simulations because the model does not consider flow in the unsaturated zone. In reality, there will be some lag time associated with the decrease in groundwater recharge due to the fluid storage in the unsaturated zone. After capping, the actual amount of recharge will decrease with time until it approximately equals the amount of fluid infiltrating the landfill cap.

For capping Scenarios 1 and 2 the maximum hydraulic responses resulting from capping were evaluated with steady-state simulations. Through a steady-state analysis the maximum effects from capping can be examined independent of time. Both the change in head and the change in the model volumetric flow balance were examined. In addition, changes in particle track trajectory and travel times to discharge points were evaluated for the various capping scenarios. The time dependent transient effects of capping were also evaluated. For the transient analysis, the capping conditions were implemented to evaluate the time dependent response in water levels and the time taken to reach a new steady-state hydraulic condition under capped conditions. In addition, a transient simulation was

performed to evaluate the transient head responses under and around the Sanitary Landfill resulting from extreme transient precipitation events under capped conditions. This analysis was done to assist WSRC closure staff in predicting the potential range of water levels under the Sanitary Landfill under capped conditions. This analysis is meant to aid WSRC staff in the determination of whether water levels could rise to the height of the trenches under high recharge events.

6.1.1.1 Transient Simulations

Capping Scenario 1 maximizes cap area because it is assumed that the entire original landfill and the entire Southern Expansion are capped. Capping Scenario 1 under the assumption of 100% cap efficiency eliminates recharge to the entire Sanitary Landfill area that is known to have received waste. For this reason, Scenario 1 assuming a cap efficiency of 100% was chosen to illustrate the transient effects of capping on the heads in the vicinity of the Sanitary Landfill because it will produce the largest effects. In reality, the cap efficiency for any geosynthetic cap will be less than 100%. It should be noted that only head difference maps are presented for each capping scenario because visual differences in head for conditions of no-cap and cap conditions are not discernable with a practical contour interval.

The transient hydraulic response to capping for Scenario 1 assuming 100% cap efficiency is shown in Figure 6.1. with a series of hydrographs. In Figure 6.1, head change in feet is plotted as a function of time since capping for three different locations underneath the Sanitary Landfill. As one might expect, the net effect of capping on the water levels underneath the Sanitary Landfill is to cause heads to decrease as a result of decreased vertical recharge resulting from precipitation and infiltration. The magnitude of head decrease is a function of location beneath the Sanitary Landfill. The hydrographs shown in Figure 6.1 are from three grid blocks in the areal flow model; one is located at the northern end of the Northern Expansion of the Sanitary Landfill (denoted hydrograph A), one is in the middle of

the original portion of the Sanitary Landfill (denoted hydrograph B), and one is located at the southern end of the Southern Expansion of the Sanitary Landfill (denoted hydrograph C). In this simulation, the initial conditions are the steady-state calibrated model heads. The only variable that changed from the steady-state model was the recharge rate over the Sanitary Landfill. These conditions were simulated for a five year time period to evaluate the overall effects of capping on water levels near the Sanitary Landfill.

The largest head decrease under capped conditions occurred in the middle of the Sanitary Landfill (hydrograph B). The northern and southern sections of the Sanitary Landfill experienced less head decrease. The majority of the decline in water levels occurs in the first year after capping, and under the simulated conditions, the system reaches quasi steady-state conditions after about 2 years. Figure 6.2 shows an areal view of the head decrease at 5 years after capping for Scenario 1 under 100% cap efficiency conditions. The maximum head decrease is a little over 0.6 feet and is centered around the centroid of the capped area. The area of decreased heads is limited to the vicinity of the Sanitary Landfill.

Another simulation was performed to evaluate the response of the water table elevation to temporal changes in recharge. This was accomplished by implementing a highly variable sequence of precipitation (and subsequent recharge) events which might occur at the Sanitary Landfill. A transient data set was assembled which incorporated recharge conditions that were based on the measured rainfall near the Sanitary Landfill site over a 2 year period. The primary objective of the simulation was to examine the maxima and minima of heads observed under the Sanitary Landfill and to examine the effects of the cap in "damping" water level fluctuations under the Sanitary Landfill in comparison to areas in the aquifer not under the cap. It was assumed that the initial heads for the simulation were the results of the steady-state simulation incorporating capping Scenario 1 (95% efficiency). For definition of a highly variable precipitation event that may occur in the vicinity of the Sanitary Landfill, it was assumed that the same

rainfall sequence which was observed from 1990 through 1991 at the A/M Area approximately 3 miles north of the Sanitary Landfill was observed over the entire Sanitary Landfill model area. Figure 6.3 shows a bar graph of the precipitation observed at the A/M Area over the period from January 1990 through December 1991 and Table 2.2 tabulates the values. Capped conditions were implemented over the Sanitary Landfill according to capping Scenario 1 assuming a 95% cap efficiency. Transient recharge values for the simulation were assumed equal to 31% of the measured rainfall. This model does not explicitly model the unsaturated zone. As a result, the assumption that recharge is a fixed percentage of precipitation may underestimate infiltration during extended wet time periods. Figure 6.3 illustrates the estimated recharge from January 1990 through December 1991 for the Sanitary Landfill vicinity based upon the rainfall data and a constant relationship between precipitation and recharge. Recharge over the Sanitary Landfill model area was changed every 30 days in a stepwise manner as shown in Figure 6.3. The simulation started at steady-state conditions for Scenario 1 under an assumed cap efficiency of 95%. The transient portion of the simulation begins in January 1990 which corresponds to the initial steady-state conditions. After the initial two-year period which implemented the transient recharge conditions, it was assumed that rainfall (and thus recharge) once again continued at the "average" values shown in Figure 6.3.

Figure 6.4 shows the location and index number of the grid blocks within the local Sanitary Landfill areal model which were monitored during the transient simulation. The water level response or hydrographs from the monitored grid blocks is shown in Figures 6.5 and 6.6. Figure 6.5 illustrates changes along the axis of the Sanitary Landfill running parallel to the mean groundwater flow direction (north to south) and Figure 6.6 shows the head changes along an axis normal to the mean groundwater flow direction (east to west). Also plotted on Figures 6.5 and 6.6 is the regional recharge rate in inches per month imposed upon the Sanitary Landfill areal model as a boundary condition in the areal model excluding the Sanitary Landfill cap area. By comparing the hydrographs in Figures 6.5 and

6.6 to the recharge time series plot included on these figures it is easy to see the correlation between the regional recharge and the change in water levels underneath and around the Sanitary Landfill. As shown in Figure 6.4, Block 7 is located in the center of the Sanitary Landfill. Figures 6.5 and 6.6 indicate that water level increases are typically smaller in areas interior and underneath the cap during high recharge events as compared to locations just outside the Sanitary Landfill. However, during low recharge events, the water level seems to decrease more in the center of the Sanitary Landfill than at locations outside the landfill. In general, the overall water level response to the fluctuation in recharge tends to mimic the changes that are observed when comparing steady-state heads under capped and no-cap conditions. In other words, the water level fluctuations which might be expected under transient recharge conditions with a cap can be estimated by superimposing the steady-state decrease in water levels due to capping on the transient water levels throughout the region. For example, if heads immediately underneath the Sanitary Landfill have historically ranged from 156 to 146 ft above mean sea level (amsl), then under cap Scenario 1 conditions assuming 100% cap efficiency, one could expect heads to range over 155.4 to 145.4 ft amsl.

6.1.1.2 Steady-State Simulations

It was discovered in the transient simulations that the flow system reaches quasi steady-state conditions relatively quickly (within 1 to 2 years) after "cap conditions" are implemented. For this reason, it is sufficient to compare effects of different capping scenarios under steady-state conditions. Comparison of Figures 6.2 and 6.7 clearly illustrates this point. Figure 6.2 shows the head decrease under transient conditions five years after a cap 100% efficient Scenario 1 cap was implemented. Figure 6.7 shows the steady-state conditions for the same scenario. There is no discernable difference in the head difference maps. For this reason, the remaining simulations incorporating different capping scenarios and

efficiencies were modeled under steady-state conditions to quantify maximum hydraulic responses.

Figures 6.8, 6.9 and 6.10 show areal head decrease maps for Scenario 1 assuming capping efficiencies of 95%, 90% and 85% respectively. Scenario 1 assumes that the cap covers the entire original and Southern Expansion portions of the Sanitary Landfill. As expected, the decrease in heads due to capping becomes less significant as the efficiency of the cap decreases. The areal extent of decreased heads is also smaller under less efficient capping scenarios, as noted by the decreased size of the 0.2 foot contour.

Figures 6.11, 6.12, 6.13 and 6.14 illustrate the areal head decreases at steady-state conditions resulting from capping Scenario 2 under assumed cap efficiencies of 100%, 95%, 90% and 85%, respectively. Scenario 2 assumes that the cap covers the entire original portion of the Sanitary Landfill and the lower one-half of the Southern Expansion which is thought to have received solvent rags. As anticipated, the magnitude of the decrease in the heads for Scenario 2 are less than those simulated for Scenario 1 because the area over which recharge is diminished is smaller in Scenario 2. The areal extent of head decreases resulting from capping are smaller for Scenario 2 as compared with Scenario 1. Also, the point of maximum head decrease is centered slightly further south for Scenario 1 for a given cap efficiency when compared to the results from Scenario 2. This is the result of the northern one-half of the Southern Expansion being not capped in Scenario 2.

The overall change in heads and resulting flow system conditions due to capping the Sanitary Landfill is minimal. Table 6.1 summarizes the overall volumetric flow balance for the various flow boundary conditions for each of the eight different capping simulations. A complete discussion of the various model boundary conditions and their specified use can be found in Section 4 of this report and in Figure 4.1. In review, the Prescribed Head Boundary Conditions

(PHBs) are specified on the northern boundary of the flow model and represent up-gradient groundwater flow into the local-scale model. The recharge boundary condition is specified to the top of the model grid and is representative of infiltration due to precipitation. The Drain Boundary Conditions represent the model wetlands bordering UTRC and Tims Branch to the east. The General Head Boundary Conditions (GHBs) are specified along the western border of the model which is dominantly a no-flow boundary and also define UTRC. Most changes in the GHBs volumetric flow are a result of flow changes through the UTRC nodes.

Table 6.1 shows the volumetric flow rates attributed to each set of boundary conditions and also tabulates the relative percentage difference (RPD) in the flow rates for each simulation as compared to the calibrated steady-state volumetric flow rates assuming no-cap conditions. The RPD is defined as:

$$RPD = \frac{V_c - V_{nc}}{V_{nc}} \times 100$$

where: V_{nc} = Flow rate (ft³/day) of model component under steady-state conditions (no capping)

V_c = Flow rate (ft³/day) of model component under steady-state conditions (with cap of specified efficiency)

For Scenario 1, the RPD in recharge for the entire model ranged from -1.5 to -2.8, depending on the capping efficiency. The prescribed head condition implemented on the northern boundary of the model (representing regional underflow) is the only source for groundwater except for vertical recharge. Because capping creates a decrease in heads in the near vicinity of the Sanitary Landfill, flow through the northern PHBs increases in response to the increased hydraulic gradients directed towards the Sanitary Landfill area. Recharge obviously

decreases over the model area as a result of increased cap efficiency and the relative percent difference ranges from -1.45 to -2.79. Notice that the increase in flow through the PHBs does not match in magnitude the decrease in recharge resulting from capping. This is manifested in the net decrease in heads over the vicinity of the Sanitary Landfill as a result of capping. The GHBs which represent UTRC also have a reduced volumetric flux with a RPD ranging from -0.17 to -0.33. The changes at the GHBs is less than for the other boundaries. The overall difference in flow rate in and out of the model for the no capping case and capping Scenario 1 was less than -1.4% regardless of the simulation cap efficiency and ranged from -0.71 to -1.4. For Scenario 2, the general flow balance behavior is similar to Scenario 1 but with smaller differences as compared to the steady-state no cap simulation.

6.2 Effects of Capping on Contaminant Transport

Conservative groundwater transport of a generic contaminant from the Sanitary Landfill under capped conditions was evaluated using the areal flow model and the cross-sectional flow and transport model. The areal flow model was used to analyze changes in the areal movement of contaminants through particle tracking techniques. The advective travel times of a series of particles to discharge to points down gradient were compared between various capping scenarios and also were compared to the steady-state calibrated results predicted under no-cap conditions. The vertical flow and transport model was used to transport a generic conservative constituent from the Sanitary Landfill down gradient to discharge points down gradient. Simulations were compared for various capped conditions versus the steady-state calibrated no-cap conditions. Comparisons were made between actual contaminant distributions and changes in seepage and UTRC concentrations under capping and no-cap conditions.

Prior to evaluating the changes in advective travel times from the particle tracking simulations, maximum groundwater flow velocity changes as a result of

capping should be evaluated. The areal flow model was used to analyze changes in groundwater velocities resulting from capping of the Sanitary Landfill. In the previous analyses of head changes due to capping (discussed in Section 6.1.1), the transient change in heads was evaluated for three locations within the Sanitary Landfill for capping Scenario 1 assuming a cap of 100% efficiency (see Figure 6.1). As a comparison, here we will look at the transient change in groundwater flow velocity calculated for the same three locations in the model for Scenario 1 with an assumed cap efficiency of 100%.

Figure 6.15 illustrates the effects of capping on the groundwater velocity at locations A, B, and C corresponding to the north perimeter, middle, and south perimeter of the Sanitary Landfill. Figure 6.15 shows the relative percent difference in the magnitude of the groundwater velocity vector as a function of time after the 100% efficient cap of Scenario 1 is implemented. Because this case eliminates infiltration over the Sanitary Landfill, the changes in velocity for this simulation represent maximum values or maximum effects resulting from capping. At location A (the north end of the Sanitary Landfill), the groundwater velocity increases through time and reaches a new quasi steady-state velocity approximately 2.5% greater than the velocity calculated before capping. The increase in velocity is a result of slightly increased hydraulic gradients toward the center of the Sanitary Landfill caused by the decrease in recharge over that area. At location B, in the middle of the original Sanitary Landfill, the velocity only increases about 1%. At location C, at the southern end of the Sanitary Landfill, the groundwater velocity actually decreases by approximately 5% after reaching a new quasi steady-state flow field. The decreased velocity is caused by a reduction in the hydraulic gradients in this region of the model. In summary, the velocity increases in the region up-gradient from the location of maximum head decrease and will decrease down-gradient from that point. The amount of time required for the system to reach hydraulic equilibrium after recharge reduction (with respect to variations in groundwater velocity) is about 1 to 2 years which is consistent with the heads. As mentioned previously, this does not consider the lag time for reduction of fluid

storage in the unsaturated zone. If this time lag were incorporated into this analysis, the expected time to reach a new equilibrium would be increased.

6.2.1 Particle Tracking Results

6.2.1.1 Areal Flow Model

Particle tracking simulations were also completed in order to analyze the effects of capping on advective groundwater travel times to potential discharge points in the wetlands and in UTRC down gradient. Figure 6.16 illustrates groundwater travel paths and times in the vicinity of the Sanitary Landfill within the steady-state calibrated areal flow model under no-cap conditions. The particles are plotted at 1 year time increments along the pathline. It should be noted that the MODPATH particle tracking code allows particles to be tracked through discharge areas without being taken out of the system. In other words, the particles are tracked through the wetlands as part of the underflow which passes through the wetlands and on to discharge at UTRC. Since the areal model consists of one layer and does not consider vertical flow components, it is not possible to identify which particles would actually discharge in the wetlands and which would travel through and exit at UTRC. In this simulation, it was assumed that the particles move through the wetlands. The convergence of the particle pathlines in some areas of the wetlands indicates that significant discharge occurs in these areas. Also, the travel velocity within the wetlands decreases as a result of a steady decrease in volumetric groundwater flux resulting from wetland discharge.

Figure 6.17 illustrates the movement of a group of particles which were initially placed at the perimeter of the original Sanitary Landfill and the Southern Expansion. This simulation was completed to show the potential advective transport flow paths from the potential source area at the Sanitary Landfill. As with previous particle tracking results, it is evident that discharge occurring in the

wetlands serves to decrease the width of the "plume" under the particle discharge assumptions of this model.

As was performed on the volumetric flow balance (Section 6.1.1.2), here changes in the groundwater flow velocity in the vicinity of the Sanitary Landfill was investigated by comparing differences in the travel times of five particles tracked from the landfill to potential discharge points in the wetlands and UTRC down gradient under the different capping scenarios and efficiencies. Five particles were tracked from different locations within the Sanitary Landfill to observe the potential variations in travel time changes of contaminants emanating from anywhere within the original and southern sector of the Sanitary Landfill. One analysis involved tracking particles from their origin at the Sanitary Landfill to the first potential discharge point at the wetlands. This discharge point would be equivalent to the seepline location (see Figure 1.2). Another evaluation involved tracking the particles from the Sanitary Landfill through the wetlands to UTRC. Both evaluations were performed for capping Scenarios 1 and 2 for cap efficiencies of 100, 95, 90, and 85% to illustrate the full range of estimated travel time changes as a result of the various capping possibilities.

Figure 6.18 illustrates the initial start points and particle pathlines and travel times (particle positions indicate one year travel time) for the five particles dropped at the Sanitary Landfill. This figure shows the particle pathlines from the Sanitary Landfill to the seepline as predicted by the steady-state calibrated areal flow model under no-cap conditions. Figure 6.19 shows the pathlines for the 5 particles tracked from the Sanitary Landfill through the wetlands to UTRC as predicted by the steady-state calibrated areal flow model under no-cap conditions. Figures 6.20 and 6.21 illustrate the five particle traces from the Sanitary Landfill to the seepline and to UTRC, respectively. However, these particle paths are tracked in the steady-state flow field under Scenario 1 capping assumptions with 100% efficiency. The deviation from no-cap conditions should be greatest for the Scenario 1 100% capping efficiency case. The pathlines are shown to illustrate that only minor

differences exist in predicted particle paths and travel times, regardless of capping strategies. The particle travel time results for the entire series of simulations for capping Scenario 1 (assuming various capping efficiencies) are summarized in Table 6.2. Similarly, results from capping Scenario 2 are summarized in Table 6.3. The RPD values in the tables indicate the percent difference between the travel times under "no-cap" conditions and the those calculated for various capping efficiencies. As shown in the tables, the travel time to the beginning of the wetlands (the seepage line) increases regardless of the capping scenario or the efficiency of the cap. The increase in the travel times to UTRC are very small. In fact, for particle 1 the travel time to the wetlands decreases from 1 to 2.8% relative to no-cap travel times. This analysis indicates that capping the Sanitary Landfill will not significantly change the groundwater velocities or travel times between the Sanitary Landfill and potential discharge points located down gradient of the Sanitary Landfill.

6.2.1.2 Vertical Cross-Section Model

While the areal model is useful in looking at the areal distribution of pathlines from the Sanitary Landfill to the discharge areas, it only allows us to look at effects averaged over the entire thickness of the aquifer. The cross-sectional model allows us to look at the effects upon vertical flowpaths caused by capping. Figure 6.22 shows the particle pathlines for the vertical steady-state calibrated cross-sectional flow model under no-cap conditions. The pathlines indicate that particles infiltrating below the Sanitary Landfill will migrate down from the Sanitary Landfill and be transported laterally to the wetland and UTRC discharge areas located down gradient. Although the effects of dispersion (mechanical mixing and diffusion) are not incorporated into the particle tracking techniques, it is apparent that significant vertical movement of fluid occurs in the aquifer simply due to infiltration. The particle paths define flow tubes and stream lines. These flow tubes show that infiltration resulting from recharge occurs homogeneously for regions both interior and exterior to the Sanitary Landfill. Particles that enter the saturated

zone at the northern extent of the Sanitary Landfill eventually exit in the wetlands according to this simple advection model. This cross-sectional view of the flow system clearly illustrates how the entire saturated zone at the left side of the model forms a smaller, faster moving "wedge" of water as it travels toward the wetlands. However, travel velocities between the seepage line and UTRC decrease as a result of a steady decrease in volumetric flux as the wetlands discharge groundwater to the surface. Because UTRC is considered as a groundwater divide in Aquifer Unit I/II C, it is treated as a no-flow boundary condition. Groundwater from the deepest portions of the aquifer eventually exit very near or directly into UTRC.

Figure 6.23 illustrates the particle pathlines for a steady-state simulation for Scenario 1 cap conditions assuming a cap efficiency of 95%. The area where change in particle pathlines is most evident is directly beneath the landfill because little infiltration is occurring beneath the landfill due to capping. This is manifested in the way that particle pathlines originating from underneath the capped portion of the landfill stay close to the water table as they progress south and down gradient. Once a particle pathway moves along the water table and reaches the southern extent of the Sanitary Landfill, infiltration forces the particles deeper into the aquifer.

6.2.2 Contaminant Concentrations Under Capping Scenarios

A series of simple generic transport simulations were completed to evaluate the effects of capping on contaminant concentrations at potential discharge points down gradient of the Sanitary Landfill. Steady-state distributions of the generic contaminant relative concentrations were compared between no-cap and Scenario 1 simulations. A complete comparison between steady-state relative concentrations for no-cap, and Scenario 1 and 2 cap conditions was performed to quantify the concentration reduction resulting from capping and to compare various cap efficiencies. The two points chosen to monitor and compare concentration were the first discharge block in the wetlands which can be

considered equivalent to the seepline and UTRC which is the last discharge block down gradient from the Sanitary Landfill. A more complete COC specific analysis is performed in Section 8 in support of the ACL demonstrations. This analysis is for a generic constituent which is assumed to be conservative (distribution coefficient $K_d = 0.0$ mL/g) and does not decay. Steady-state concentrations will be compared at the seepline and UTRC between no-cap and various capping conditions. The generic simulations assume that a constant source relative concentration (C/C_0) of 1.0 is input over the Sanitary Landfill consistent with the maximum input source scenario consistent with the COC tetrachloroethylene (see Section 5.5.3 or Table 5.5). This assumes that only quadrants 1-4, 6 and 7 are source areas and that these areas input contaminants at a constant rate for all time (see Figure 3.1).

Figure 6.24 illustrates the steady-state relative concentration contours for the aquifer assuming no cap is in place. Since the loading scenario for this simulation assumes that quadrants 5 and 8 are not contaminant sources, a two-lobed plume is produced as is seen in Figure 6.24. Horizontal underflow of uncontaminated groundwater from up gradient of the Sanitary Landfill dilutes the contaminants significantly. The highest relative concentration in the two lobes below the Sanitary Landfill is about 0.28. The major factor in reducing concentrations is dilution of the contaminants over the thickness of the aquifer. Dispersion tends to mix contaminants significantly lowering concentrations as the travel distance increases. In fact, the plume is nearly completely mixed in the vertical direction (same concentration with depth) in the wetland region. Concentrations are also lowered in areas down gradient from the Sanitary Landfill by vertical recharge of uncontaminated groundwaters. The first discharge block (seepline) is located on Figure 6.24 at an approximate x-coordinate of 5750 ft. The steady-state relative concentration at the seepline is just less than 0.15. As can be seen in this figure, the concentrations in the aquifer between the seepline and UTRC are well mixed and are within relative concentrations 0.149 and 0.147.

Figure 6.25 shows the steady-state relative concentration within the aquifer when capping Scenario 1 with an assumed cap efficiency of 95%. The two lobes of relatively higher concentration are still present, however the overall concentration throughout the aquifer is significantly reduced because the contaminant loading through infiltration has been significantly reduced. The maximum relative concentration observed under the Sanitary Landfill is approximately 0.07 which is a factor of four smaller than for the no-cap simulation. The infiltration due to recharge through the cap has been reduced by a factor of 6.2. As in the no-cap case, the contamination is well mixed in the vertical direction from the seepage to UTRC.

For a conservative constituent, the steady-state concentration down gradient assuming perfect vertical mixing is easily calculated. The maximum relative concentration is simply the source area volumetric flux divided by the total volumetric flux entering the model from up-gradient aquifer flow and recharge infiltration in areas of the model excluding the source area.

Table 6.4 lists the steady-state relative concentration at the seepage and at UTRC for no-cap and Scenario 1 and 2 capping conditions for various cap efficiencies. As above, the simulations input a constant relative concentration of 1.0 in a way consistent with the maximum source scenario for tetrachloroethylene. The concentrations are very similar at the seepage as compared to UTRC which is consistent with Figures 6.24 and 6.25 and the fact that the transported constituent is conservative. As would be expected, concentrations increase as the cap efficiency decreases. Also, the concentrations decrease from Scenario 1 to Scenario 2 for a given cap efficiency. This is because the portion of the southern sector not capped in Scenario 2 is also not a source of contaminant in these simulations. Therefore, infiltration of non-contaminated water is occurring at a rate of 15 in/yr over the north half of the southern sector effectively diluting concentrations observed down gradient.

Table 6.4 also lists the relative percent difference (RPD) in steady-state concentrations simulated for the various capping scenarios as compared to no-cap conditions. From this analysis one can see that the cap offers an effective method for long-term reduction of groundwater concentrations at the Sanitary Landfill and vicinity.

Location of monitored grid blocks for graph shown above.

Figure 6.1 Simulated Head Change at Selected Locations of the Sanitary Landfill in Response to Scenario 1 Capping (100% Efficiency)

Figure 6.2 Areal Head Change at the Sanitary Landfill (under quasi-steady-state conditions) in Response to Scenario 1 Capping (100% Efficiency)

Figure 6.3 Variable Precipitation and Recharge History Implemented in the Variable Recharge Simulation

Figure 6.4 Location and Reference Number of Grid Blocks Monitored during the Variable Recharge Simulation

Figure 6.5 Simulated Head Change at Selected Monitor Blocks in Response to Transient Recharge (North-South)

Figure 6.6 Simulated Head Change at Selected Monitor Blocks in Response to Transient Recharge (East-West)

Figure 6.7 Simulated Areal Head Change at the Sanitary Landfill (Under Steady-State Conditions) in Response to Scenario 1 Capping (100% Efficiency)

Figure 6.8 Simulated Areal Head Change at the Sanitary Landfill (Under Steady-State Conditions) in Response to Scenario 1 Capping (95% Efficiency)

Figure 6.9 Simulated Areal Head Change at the Sanitary Landfill (Under Steady-State Conditions) in Response to Scenario 1 Capping (90% Efficiency)

Figure 6.10 Simulated Areal Head Change at the Sanitary Landfill (Under Steady-State Conditions) in Response to Scenario 1 Capping (85% Efficiency)

Figure 6.11 Simulated Areal Head Change at the Sanitary Landfill (Under Steady-State Conditions) in Response to Scenario 2 Capping (100% Efficiency)

Figure 6.12 Simulated Areal Head Change at the Sanitary Landfill (Under Steady-State Conditions) in Response to Scenario 2 Capping (95% Efficiency)

Figure 6.13 Simulated Areal Head Change at the Sanitary Landfill (Under Steady-State Conditions) in Response to Scenario 2 Capping (90% Efficiency)

Figure 6.14 Simulated Areal Head Change at the Sanitary Landfill (Under Steady-State Conditions) in Response to Scenario 2 Capping (85% Efficiency)

Location of monitored grid blocks for graph shown above.

Figure 6.15 Groundwater Velocity Change at Selected Locations of the Sanitary Landfill in Response to Capping Scenario 1 (100% Efficiency)

Figure 6.16 Particle Pathlines in the Vicinity of the Sanitary Landfill Under Steady-State No Cap Conditions

Figure 6.17 Pathlines of Particles Originating from the Outer Boundary of the Sanitary Landfill under Steady-State No Cap Conditions

Figure 6.18 Particle Pathlines for 5 Particles Originating from Within the Sanitary Landfill Moving to the Seepage Line Under Steady-State No Cap Conditions

Figure 6.19 Particle Pathlines for 5 Particles Originating from Within the Sanitary Landfill Moving to UTRC Under Steady-State No Cap Conditions

Figure 6.20 Particle Pathlines for 5 Particles Originating from Within the Sanitary Landfill Moving to the Seepage Line Under Steady-State Conditions, Scenario 1 Capping (100% Efficiency)

Figure 6.21 Particle Pathlines for 5 Particles Originating from Within the Sanitary Landfill moving to UTRC Under Steady-State Conditions, Scenario 1 Capping (100% Efficiency)

Figure 6.22 Steady-State Particle Pathlines for the Cross-Section Flow Model Under No-Cap Conditions

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Figure 6.23 Steady-State Particle Pathlines for the Cross-Section Flow Model Under Scenario 1 Capping (100% efficiency)

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Figure 6.24 Steady-State Relative Concentration Distribution Under No-Cap Conditions

Figure 6.25 Steady-State Relative Concentration Distribution Under Scenario 1 Capping (95% Efficiency)

Table 6.1 Summary of Volumetric Flow Balance for Each Capping Scenario

CAPPING SCENARIO 1					
Boundary Condition	Cap Efficiency				
	No Cap	85%	90%	95%	100%
	Volume (ft ³ /day)				
PHBs	181410	182480	182800	183130	183460
Recharge	316310	311730	310320	308910	307490
Drains	265300	262170	261210	260250	259290
GHBs	232410	232010	231890	231770	231640
Total In:	497720	494210	493120	492040	490950
Total Out:	497710	494210	493120	492040	490930
Boundary Condition		Relative Percent Difference (RPD) from No-Cap Scenario			
PHBs		0.59	0.77	0.95	1.13
Recharge		-1.45	-1.89	-2.34	-2.79
Drains		-1.18	-1.54	-1.90	-2.27
GHBs		-0.17	-0.22	-0.28	-0.33
Total In:		-0.71	-0.92	-1.14	-1.36
Total Out:		-0.71	-0.92	-1.14	-1.36

PHB = Prescribed Head Boundary

GHB = General Head Boundary

Table 6.1 Summary of Volumetric Flow Balance for Each Capping Scenario (Cont.)

CAPPING SCENARIO 2					
Boundary Condition	Cap Efficiency				
	No Cap	85%	90%	95%	100%
	Volume (ft ³ /day)				
PHBs	181410	182320	182600	182880	183160
Recharge	316310	312580	311430	310280	309130
Drains	265300	262800	262030	261260	260500
GHBs	232410	232080	231980	231880	231780
Total In:	497720	494900	494030	493160	492290
Total Out:	497710	494900	494030	493160	492280
Boundary Condition		Relative Percent Difference (RPD) from No-Cap Scenario			
PHBs		0.50	0.66	0.81	0.96
Recharge		-1.18	-1.54	-1.91	-2.27
Drains		-0.94	-1.23	-1.52	-1.18
GHBs		-0.14	-0.19	-0.23	-0.27
Total In:		-0.57	-0.74	-0.92	-1.09
Total Out:		-0.57	-0.74	-0.92	-1.09

PHB = Prescribed Head Boundary
 GHB = General Head Boundary

Table 6.2 Summary of Particle Pathline Analyses to Seepage and UTRC for Capping Scenario 1

SEEPAGE SUMMARY					
Particle	Cap Efficiency				
	No Cap	85%	90%	95%	100%
	Travel Time to Wetlands (years)				
1	4.973	5.016	5.029	5.181	5.199
2	5.684	5.709	5.718	5.727	5.736
3	5.259	5.408	5.411	5.414	5.417
4	3.157	3.208	3.224	3.241	3.257
5	1.287	1.313	1.321	1.330	1.338
Particle		Relative Percent Difference (RPD) from No-Cap Scenario			
1		0.86	1.13	4.18	4.54
2		0.44	0.60	0.76	0.91
3		2.83	2.89	2.95	3.00
4		1.62	2.12	2.66	3.17
5		2.02	2.64	3.34	3.96
Average		1.55	1.88	2.78	3.12
Overall Average = 2.33 Percent					

Table 6.2 **Summary of Particle Pathline Analyses to Seepage and UTRC for**
(Cont.) **Capping Scenario 1**

UPPER THREE RUNS CREEK SUMMARY					
Particle	Cap Efficiency				
	No Cap	85%	90%	95%	100%
	Travel Time to Upper Three Runs Creek (years)				
1	11.755	11.611	11.549	11.488	11.417
2	11.718	11.788	11.805	11.821	11.837
3	11.409	11.473	11.488	11.498	11.510
4	8.880	8.941	8.961	8.980	8.998
5	7.800	7.860	7.890	7.900	7.910
Particle		Relative Percent Difference (RPD) from No-Cap Scenario			
1		-1.23	-1.75	-2.27	-2.88
2		0.60	0.74	0.88	1.02
3		0.56	0.69	0.78	0.89
4		0.69	0.91	1.13	1.33
5		0.77	1.15	1.28	1.41
Average		0.28	0.35	0.36	0.35
Overall Average = 0.33 Percent					

Table 6.3 Summary of Particle Pathline Analyses to Seepage and UTRC for Capping Scenario 2

SEEPAGE SUMMARY					
Particle	Cap Efficiency				
	NO CAP	85%	90%	95%	100%
	Travel Time to Wetlands (years)				
1	4.973	5.013	5.025	5.181	5.185
2	5.684	5.711	5.720	5.727	5.739
3	5.259	5.412	5.417	5.414	5.426
4	3.157	3.197	3.209	3.241	3.235
5	1.287	1.307	1.314	1.330	1.327
Particle		Relative Percent Difference (RPD) from No-Cap Scenario			
1		0.80	1.05	4.18	4.26
2		0.48	0.63	0.76	0.97
3		2.91	3.00	2.95	3.18
4		1.27	1.65	2.66	2.47
5		1.55	2.10	3.34	3.11
Average		1.40	1.69	2.78	2.80
Overall Average = 2.17 Percent					

Table 6.3 **Summary of Particle Pathline Analyses to Seepage and UTRC for**
(Cont.) **Capping Scenario 2**

UPPER THREE RUNS CREEK SUMMARY					
Particle	Cap Efficiency				
	NO CAP	85%	90%	95%	100%
	Travel Time to Upper Three Runs Creek (years)				
1	11.755	11.659	11.621	11.576	11.531
2	11.718	11.785	11.802	11.817	11.833
3	11.409	11.468	11.483	11.497	11.507
4	8.880	8.928	8.943	8.958	8.973
5	7.800	7.849	7.862	7.872	7.910
Particle		Relative Percent Difference (RPD) from No-Cap Scenario			
1		-0.82	-1.14	-1.52	-1.91
2		0.57	0.72	0.84	0.98
3		0.52	0.65	0.77	0.86
4		0.54	0.71	0.88	1.05
5		0.63	0.79	0.92	1.41
Average		0.29	0.35	0.38	0.48
Overall Average = 0.37 Percent					

Table 6.4 Simulated Steady-State Relative Concentration at the Seepline and UTRC Under Varying Cap Efficiencies; Maximum Source Scenario; Representative of PCE.

	Simulated Relative Concentration	
	Seepline	UTRC
NO CAP	0.1490	0.1466
SCENARIO 1		
100% Efficiency	0.0000	0.0000
95% Efficiency	0.0273	0.0260
90% Efficiency	0.0530	0.0510
85% Efficiency	0.0744	0.0750
SCENARIO 2		
100% Efficiency	0.0000	0.0000
95% Efficiency	0.0262	0.0253
90% Efficiency	0.0515	0.0499
85% Efficiency	0.0758	0.0738
	Relative Percent Difference (RPD) from No-Cap Scenario	
	Seepline	UTRC
SCENARIO 1		
100% Efficiency	-100	-100
95% Efficiency	-81.68	-82.26
90% Efficiency	-64.43	-65.21
85% Efficiency	-48.05	-48.84
SCENARIO 2		
100% Efficiency	-100	-100
95% Efficiency	-82.42	-82.74
90% Efficiency	-65.44	-65.96
85% Efficiency	-49.13	-49.66

7.0 TERMINOLOGY FOR THE PRESENTATION OF ACL TRANSPORT RESULTS

As required under the Resource Conservation and Recovery Act (RCRA), owners and operators of hazardous waste facilities must establish groundwater protection standards (GWPS) which, if exceeded, trigger corrective action. There are three possible GWPS. They are; (1) background concentration, (2) the maximum concentration limit (MCL) defined in 40 CFR 264.94(a), or (3) an alternative concentration limit (ACL). Two fundamental guidelines that an ACL must be based upon are that the COC plume should not increase in size or concentration above allowable exposure levels and that it will not allow contamination of off-site groundwater above allowable limits (EPA, 1987).

Several terms and definitions are needed before results from the transport calculations specific to the ACL demonstration can be meaningfully discussed. Two definitions primary to the concept of an Alternative Concentration Limit are the Point of Compliance (POC) and the Point of Exposure (POE). The POC is defined as a vertical surface located at the hydraulically down-gradient limit of the waste management area and that extends into the uppermost aquifer. Wells considered as POC wells at the Sanitary Landfill are all wells completed across the water table and that are peripheral to the Sanitary Landfill (see Figure 1.3). These do not include well LFW-43D through LFW-62D which are relatively new, have a short sampling history, and are not immediately adjacent to the landfill periphery. It also does not include wells LFW-29, LFW-30, and LFW-31 because these wells are up gradient of the unused Northern Expansion and are classified as background monitoring wells.

The Point of Exposure is defined as being any point where a potential receptor, either now or in the future, can come in contact with the groundwater contaminants emanating from the waste management area. For these transport calculations, the POE is considered the closest down-gradient wetlands where

groundwater is discharging to the surface. In the cross-sectional transport model, the POE is located approximately 1,100 feet down gradient from the southern edge of the Southern Expansion of the Sanitary Landfill. This location approximately coincides with the locations of Wetland Sampling Points SP-262 and SP-263. The POE location in the transport model coincides with the location of the seepage as it was discussed in Chapter 6 and shown in Figure 6.24. The assumption that the closest discharge block to the Sanitary Landfill is equal to the designated POE is conservative, because the highest concentrations are monitored there for most cases. This location also coincides with the shortest transit time between the Sanitary Landfill and groundwater discharge points. Figure 7.1 shows a schematic of the transport pathway modeled with both the POC and POE shown.

Two loading scenarios are examined in the transport calculations in support of the ACL Demonstrations: a conservative best-estimate source scenario and a maximum possible source scenario. For these two scenarios, two cases are examined. First, it is assumed that the Sanitary Landfill will never be capped and transport simulations are performed for no-cap conditions. In this case, it is assumed that the infiltration through the waste is equal to the calibrated recharge estimate of 15 in/year. The second case is equivalent to Scenario 1 from Chapter 6 where a geosynthetic cap is emplaced over the entire area of the original and Southern Expansions of the Sanitary Landfill. For the ACL supporting transport calculations, the cap is assumed to have a 97% efficiency which implies that only 3% of average precipitation (1.4 in/yr) infiltrates the cap and produces leachate (Bill Pidcoe, personal communication.). As well, the cap is assumed to be put in place in the middle of 1995 and institutional control assumed to occur for at least 30 years or until the middle of year 2025 (Bill Pidcoe, personal communication).

A cap efficiency of 97% is considered conservative based upon numerical modeling of the Sanitary Landfill cap efficiency by WSRC personnel. The cap efficiency modeling was performed using the U.S. Army Corps of Engineers Hydrologic Evaluation of Landfill Performance (HELP) model. HELP (Schroeder

et al., 1988) uses climatological data and soil properties of the RCRA closure cap to model rain infiltration or recharge through the cap. The cap consists of a bentonite mat, a flexible membrane liner (FML), a drainage layer, select fill, topsoil, and vegetation. Table 7.1 summarizes the soil properties used as base case values to simulate recharge through the cap with the HELP model. These values are conservative estimates which should result in increased recharge. The climatological data used included 10 years (1982-1991) of precipitation records at the F Area and temperature data from Augusta, GA.

Based upon the design specifications for the Sanitary Landfill cap and based upon best-estimate values of model parameters (Table 7.1), the HELP model predicted that the Sanitary Landfill cap would be 99.9% efficient. Through a single-parameter variation sensitivity analysis the cap efficiency did not decrease more than 1.1% for any sensitivity run. For conservatism, a simulation was run where the bentonite mat hydraulic conductivity was increased to 1×10^{-7} cm/sec (2.8×10^{-4} ft/d) and the SCS runoff number and the FML leakage fraction were set equal to 30 and 0.02, respectively. For this case, the HELP model predicted a cap efficiency of 97.3%. Based upon this conservative calculation, a cap efficiency of 97% was adopted for these transport calculations in support of the Sanitary Landfill ACL Demonstrations.

The cross-sectional transport model has been used to conservatively quantify the relationship between COC concentrations at the POC and POE for cap and no-cap conditions and for two different loading scenarios. The results from these simulations will be reported in terms of relative and absolute concentrations. For COC transport results under no-cap conditions, maximum concentrations at the end of institutional control (year 2025) are reported both as relative and absolute values. For COC transport results under Scenario 1 capped conditions, peak concentrations and concentrations at the end of institutional control are reported both as relative and absolute values. The absolute maximum concentrations are calculated assuming that the simulated maximum POC relative concentration is

equivalent to the maximum observed concentration measured at any POC well from 1984 through the fourth quarter of 1991 (Dames and Moore, 1992). Therefore, absolute concentration estimates can be considered conservative. Relative and absolute concentrations for each COC for each scenario are reported for the POC, POE, and UTRC. In addition to concentrations, attenuation factors were calculated based upon simulated concentrations. Attenuation factors will be formally defined in the following section. However, a porous media attenuation factor can be defined as a factor which quantifies the relationship between a COC concentration between two points in an aquifer.

7.1 Estimation of Attenuation Factors

The primary goal of these transport calculations is to quantify, under conservative conditions, the relationship between the concentration of a given COC at the POC and POE as defined above. One method to do this is to calculate an attenuation factor. The attenuation factor represents all of the attenuating processes which occur for a given COC during transport in the groundwater. These processes include dispersion, diffusion, dilution, degradation, and sorption. The attenuation factor can be defined as the ratio between the simulated relative concentrations between two points within the porous media. Therefore, an attenuation factor implies a space over which it is measured. For example, the attenuation factor between the POC and POE is equal to the ratio between the simulated relative concentration at the POC and POE. This can be expressed as

$$A = \frac{[C/C_o]_{POC}}{[C/C_o]_{POE}} \quad (7.1)$$

The attenuation factor for the example depicted in Figure 7.1 would be equal to 3. An attenuation factor also can be dependent on time. That is, the concentrations in the porous media must be related to some time. In a steady-

state concentration distribution, the attenuation factor is independent of time. Under the assumption of a steady-state concentration distribution, a single value of attenuation factor has meaning and utility. However, for transient source conditions such as occurs with capping, an attenuation factor can be transient and should be related to the time it is evaluated.

For all the transport simulations performed to support the ACL demonstration, the input relative concentration (C/C_o) is set equal to unity. Because there is significant flow of groundwater from up gradient of the Sanitary Landfill, one cannot assume that the maximum observed concentration at the POC is equivalent to the initial source relative concentration of 1.0. This can be seen in Figure 7.1 where the input relative concentration is equal to 1.0 but the maximum simulated relative concentration at the POC is equal to 0.3. The maximum simulated relative concentration under the landfill ($[C/C_o]_{POC}^{MAX}$) is assumed equivalent to the maximum observed concentration of a given COC at the POC wells (C_{POC}^{MAX}) as defined by Dames and Moore (1992) and listed in Table 3.1. Attenuation factors between the POC and UTRC were also calculated using Equation 7.1.

The predicted concentration history of a given COC at the POC and POE are quite different under no-cap versus cap conditions. Figure 7.2 shows an example of the behavior of representative breakthrough curves at the POC and POE for both cap and no-cap conditions. As can be seen, the no-cap case breakthrough curve builds to a maximum which for most Sanitary Landfill COCs occurs well before the end of institutional control (year 2025). For the no-cap simulations, porous media attenuation factors were calculated between the POC and POE and the POC and UTRC. The simulated concentrations to calculate attenuation factors were taken at the end of institutional control (year 2025) which represents maximum simulated concentrations as well as steady-state concentrations at all points.

For capped conditions, the concentrations at the POC will increase until the landfill is capped (the middle of 1995) at which time they will start to diminish until a new much lower concentration is established. At various places down gradient in the aquifer, the concentration will increase and fall off also and the timing and magnitude of the peak will be determined by the transport properties of the individual COC. Therefore, the concentration at the POC will be smaller than at the POE for some time period after capping and before a new steady-state concentration distribution is established. This will result in an attenuation factor smaller than 1. Even though the porous media attenuation factors are highly time dependent, they have been calculated for the POE, wetlands, and UTRC based upon simulated concentrations at the end of institutional control. In addition, because of this transient peak concentration behavior, porous media attenuation factors for the capping scenarios were also calculated using the maximum simulated concentrations whenever they occurred. The maximum concentrations under capped conditions occur at different times for the POC, POE, and UTRC. Therefore, an attenuation factor derived from the peak simulated concentrations is not representative of a single discrete time. However, they should be conservative in that they are smaller in magnitude than the attenuation factor calculated at the end of institutional control.

7.2 Estimation of Absolute Concentrations

All simulations assume that the relative initial concentration (C/C_o) for all COCs is equal to unity. Therefore, SWIFT II calculates concentration in the aquifer as a relative concentration. One can easily convert the simulated relative concentration to an absolute concentration by assigning the initial concentration (C_o) an absolute concentration value. Then one multiplies this concentration by the simulated relative concentration to get an absolute concentration.

For no-cap conditions, we have assumed that the maximum observed absolute concentration of a given COC at any of the POC wells (C_{POC}^{MAX}) is equal to

the maximum simulated relative concentration calculated under the Sanitary Landfill ($[C/C_o]_{POC}^{MAX}$ no cap). This is not equivalent to assuming that the maximum observed POC concentration is equal to the initial concentration (C_o) because there is significant dilution which reduces concentrations by a factor of 4 or greater immediately under the landfill depending upon the source loading. The concentration first arriving to the saturated zone is significantly higher than the maximum concentration observed in a 15 to 20 foot saturated screened interval below the water table or simulated with a 10 foot grid block. Therefore, one must calculate the initial concentration for a given simulation and it is equal to:

$$\hat{C}_o = \frac{C_{POC}^{MAX}}{[C/C_o]_{POC}^{MAX} \text{ no cap}} \quad (7.2)$$

where $[C/C_o]_{POC}^{MAX}$ no cap) is equal to the maximum simulated relative concentration evaluated under no cap conditions (i.e., at 2025) under the Sanitary Landfill and C_{POC}^{MAX} is equal to the observed maximum POC concentration (see Table 3.1). Then an absolute concentration at the POE is calculated as:

$$\hat{C}_{POE} = [C/C_o]_{POE} \times \hat{C}_o \quad (7.3)$$

Under capped conditions, absolute concentrations can also be calculated using Equation 7.4. The initial concentration ($\hat{C}_o$) is equal to the no-cap initial concentration and again is calculated by Equation 7.2.

Figure 7.1 Schematic of ACL Transport Pathway with Representative Breakthrough Curves for the POC and POE

Figure 7.2 Representative Breakthrough Curves at the POC and POE under Cap and No-Cap Conditions

Table 7.1 Soil Properties of Engineered Sanitary Landfill Cap Used in HELP Model.

Engineered Cap Layer	Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Hydraulic Conductivity (cm/sec)	Hydraulic Conductivity (ft/day)
Top Soil	0.457	0.083	0.033	0.0130	36.9
Select Fill	0.457	0.083	0.033	0.0031	8.8
Drainage Layer	0.417	0.045	0.020	0.010	28.3
Bentonite Mat	0.400	0.356	0.290	1×10^{-8}	2.8×10^{-8}

8.0 TRANSPORT CALCULATIONS IN SUPPORT OF DETERMINATION OF ALTERNATIVE CONCENTRATION LIMITS

The cross-sectional flow and transport model has been used to conservatively quantify the relationship between COC concentrations at the POC and POE for no-cap and Scenario 1 capped conditions. For the capped and no-cap cases two different loading scenarios have been modeled. The first source loading scenario is termed the conservative best-estimate source scenario. For this case a conservative estimate of source dimensions has been derived based upon the presence or absence of a given COC in the monitoring wells at the Sanitary Landfill. Section 5.5.3 presents the details on how the source scenarios were defined and specified. A second source scenario modeled is the maximum possible source scenario. This source loading scenario assumes that the entire Sanitary Landfill original and southern sector are potential source areas. For the constituents assumed to be products of the solvent rags (PCE and TCE), the maximum source scenario only assumes that those portions of the southern sector which had rag deposition are potential source areas. The reason for simulating two source loading scenarios is because there is a great deal of uncertainty in the potential source areas for the Sanitary Landfill COCs. Both scenarios are conservative by design because once a source parcel begins to act as a source of mass flux, this area is assumed not to deplete over the entire model simulation period. The input of a constant source equivalent to the maximum observed concentration is thought to be conservative. However, because so little is known regarding inventory and because the calculations must predict far into the future, a conservative source input is warranted.

The transport model is described in Chapter 4. The model is a cross-sectional flow and transport model which is representative of a centerline concentration of a COC plume emanating from the Sanitary Landfill. Because the plume centerline concentrations are the maximum, this model formulation is conservative. Each COC is modeled independently based upon the specific

transport properties unique to that COC. Such COC specific transport properties are the distribution coefficient, the degradation rate if applicable, and the source loading scheme. All of the properties are described in detail in Chapter 5. The simulations are performed using relative concentrations assuming that an input concentration is emanating from the Sanitary Landfill with a relative input concentration (C/C_o) of 1.0. The simulations assume that the absolute input concentration for a given COC is representative of the maximum observed POC well concentration of that COC (see Table 3.1). Therefore, predicted absolute concentrations calculated from model simulated relative concentrations should represent historical maximums. However, if concentrations observed at the POC wells of the Sanitary Landfill exceed the historical maximum observed concentrations as listed in Table 3.1, these simulations may cease to be conservative.

8.1 ACL Transport Model Assumptions

The transport model used to provide transport results in support of the ACL Demonstrations is a simplification of the complex and heterogeneous transport system controlling the fate of COCs between the Sanitary Landfill and discharge areas down gradient. However, because of significant data uncertainties, a more complex model is not justifiable. The model used incorporates what are thought to be the controlling mechanisms governing transport between the Sanitary Landfill and the discharge points.

The primary model assumptions are:

- (1) Transverse dispersive spreading normal to the selected cross section is insignificant;

- (2) Source terms are constant throughout the simulation period and the source input concentration is representative of the highest observed POC well concentration recorded between 1984 and the fourth quarter of 1991;
- (3) Methanogenic conditions are thought to be representative of the flowpath;
- (4) Organic contaminants which are reported in the literature to degrade under methanogenic conditions are assumed to degrade according to a first order decay constant;
- (5) Organic COC half lives are representative of methanogenic conditions where data are available;
- (6) The Point of Exposure (POE) is assumed to occur at the closest discharge point down gradient from the Sanitary Landfill.
- (7) Source loading is assumed to commence in 1975 which is one year after the Sanitary Landfill is reported to have become active (see Table 5.6 and Figure 3.1)
- (8) For the capped cases, the Scenario 1 cap efficiency is assumed to be 97% based upon calculations performed by WSRC staff;
- (9) For the capped cases, capping occurs in the middle of the year 1995 and institutional control extends at a minimum of 30 years (middle of year 2025);

Because the source dimensions transverse to the flow direction is a significant fraction of the flow path distance, the assumption of no lateral spreading due to dispersion is justified.

8.2 ACL Transport Model Conservatism and Uncertainties

Because there are uncertainties inherent in the Sanitary Landfill database needed for transport calculations, there is a degree of uncertainty in the model results. However, the model is built to be conservative in an effort to account for the uncertainty in model input parameters. Because the information needed to quantify the COC plumes down gradient of the Sanitary Landfill was inadequate, initialization and calibration of the transport model using a standard solute transport modeling approach was not possible. As a result, the model was designed to be conservative in estimation of down-gradient concentrations of COCs.

A list of uncertainties regarding the transport modeling in support of the ACL Demonstrations are summarized below. This list is followed by the assumptions regarding the modeling effort which attempt to compensate for the uncertainties through conservatism where possible. It is understood that uncertainty is still inherent in the calculations regardless of the nature of the modeling assumptions adopted.

- Inventory of COCs is uncertain as well as COC source dimensions;
- Initial Concentration of COC sources is unknown;
- The COC plumes have not been characterized beyond the Sanitary Landfill at the site;
- Geochemistry of the Sanitary Landfill and flowpath are uncertain, especially with regards to redox conditions;
- Degradation rates for organic constituents which are known to degrade are uncertain;
- Distribution coefficients for the COCs are uncertain;
- There is uncertainty in the effects of capping on the Sanitary Landfill local infiltration rate and source concentrations in the short time frame after capping. This includes effects due to consolidation.

- Sensitivity analyses with the local-scale areal flow model indicate that the hydraulic conductivity could vary by a factor of 1.5 which implies that travel times predicted by this model could vary by a factor of 1.5.

Because of the large number of uncertainties in the data required to perform transport calculations at the Sanitary Landfill, several assumptions have been made with varying degrees of conservatism. In the following list, the assumptions based upon the uncertainties listed above will be summarized along with a qualifier as to the conservatism of the assumption.

- It is assumed that the COC inventories are large enough to allow constant source conditions for the period of the transport calculations. Based upon the groundwater contamination at the Sanitary Landfill, this is seemingly conservative. However, the potential for this size of inventory cannot be dismissed based upon scoping calculations (see Section 5.5.3). Two source scenarios are simulated to address the uncertainty in source dimension. Both scenarios are felt to be conservative and the maximum possible source scenario must be considered conservative.
- It is assumed that the initial concentration of a given COC input over the source dimension is representative of the maximum observed POC well concentration for that COC measured between 1984 and the fourth quarter of 1991. Based upon the constant source assumption and conservative source dimensions, this initial concentration should be considered conservative. However, concentrations could increase past 1991 for any given COC.
- COC plumes are not characterized beyond the Sanitary Landfill source region. As a result, history matching of transport plumes could not be

performed. The net result cannot be considered conservative or not conservative.

- The geochemistry of the Sanitary Landfill groundwater and the exposure pathway is not fully characterized. It was assumed that reducing conditions at the Sanitary Landfill prevailed and that the geochemical conditions along the flowpath were equivalent to the Sanitary Landfill. Reducing conditions are supported by the methanogenesis at the Sanitary Landfill. The assumption of equivalent geochemical conditions along the flowpath is a simplification based upon the lack of data and complexity of modeling justifiable. Because the discharge point is within 1 to 1.5 source dimensions, this assumption was considered practical. It is not known whether this is conservative.
- Degradation rates for the organic COCs are uncertain. The estimates of degradation rates were based upon the available literature for conditions similar to the Sanitary Landfill. Results from bioreactors were not used. Degradation as a process reduces concentrations and is therefore not conservative. However, degradation products are present at the Sanitary Landfill as well as are conditions thought to cause degradation. No degradation of COCs is assumed to occur in the source inventory which is located in the unsaturated zone. This assumption is conservative.
- Distribution coefficients for the organic COCs are based upon accepted estimation methods because Sanitary Landfill specific measured estimates are not available. To minimize predicted distribution coefficients for the organic COCs, a minimum fraction organic content (foc) was used which is conservative. For metals, a

distribution coefficient of 0.001 mL/g (after Looney et al., 1987) was used assuming facilitated transport which again is conservative.

- It is assumed that upon capping the infiltration and subsequent COC mass flux instantaneously decreases to a value representative of the cap efficiency. For short term effects this assumption may not be conservative, however, for long term concentration maximums it can be considered an appropriate assumption. The cap efficiency modeled (97%) is conservative based upon sensitivity modeling performed by WSRC personnel.

In the following two sections, the results from the no-cap and Scenario 1 cap transport simulations, respectively, are discussed.

8.3 No-Cap Conditions

The source loading assumptions assume that once a quadrant of the landfill becomes an active source, this quadrant of the landfill will remain a source for the remainder of the simulation. For the no-cap case, the source blocks emanate with a constant mass flux to the groundwater system resulting from the steady recharge rate of 15 in/yr. As a result, no-cap conditions produce breakthrough curves which build over time and come to steady-state conditions at late time (see Figure 7.2). For the majority of the COCs, the concentration distribution in the aquifer has come to a steady-state concentration many years before the end of institutional control. The following discussion will contrast the best-estimate source scenario with the maximum source scenario under no-cap conditions.

Table 8.1 lists the simulated relative concentrations (C/C_0) at the POC, POE, and UTRC at the end of institutional control for the conservative best-estimate source scenario. Because these concentrations always increase until they reach an asymptotic value under no-cap conditions, these concentrations represent

maximums. These concentrations are based upon the assumption that the source input C/C_0 to the groundwater zone is equal to unity. One can see that the maximum concentrations at the POC are far below 1.0 which means that a significant amount of mixing and dilution occur below the Sanitary Landfill. In addition, some constituents undergo degradation which further reduces their concentrations. As expected, concentrations decrease with distance from the source (i.e., POC to POE to UTRC). Table 8.2 lists the simulated relative concentrations at the POC, POE and UTRC at the end of institutional control for the maximum possible source scenario. As one would expect, these concentrations are somewhat larger than the concentrations based upon the more-likely and smaller source dimension.

In Table 8.3, attenuation factors have been calculated between the POC and POE and the POC and UTRC for the conservative source scenario under no-cap conditions. The highest attenuation factor between the POC and the POE is for lead with a K_d of 100 mL/g. This high attenuation is resulting from strong sorption to the aquifer matrix. The second highest value is for tetrachloroethylene (PCE) which has a half life of 70 days. In this case, the attenuating mechanism is degradation of the COC. For most of the organic COCs, the attenuation factors are very high which means that concentrations at the POE are very low in comparison to concentrations at the POC. One exception to this is vinyl chloride (chloroethylene). Because vinyl chloride resides at the end of the PCE degradation chain and has a large half life, this constituent acts very conservatively and is constantly being produced as a result of the constant parent (PCE) source input.

In Table 8.4, attenuation factors have been calculated between the POC and POE and the POC and UTRC for the maximum possible source scenario under no-cap conditions. Changes between the attenuation factors in Tables 8.3 and 8.4 result primarily by the timing and inclusion of additional source blocks in this scenario. For example, if the best-estimate source scenario only inputs a COC over two source quadrants located at the north end of the Sanitary Landfill, then

the flow path available for attenuation is longer than if one assumes that the same COC is inputting over the entire Sanitary Landfill area (i.e., maximum source case).

In Tables 8.5 and 8.6, the absolute concentrations have been calculated at the POC, POE, and UTRC at the end of institutional control for the best-estimate and maximum source scenarios, respectively. These absolute concentrations assume that the input concentration can be represented by the maximum observed POC concentration as listed in Table 3.1 with the exception of daughter products. For daughter products, concentrations are relative to the parent observed POC concentration maximum. Therefore, these concentrations should be considered conservative and maximums. For the three daughter products of tetrachloroethylene, that is, trichloroethylene, trans-1,2-dichloroethene, and chloroethylene, the POC maximum concentrations are not equal to the observed maximum as listed in Table 3.1 because the concentration of the daughter products depends upon the initial concentration of the parent. For trichloroethylene, the simulations overestimate the maximum POC concentration. However, for trans-1,2-dichloroethene and chloroethylene the maximum POC concentrations are underestimated by approximately a factor of two.

From Tables 8.5 and 8.6, it can be seen that the concentrations of most of the COCs are below groundwater standards at the POE as defined by the standards listed in Table 3.1. However, there is one case where this is not the case. The POE calculated concentration for chloroethylene is 175.8 and 221.8 ug/L for the best-estimate and maximum source scenarios, respectively. These values are above the groundwater protection standard of 2 ug/L. The seep line sampling results did not detect chloroethylene at the wetlands but this could be attributed to the fact that chloroethylene has only recently been detected at the landfill (Norrell et al., 1992). The large simulated chloroethylene concentrations at the POE suggest the overconservative nature of the transport calculations. First, chloroethylene has only been identified to date in the western portion of the original Sanitary Landfill. The source dimension assumed for

tetrachloroethylene (PCE), and subsequently chloroethylene, was based upon the cumulative or intersection of source quadrants determined for all members of the tetrachloroethylene chain. For this reason, it is conservatively large. Secondly, there is a finite amount of parent PCE available in the landfill to degrade to chloroethylene. The assumption that PCE is being input to the saturated zone over the thirty years of institutional control is conservative because the simulation continually provides a new source of PCE which implies that the source inventory of PCE never degrades.

8.4 Scenario 1 Capped Conditions

The two scenarios of best-estimate and maximum source loading were also simulated considering that the Sanitary Landfill original and Southern Expansion portions of the landfill were covered with a geosynthetic cap (Scenario 1 cap conditions). The source boundary condition is assumed to be effectively decreased by a factor of ten (97% efficiency) when the cap is emplaced in the middle of 1995. The infiltration rate under no cap conditions is 15 in/yr and under cap conditions is 1.4 in/yr. This transient source creates breakthrough curves which are much more complex than the no-cap conditions and are characterized by building to a peak concentration and then falling to a much reduced concentration at some time after capping (please refer to Figure 7.2).

For an example of the transient behavior of the COC breakthrough curves under capped conditions, Figures 8.1 and 8.2 plot the breakthrough curves for chlorobenzene and 1,4-dichlorobenzene.

Figure 8.1 shows the POC, POE, and UTRC breakthrough curves for chlorobenzene. Chlorobenzene is not considered to degrade under methanogenic conditions (Barker and Wilson, 1992) and is modeled as a stable constituent. From Figure 8.1, one can see the transport of the concentration pulse which moves down gradient from the Sanitary Landfill after capping. The maximum

relative concentration simulated at the POE was 0.166 and the maximum simulated for UTRC was 0.137. Because chlorobenzene is stable, these peak concentrations are fairly close in magnitude with dispersion acting as the primary attenuating mechanism between monitoring points. Between the time the peak occurs at the POE and at UTRC, the maximum discharging concentration is occurring between these two points within the wetlands.

For 1,4-dichlorobenzene, the breakthrough curves at the POE and UTRC are very different. The COC 1,4-dichlorobenzene has a half life of 730 days and a retardation factor of 2.39. Because of degradation of the COC, the breakthrough curves at the POE and UTRC are extremely attenuated. Because the organic COCs have low retardation factors, the half lives along with the source assumptions will control the magnitude of the concentrations simulated at the POE and locations further down gradient. Likewise, the combination of distribution coefficient and half life will govern the timing of the COC peak as it migrates from the Sanitary Landfill and past the POE and through the wetlands to UTRC. Because a pulse moves through the groundwater down gradient of the Sanitary Landfill after capping, the POE does not record the highest simulated concentration independent of time as was generally the case under no-cap conditions. However, the maximum peak concentration and the steady-state maximum concentration (i.e., at year 2025) are highest at the POE. Therefore, these concentrations are conservative for use in risk calculations.

For capped conditions concentrations were tabulated and absolute concentrations were calculated for the peak concentrations and concentrations at the end of institutional control. In addition, attenuation factors were calculated for these two sets of concentration data. Table 8.7 tabulates the relative concentrations simulated at year 2025 for the POC, POE, and UTRC for the best-estimate source scenario under capped conditions. A comparison between Table 8.7 and 8.1 shows that the cap has substantially lowered concentrations for all COCs at late time. Table 8.8 tabulates the peak relative concentrations

simulated at the POC, POE, and UTRC for the best-estimate source scenario under capped conditions. Table 8.9 tabulates the time at which these peak concentrations were observed. If one compares Table 8.8 concentrations with Table 8.1, one can see that the peak concentrations observed under capped conditions almost equal in magnitude those observed without a cap. The reason the peaks are so high relative to uncapped conditions is because the capping takes place late in the loading history and advective groundwater travel times between the POC and the POE are exceedingly fast (i.e., 1 to 5 years). Tables 8.10 through 8.12 correspond to Tables 8.7 through 8.9 but summarize results from the simulations for the maximum possible source scenario under capped conditions. Again a similar behavior is observed when compared to the best-estimate source scenario with the exception that concentrations have increased somewhat and the peak relative concentrations occur later in time.

Table 8.13 tabulates the attenuation factors calculated between the POC and POE and the POC and UTRC for the best-estimate source scenario under capped conditions using relative concentrations taken at the end of institutional control. Comparison of the attenuation factors here with the best-estimate source no-cap attenuation factors (Table 8.3) illustrates that the attenuation factors have increased. Table 8.14 tabulates the attenuation factors calculated between the POC and POE and the POC and UTRC for the best-estimate source scenario under capped conditions using the peak simulated relative concentrations (Table 8.8). The attenuation factors measured between the POC and the POE are in most cases comparable in magnitude to the attenuation factors calculated under no-cap conditions (Table 8.3). As one moves further down gradient to UTRC, the attenuation factors are generally larger than for no-cap conditions because of greater peak attenuation over the larger flow distance. Tables 8.15 and 8.16 tabulate the attenuation factors for the maximum possible source scenario under capped conditions. These attenuation factors are generally smaller than the attenuation factors calculated for the best-estimate source scenario because of the increased source area.

Table 8.17 tabulates the absolute concentrations calculated for the POC, POE, and UTRC for the best-estimate source scenario under capped conditions at year 2025. These absolute concentrations assume that the input concentration can be represented by the maximum observed POC concentration as listed in Table 3.1 with the exception of daughter products. Therefore, these concentrations should be considered conservative maximums. The calculated concentrations are well below groundwater standards at the POE for all COCs. However, the simulations predict that the concentrations will peak in the aquifer after capping and that these concentrations are on the same order as under no-cap conditions. Table 8.18 tabulates the absolute peak concentrations calculated for the POC, POE, and UTRC for the best-estimate source scenario under capped conditions. When compared to no-cap conditions under a similar loading scenario (Table 8.5), one can see that the peak concentrations at the POE are as high as under no-cap conditions. However, the peak concentrations are slightly smaller at points further down gradient.

Similarly, Tables 8.19 and 8.20 tabulate the absolute concentrations at year 2025 and at peak values, respectively, calculated for the POC, POE, and UTRC for the maximum possible source scenario under capped conditions. The overall results are similar to the best-estimate source case and, from Table 8.19, it can be seen that the concentrations at the POE at the year 2025 are calculated to be much less than groundwater standards even under the maximum source conditions with the exception of chloroethylene which accumulates in the groundwater because of the constant parent PCE source term and a relatively high half life.

Figure 8.1 Breakthrough Curves for Chlorobenzene for the Maximum Source Scenario Under Scenario 1 Capping (97% Efficiency)

Figure 8.2 Breakthrough Curves for 1,4-Dichlorobenzene for the Maximum Source Scenario Under Scenario 1 Capping (97% Efficiency)

**Table 8.1 Simulated Relative Concentration at Selected Locations at Year 2025;
No Cap; Conservative Best-Estimate Source Scenario.**

Constituent	Relative Concentration		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	0.1399	0.0077	5.4E-05
Tetrachloroethylene	0.0650	0.00003	1.7E-12
Trichloroethylene	0.0502	0.0009	3.8E-08
Trans 1,2-Dichloroethene	0.0268	0.0048	9.3E-06
Chloroethylene	0.0210	0.0203	0.0052
Dichlorodifluoromethane	0.2353	0.0497	0.0051
Trichlorofluoromethane	0.2313	0.0465	0.0042
Dichloromethane	0.1105	0.0009	6.2E-08
Chlorobenzenes			
1,4-Dichlorobenzene	0.0955	0.0038	0.0001
Chlorobenzene	0.1141	0.0276	0.0277
Aromatics			
Benzene	0.1832	0.0270	0.0060
Ethylbenzene	0.1747	0.0146	0.0024
Xylenes	0.1425	0.0017	0.00002
Metals			
Arsenic	0.1613	0.0547	0.0555
Lead (Kd = 100)	0.0320	< 1E-12	< 1E-12
Lead(Kd = 0.001)	0.2349	0.0731	0.0740
Radioactive			
Tritium	0.1979	0.0430	0.0304

POC = Point of Compliance
POE = Point of Exposure
UTRC = Upper Three Runs Creek

**Table 8.2 Simulated Relative Concentration at Selected Locations at Year 2025;
No Cap; Maximum Possible Source Scenario.**

Constituent	Relative Concentration		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	0.1643	0.0081	0.0001
Tetrachloroethylene	0.0670	0.00003	1.7E-12
Trichloroethylene	0.0586	0.0009	3.9E-08
Trans1,2-Dichloroethene	0.0386	0.0050	9.5E-06
Chloroethylene	0.0312	0.0264	0.0066
Dichlorodifluoromethane	0.2353	0.0497	0.0051
Trichlorofluoromethane	0.2313	0.0465	0.0042
Dichloromethane	0.1105	0.0009	6.2E-08
Chlorobenzenes			
1,4-Dichlorobenzene	0.1904	0.0195	0.0004
Chlorobenzene	0.3523	0.1959	0.1923
Aromatics			
Benzene	0.2649	0.07766	0.01675
Ethylbenzene	0.2508	0.0636	0.0098
Xylenes	0.1746	0.0127	0.0001
Metals			
Arsenic	0.3523	0.1959	0.1923
Lead (Kd = 100)	0.0327	< 1E-12	< 1E-12
Lead(Kd = 0.001)	0.3523	0.1959	0.1923
Radioactive			
Tritium	0.3274	0.1573	0.1083

Table 8.3 Attenuation Factors at Year 2025; No Cap; Conservative Best-Estimate Source Scenario.

Constituent	Porous Media Attenuation Factor	
	POE	UTRC
Chlorinated Aliphatics		
1,1,1-Trichloroethane	18.1	2586
Tetrachloroethylene	2468	> 1E+10
Trichloroethylene	55.8	1.3E+06
Trans1,2-Dichloroethene	5.6	2893
Chloroethylene	1.0	4.1
Dichlorodifluoromethane	4.7	46.1
Trichlorofluoromethane	5.0	55.1
Dichloromethane	122.8	1.8E+06
Chlorobenzenes		
1,4-Dichlorobenzene	25.3	1206
Chlorobenzene	4.1	4.1
Aromatics		
Benzene	6.8	30.6
Ethylbenzene	11.9	74.1
Xylenes	82.7	8180
Metals		
Arsenic	2.9	2.9
Lead (Kd=100)	> 1E10	> 1E10
Lead(Kd=0.001)	3.2	3.2
Radioactive		
Tritium	4.6	6.5

Table 8.4 Attenuation Factors at Year 2025; No Cap; Maximum Possible Source Scenario.

Constituent	Porous Media Attenuation Factor	
	POE	UTRC
Chlorinated Aliphatics		
1,1,1-Trichloroethane	20.2	2853
Tetrachloroethylene	2545	> 1E+10
Trichloroethylene	64.8	1.5E+06
Trans1,2-Dichloroethene	7.7	4038
Chloroethylene	1.2	4.7
Dichlorodifluoromethane	4.7	46.5
Trichlorofluoromethane	5.0	54.8
Dichloromethane	118.3	1.8E+06
Chlorobenzenes		
1,4-Dichlorobenzene	9.8	486.0
Chlorobenzene	1.8	1.8
Aromatics		
Benzene	3.4	15.8
Ethylbenzene	3.9	25.6
Xylenes	13.7	1469
Metals		
Arsenic	1.8	1.8
Lead (Kd = 100)	> 1E10	> 1E10
Lead(Kd = 0.001)	1.8	1.8
Radioactive		
Tritium	2.1	3.0

Table 8.5 Absolute Concentrations at Selected Locations at Year 2025; No Cap; Conservative Best-Estimate Source Scenario.

Constituent	Absolute Concentration (ug/L)		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	249.0	13.7	0.1
Tetrachloroethylene	564.0	0.2	0.0
Trichloroethylene	436.0	7.8	0.0
Trans1,2-Dichloroethene	232.8	41.8	0.1
Chloroethylene	182.1	175.8	44.7
Dichlorodifluoromethane	74.6	15.8	1.6
Trichlorofluoromethane	580.0	116.6	10.5
Dichloromethane	103.0	0.8	0.0
Chlorobenzenes			
1,4-Dichlorobenzene	99.0	3.9	0.1
Chlorobenzene	52.0	12.6	12.6
Aromatics			
Benzene	8.0	1.2	0.3
Ethylbenzene	52.6	4.4	0.7
Xylenes	28.0	0.3	0.0
Metals			
Arsenic	37.6	12.8	12.9
Lead (Kd = 100)	40.0	0.0	0.0
Lead(Kd = 0.001)	40.0	12.5	12.6
Radioactive			
Tritium	101.0	21.9	15.5

Table 8.6 Absolute Concentrations at Selected Locations at Year 2025; No Cap; Maximum Possible Source Scenario.

Constituent	Absolute Concentration (ug/L)		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	249.0	12.3	0.1
Tetrachloroethylene	564.0	0.2	0.0
Trichloroethylene	492.7	7.6	0.0
Trans1,2-Dichloroethene	324.3	41.9	0.1
Chloroethylene	262.2	221.8	55.2
Dichlorodifluoromethane	74.6	15.8	1.6
Trichlorofluoromethane	580.0	116.6	10.6
Dichloromethane	103.0	0.9	0.0
Chlorobenzenes			
1,4-Dichlorobenzene	99.0	10.1	0.2
Chlorobenzene	52.0	28.9	28.4
Aromatics			
Benzene	8.0	2.3	0.5
Ethylbenzene	52.6	13.3	2.1
Xylenes	28.0	2.0	0.0
Metals			
Arsenic	37.6	20.9	20.5
Lead (Kd = 100)	40.0	0.0	0.0
Lead(Kd=0.001)	40.0	22.2	21.8
Constituent	Absolute Concentration (pCi/mL)		
	POC	POE	UTRC
Radioactive			
Tritium	101.0	48.5	33.4

Table 8.7 Simulated Relative Concentration at Selected Locations at Year 2025;
Full Cap at 97% Efficiency; Conservative Best-Estimate Source
Scenario.

Constituent	Relative Concentration		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	0.0227	0.0007	4.7E-06
Tetrachloroethylene	0.0095	1.8E-06	< 1E-12
Trichloroethylene	0.0086	0.00007	2.8E-09
Trans1,2-Dichloroethene	0.0050	0.0004	7.6E-07
Chloroethylene	0.0035	0.0022	0.0005
Dichlorodifluoromethane	0.0402	0.0051	0.0005
Trichlorofluoromethane	0.0396	0.0047	0.0004
Dichloromethane	0.0175	0.00008	4.5E-09
Chlorobenzenes			
1,4-Dichlorobenzene	0.0157	0.0004	7.0E-06
Chlorobenzene	0.0188	0.0031	0.0030
Aromatics			
Benzene	0.0307	0.0028	0.0006
Ethylbenzene	0.0292	0.0015	0.0002
Xylenes	0.0231	0.0002	1.4E-06
Metals			
Arsenic	0.0242	0.0051	0.0050
Lead (Kd = 100)	0.0141	< 1E-12	< 1E-12
Lead(Kd = 0.001)	0.0398	0.0081	0.0079
Radioactive			
Tritium	0.0334	0.0047	0.0032

Table 8.8 Simulated Peak Relative Concentration at Selected Locations; Full Cap at 97% Efficiency; Conservative Best-Estimate Source Scenario.

Constituent	Relative Peak Concentration		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	0.1399	0.0078	0.00005
Tetrachloroethylene	0.0650	0.00003	1.7E-12
Trichloroethylene	0.0502	0.0009	3.8E-08
Trans1,2-Dichloroethene	0.0271	0.0049	9.0E-06
Chloroethylene	0.0219	0.0204	0.0048
Dichlorodifluoromethane	0.2332	0.0443	0.0039
Trichlorofluoromethane	0.2293	0.0410	0.0032
Dichloromethane	0.1105	0.0009	5.8E-08
Chlorobenzenes			
1,4-Dichlorobenzene	0.0951	0.0028	0.00005
Chlorobenzene	0.1141	0.0277	0.0264
Aromatics			
Benzene	0.1831	0.0249	0.0050
Ethylbenzene	0.1747	0.0140	0.0021
Xylenes	0.1425	0.0017	0.00002
Metals			
Arsenic	0.1613	0.0551	0.0553
Lead (Kd = 100)	0.0151	< 1E-12	< 1E-12
Lead(Kd = 0.001)	0.2349	0.0725	0.0697
Radioactive			
Tritium	0.1979	0.0432	0.0296

Table 8.9 Year of Simulated Peak Relative Concentration at Selected Locations; Full Cap at 97% Efficiency; Conservative Best-Estimate Source Scenario.

Constituent	Year of Relative Peak Concentration		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	1990.5	1995.9	1999.9
Tetrachloroethylene	1988.5	1995.8	1997.3
Trichloroethylene	1989.4	1995.9	1998.1
Trans1,2-Dichloroethene	1995.6	1995.9	1998.9
Chloroethylene	1996.0	1996.0	2000.8
Dichlorodifluoromethane	1994.8	1995.9	1998.1
Trichlorofluoromethane	1995.5	1996.8	2002.0
Dichloromethane	1988.5	1995.8	1997.6
Chlorobenzenes			
1,4-Dichlorobenzene	1995.5	1999.3	2007.8
Chlorobenzene	1990.8	1996.0	2004.8
Aromatics			
Benzene	1994.6	1997.4	2002.5
Ethylbenzene	1993.3	1996.8	2006.4
Xylenes	1991.1	1996.0	2003.0
Metals			
Arsenic	1989.5	1995.9	2001.3
Lead (Kd = 100)	2045.0	2045.0	2045.0
Lead(Kd=0.001)	1994.4	1996.0	2002.6
Radioactive			
Tritium	1991.8	1995.9	2001.6

Table 8.10 Simulated Relative Concentration at Selected Locations at Year 2025;
Full Cap at 97% Efficiency; Maximum Possible Source Scenario.

Constituent	Relative Concentration		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	0.0271	0.0008	5.0E-06
Tetrachloroethylene	0.0099	1.8E-06	1.0E-13
Trichloroethylene	0.0101	0.00007	2.8E-09
Trans1,2-Dichloroethene	0.0070	0.0005	7.8E-07
Chloroethylene	0.0052	0.0028	0.0007
Dichlorodifluoromethane	0.0402	0.0051	0.0005
Trichlorofluoromethane	0.0396	0.0047	0.0004
Dichloromethane	0.0175	0.00008	4.5E-09
Chlorobenzenes			
1,4-Dichlorobenzene	0.0322	0.0019	0.00004
Chlorobenzene	0.0594	0.0219	0.0208
Aromatics			
Benzene	0.0451	0.0082	0.0017
Ethylbenzene	0.0428	0.0066	0.0010
Xylenes	0.0292	0.0012	0.00001
Metals			
Arsenic	0.0594	0.0219	0.0206
Lead (Kd = 100)	0.0155	<1E-12	<1E-12
Lead(Kd = 0.001)	0.0594	0.0219	0.0206
Radioactive			
Tritium	0.0554	0.0173	0.0114

Table 8.11 Simulated Peak Relative Concentration at Selected Locations; Full Cap at 97% Efficiency; Maximum Possible Source Scenario.

Constituent	Relative Peak Concentration		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	0.1643	0.0082	0.00005
Tetrachloroethylene	0.0670	0.00003	1.7E-12
Trichloroethylene	0.0586	0.0009	3.8E-08
Trans1,2-Dichloroethene	0.0391	0.0050	9.3E-06
Chloroethylene	0.0333	0.0264	0.0061
Dichlorodifluoromethane	0.2332	0.0443	0.0039
Trichlorofluoromethane	0.2293	0.0410	0.0032
Dichloromethane	0.1105	0.0009	5.8E-08
Chlorobenzenes			
1,4-Dichlorobenzene	0.1881	0.0151	0.0002
Chlorobenzene	0.3361	0.1656	0.1374
Aromatics			
Benzene	0.2609	0.0693	0.0130
Ethylbenzene	0.2433	0.0504	0.0063
Xylenes	0.1738	0.0108	0.00008
Metals			
Arsenic	0.3443	0.1776	0.1530
Lead (Kd = 100)	0.0159	< 1E-12	< 1E-12
Lead(Kd = 0.001)	0.3443	0.1776	0.1530
Radioactive			
Tritium	0.3213	0.1427	0.0863

**Table 8.12 Year of Simulated Peak Relative Concentration at Selected Locations;
Full Cap at 97% Efficiency; Maximum Possible Source Scenario.**

Constituent	Year of Relative Peak Concentration		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	1992.1	1995.9	1999.9
Tetrachloroethylene	1989.8	1995.8	1997.3
Trichloroethylene	1990.8	1995.9	1998.1
Trans 1,2-Dichloroethene	1995.6	1995.9	1998.9
Chloroethylene	1996.4	1996.0	2000.9
Dichlorodifluoromethane	1995.5	1996.5	2001.8
Trichlorofluoromethane	1995.5	1996.8	2002.0
Dichloromethane	1994.8	1995.9	1998.1
Chlorobenzenes			
1,4-Dichlorobenzene	1995.5	1997.5	2006.6
Chlorobenzene	1995.5	1997.6	2005.8
Aromatics			
Benzene	1995.5	1996.8	2002.0
Ethylbenzene	1995.5	1997.6	2006.4
Xylenes	1995.5	1996.8	2003.4
Metals			
Arsenic	1995.5	1996.8	2002.4
Lead (Kd = 100)	1998.3	2045.0	2045.0
Lead(Kd = 0.001)	1995.5	1996.8	2002.4
Radioactive			
Tritium	1995.5	1996.8	2002.1

Table 8.13 Attenuation Factors at Year 2025; Full Cap at 97% Efficiency;
Conservative Best-Estimate Source Scenario.

Constituent	Porous Media Attenuation Factor	
	POE	UTRC
Chlorinated Aliphatics		
1,1,1-Trichloroethane	31.3	4838
Tetrachloroethylene	5311	> 1E+10
Trichloroethylene	118.0	3.1E+06
Trans1,2-Dichloroethene	11.1	6539
Chloroethylene	1.6	6.8
Dichlorodifluoromethane	7.9	82.6
Trichlorofluoromethane	8.4	98.0
Dichloromethane	231.1	3.9E+06
Chlorobenzenes		
1,4-Dichlorobenzene	44.4	2260.7
Chlorobenzene	6.1	6.3
Aromatics		
Benzene	11.0	52.1
Ethylbenzene	19.6	126.3
Xylenes	149.3	15970
Metals		
Arsenic	4.8	4.9
Lead (Kd=100)	> 1E+10	> 1E+10
Lead(Kd=0.001)	4.9	5.0
Radioactive		
Tritium	7.1	10.5

Table 8.14 Attenuation Factors Evaluated with Peak Concentrations; Full Cap at 97% Efficiency; Conservative Best-Estimate Source Scenario.

Constituent	Porous Media Attenuation Factor	
	POE	UTRC
Chlorinated Aliphatics		
1,1,1-Trichloroethane	18.0	2743.1
Tetrachloroethylene	2456	> 1E+10
Trichloroethylene	55.4	1.3E+06
Trans1,2-Dichloroethene	5.6	3001
Chloroethylene	1.1	4.6
Dichlorodifluoromethane	5.3	59.6
Trichlorofluoromethane	5.6	71.5
Dichloromethane	119.7	1.9E+06
Chlorobenzenes		
1,4-Dichlorobenzene	34.2	2027
Chlorobenzene	4.1	4.3
Aromatics		
Benzene	7.3	36.9
Ethylbenzene	12.4	84.8
Xylenes	83.2	8845
Metals		
Arsenic	2.9	2.9
Lead (Kd=100)	> 1E+10	> 1E+10
Lead(Kd=0.001)	3.2	3.4
Radioactive		
Tritium	4.6	6.7

Table 8.15 Attenuation Factors at Year 2025; Full Cap at 97% Efficiency;
Maximum Possible Source Scenario.

Constituent	Porous Media Attenuation Factor	
	POE	UTRC
Chlorinated Aliphatics		
1,1,1-Trichloroethane	35.4	5438
Tetrachloroethylene	5509	> 1E+10
Trichloroethylene	137.7	3.6E+06
Trans1,2-Dichloroethene	15.3	8992
Chloroethylene	1.8	7.9
Dichlorodifluoromethane	7.9	82.6
Trichlorofluoromethane	8.4	98.0
Dichloromethane	231.1	3.9E+06
Chlorobenzenes		
1,4-Dichlorobenzene	17.1	910.8
Chlorobenzene	2.7	2.8
Aromatics		
Benzene	5.5	27.1
Ethylbenzene	6.5	44.0
Xylenes	24.5	2843
Metals		
Arsenic	2.7	2.9
Lead (Kd = 100)	> 1E+10	> 1E+10
Lead(Kd=0.001)	2.7	2.9
Radioactive		
Tritium	3.2	4.8

Table 8.16 Attenuation Factors Evaluated with Peak Concentrations; Full Cap at 97% Efficiency; Maximum Possible Source Scenario.

Constituent	Porous Media Attenuation Factor	
	POE	UTRC
Chlorinated Aliphatics		
1,1,1-Trichloroethane	20.0	3025
Tetrachloroethylene	2533	> 1E+10
Trichloroethylene	64.4	1.5E+06
Trans1,2-Dichloroethene	7.8	4203
Chloroethylene	1.3	5.4
Dichlorodifluoromethane	5.3	59.6
Trichlorofluoromethane	5.6	71.5
Dichloromethane	119.7	1.9E+06
Chlorobenzenes		
1,4-Dichlorobenzene	12.5	796.4
Chlorobenzene	2.0	2.4
Aromatics		
Benzene	3.8	20.1
Ethylbenzene	4.8	38.3
Xylenes	16.1	2109
Metals		
Arsenic	1.9	2.3
Lead (Kd=100)	> 1E+10	> 1E+10
Lead(Kd=0.001)	1.9	2.3
Radioactive		
Tritium	2.3	3.7

Table 8.17 Absolute Concentrations at Selected Locations at Year 2025; Full Cap at 97% Efficiency; Conservative Best-Estimate Source Scenario.

Constituent	Absolute Concentration (ug/L)		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	40.3	1.3	0.0
Tetrachloroethylene	82.6	0.0	0.0
Trichloroethylene	75.0	0.6	0.0
Trans1,2-Dichloroethene	43.0	3.9	0.0
Chloroethylene	30.3	19.1	4.5
Dichlorodifluoromethane	12.8	1.6	0.2
Trichlorofluoromethane	99.2	11.8	1.0
Dichloromethane	16.3	0.1	0.0
Chlorobenzenes			
1,4-Dichlorobenzene	16.3	0.4	0.0
Chlorobenzene	8.6	1.4	1.4
Aromatics			
Benzene	1.3	0.1	0.0
Ethylbenzene	8.8	0.4	0.1
Xylenes	4.5	0.0	0.0
Metals			
Arsenic	5.6	1.2	1.2
Lead (Kd = 100)	17.6	0.0	0.0
Lead(Kd = 0.001)	6.8	1.4	1.3
Radioactive			
Constituent	Absolute Concentration (pCi/mL)		
	POC	POE	UTRC
Radioactive			
Tritium	17.0	2.4	1.6

Table 8.18 Absolute Peak Concentrations at Selected Locations; Full Cap at 97% Efficiency; Conservative Best-Estimate Source Scenario.

Constituent	Absolute Concentration (ug/L)		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	249.0	13.8	0.1
Tetrachloroethylene	564.0	0.2	0.0
Trichloroethylene	436.0	7.9	0.0
Trans1,2-Dichloroethene	235.1	42.2	0.1
Chloroethylene	190.2	176.9	41.6
Dichlorodifluoromethane	73.9	14.0	1.2
Trichlorofluoromethane	575.0	102.9	8.0
Dichloromethane	103.0	0.9	0.0
Chlorobenzenes			
1,4-Dichlorobenzene	98.7	2.9	0.0
Chlorobenzene	52.0	12.6	12.0
Aromatics			
Benzene	8.0	1.1	0.2
Ethylbenzene	52.6	4.2	0.6
Xylenes	28.0	0.3	0.0
Metals			
Arsenic	37.6	12.8	12.9
Lead (Kd = 100)	18.9	0.0	0.0
Lead(Kd = 0.001)	40.0	12.3	11.9
Constituent	Absolute Concentration (pCi/mL)		
	POC	POE	UTRC
Radioactive			
Tritium	101.0	22.0	15.1

Table 8.19 Absolute Concentrations at Selected Locations at Year 2025; Full Cap at 97% Efficiency; Maximum Possible Source Scenario.

Constituent	Absolute Concentration (ug/L)		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	41.0	1.2	0.0
Tetrachloroethylene	83.1	0.0	0.0
Trichloroethylene	85.1	0.6	0.0
Trans1,2-Dichloroethene	59.0	3.9	0.0
Chloroethylene	43.8	23.9	5.5
Dichlorodifluoromethane	12.8	1.6	0.2
Trichlorofluoromethane	99.2	11.8	1.0
Dichloromethane	16.3	0.1	0.0
Chlorobenzenes			
1,4-Dichlorobenzene	16.7	1.0	0.0
Chlorobenzene	8.8	3.2	3.1
Aromatics			
Benzene	1.4	0.2	0.1
Ethylbenzene	9.0	1.4	0.2
Xylenes	4.7	0.2	0.0
Metals			
Arsenic	6.3	2.3	2.2
Lead (Kd = 100)	18.9	0.0	0.0
Lead(Kd=0.001)	6.7	2.5	2.3
Radioactive			
Constituent	Absolute Concentration (pCi/mL)		
	POC	POE	UTRC
Tritium	17.1	5.3	3.5

Table 8.20 Absolute Peak Concentrations at Selected Locations; Full Cap at 97% Efficiency; Maximum Possible Source Scenario.

Constituent	Absolute Concentration (ug/L)		
	POC	POE	UTRC
Chlorinated Aliphatics			
1,1,1-Trichloroethane	249.0	12.4	0.1
Tetrachloroethylene	564.0	0.2	0.0
Trichloroethylene	492.7	7.7	0.0
Trans1,2-Dichloroethene	328.9	42.2	0.1
Chloroethylene	280.1	222.4	51.4
Dichlorodifluoromethane	73.9	14.0	1.2
Trichlorofluoromethane	575.0	102.9	8.0
Dichloromethane	103.0	0.9	0.0
Chlorobenzenes			
1,4-Dichlorobenzene	97.8	7.8	0.1
Chlorobenzene	49.6	24.4	20.3
Aromatics			
Benzene	7.9	2.1	0.4
Ethylbenzene	51.0	10.6	1.3
Xylenes	27.9	1.7	0.0
Metals			
Arsenic	36.7	19.0	16.3
Lead (Kd = 100)	19.4	0.0	0.0
Lead(Kd = 0.001)	39.1	20.2	17.4
Constituent	Absolute Concentration (pCi/mL)		
	POC	POE	UTRC
Radioactive			
Tritium	99.1	44.0	26.6

9.0 LATERAL EXTENT OF CONTAMINATION AND DERIVATION OF THE GROUNDWATER TO SURFACE WATER DILUTION FACTOR

The transport modeling performed in support of the ACL Demonstrations presented in the preceding Chapter 8 are considered conservative because the vertical-cross-sectional transport model simulates plume centerline concentrations which represent maximum concentrations. The concentrations are maximums because lateral spread of contaminants as a result of mechanical mixing and diffusion from the centerline is not modeled. However, the definition of the lateral extent of contamination and the potential discharge area and volumetric flux where contaminated groundwater discharges to the surface is required for the calculation of a groundwater to surface water dilution factor for the estimation of COC concentrations in UTRC. The area of contaminant discharge to the wetlands and the volumetric flux rate of contaminants to that area are also required by the risk assessors to define conditions for inhalation exposure risk scenarios. A simplified approach using an analytical model was implemented to determine lateral extent of contamination and a groundwater to surface water dilution factor.

9.1 Determination of Lateral Extent of Contamination and Discharge Area

The following describes the method used to determine the potential zone of contaminated groundwater in the lateral direction. A closed-form analytic advection-dispersion model was used to delineate the potential extent of discharging contaminated groundwater. The model used is an analytic solution to the advection-dispersion equation for a constant concentration, finite-width source. The analytic solution was used to predict conservative constituent concentrations areally down gradient from the Sanitary Landfill in an unidirectional flow field oriented parallel to the vertical cross-sectional transport model described in Chapter 4. The analytic solution used is the Extended Pulse model (Domenico and Robbins, 1985; Domenico, 1987) which is also known as the EPA Vertical and

Horizontal Spread (VHS) analytic model. The assumptions inherent to the closed form solution are:

- Rectangular plan AR source oriented normal to the flow direction and located within the saturated zone over some finite source thickness in the vertical direction,
- Constant concentration source,
- Constant and unidirectional flow field, and
- One dimensional advective velocity with three dimensions of dispersion and diffusion.

The solution includes retardation and first order decay; however, these attenuating processes were not considered in this calculation as a measure of conservatism. The governing analytic equation can be written as:

$$C(x, y, z, t) = \left(\frac{C_o}{8} \right) \operatorname{erfc} \left[\frac{(x - vt)}{2(a_x t)^{1/2}} \right] \left\{ \operatorname{erf} \left[\frac{(y + Y/2)}{2(a_y t)^{1/2}} \right] - \operatorname{erf} \left[\frac{(y - Y/2)}{2(a_y t)^{1/2}} \right] \right\} \left\{ \operatorname{erf} \left[\frac{(z + Z)}{2(a_z t)^{1/2}} \right] - \operatorname{erf} \left[\frac{(z - Z)}{2(a_z t)^{1/2}} \right] \right\} \quad (9.1)$$

where C_o is the initial source concentration, x , y , and z are axial coordinates, v is the average seepage velocity, t is time, a_x , a_y , and a_z are the longitudinal and transverse dispersivities, Y is the source dimension in the y -direction, and Z is the source dimension in the z -direction.

This equation assumes that spreading in the z-direction is confined at the top of the saturated zone by the water table and assumes that spreading in the y-direction occurs symmetrically and infinitely about the center line of transport. Because the bottom of the Steed Pond Aquifer is confined by a no flow boundary, y-direction spreading is limited to the thickness of the saturated zone. To account for this, the analytic equation above was solved using the method of images (de Marsily, 1986) to account for the zero flux boundary at the bottom of the aquifer.

Calculations that maximize lateral spreading predicted by the analytical solution are conservative because they maximize the estimate of the potentially contaminated groundwater discharge area. For this reason, the origin of the analytic source region was placed the furthest distance up gradient from UTRC but within the Sanitary Landfill. The source dimension must be specified as a plane oriented normal to the unidirectional flow field and thus requires specification of a Y-source dimension and a Z-source dimension (see Figure 9.1). The Y-source dimension was conservatively taken to be equal to the maximum width of the original portion of the Sanitary Landfill which is approximately 1220 ft. The Z-source dimension is calculated to ensure that the volumetric flux flowing through the surficial area of the Sanitary Landfill as a result of recharge is equal to the volumetric flux flowing through the analytic source plane assumed to be normal to the flow direction. The Z-source dimension can be calculated as

$$\text{Z source dimension} = (Q_{\text{Sanitary Landfill recharge}} / Q_{\text{aquifer}}) \times b \quad (9.2)$$

where b is the average saturated thickness of the aquifer, $Q_{\text{Sanitary Landfill recharge}}$ is the volumetric flux passing through the source area of the Sanitary Landfill due to recharge, and Q_{aquifer} is equal to the amount of volumetric flux passing underneath the Sanitary Landfill source measured normal to the flow direction over the saturated thickness of the aquifer. This formula provides that the volumetric flux passing through the Sanitary Landfill source area is the same as will pass through

the analytical source plane oriented in a Y-Z plane normal to the flow direction (please see Figure 9.1).

The area of the original and Southern Expansion of the Sanitary Landfill is approximately $2.32 \times 10^6 \text{ ft}^2$. The infiltration rate from the calibrated areal flow model is 15 in/year or 1.25 ft/year. This equates to a vertical volumetric flux through the original and southern sector portions of the Sanitary Landfill of 7,955 ft^3/day ($Q_{\text{Sanitary Landfill recharge}}$). The average groundwater volumetric flux occurring below the center of the Sanitary Landfill can be estimated by the simple relationship

$$Q_{\text{aquifer}} = v \times A \times \phi \quad (9.3)$$

where v is the seepage velocity of the aquifer which can be determined from particle tracking in the areal model, A is the cross-sectional area over which the volumetric flow is desired (the product of Y-source dimension and saturated thickness), and ϕ is the medium porosity (0.2). The average saturated thickness between the original Sanitary Landfill area and UTRC is approximately 100 ft. Therefore, the area of the aquifer immediately under the Sanitary Landfill measured normal to the flow direction is equal to 1220 ft x 100 ft or $1.22 \times 10^5 \text{ ft}^2$. The average seepage velocity can be calculated between the drop point located at the center of the northern extent of the original Sanitary Landfill to UTRC. Based on a particle tracking calculation using the steady-state calibrated flow model from Section 4, the average seepage velocity between the northern extent of the Sanitary Landfill source and UTRC is 1.24 ft/day. Therefore, the aquifer volumetric flux (Q_{aquifer}) occurring immediately below the Sanitary Landfill is approximately $3.03 \times 10^4 \text{ ft}^3/\text{day}$. With the two above volumetric flow rates, the calculated Z-source dimension is equal to 26.3 ft (using Equation 9.2). The source boundary condition is not equivalent to the type of boundary condition imposed in the cross-sectional transport model used to support the ACL Demonstrations. However, the analytic boundary condition used here provides larger down-gradient concentrations

because of the location of the source in the saturated zone and the relatively large source plane Z-source dimension relative to the saturated thickness of Aquifer Unit I/IIC. Therefore, the analytic is conservative because it provides higher plume concentrations and will also result in larger predicted lateral migration and a larger potentially contaminated region.

The Domenico and Robbins (1985) model was used to simulate a steady-state concentration profile. The source width was taken to be equal to the maximum Sanitary Landfill width normal to flow (1220 ft) and the source plane was positioned parallel to the furthest up-gradient boundary of the original landfill to conservatively provide the largest degree of lateral spreading (see Figure 9.2). The dispersivities for the x, y, and z coordinates were 100, 10, and 1 ft, respectively, which is consistent with those used for the ACL transport model (see Section 5.2). The model parameters used in the simulation are listed in Table 9.1. Figure 9.2 shows the plume x-y slice at a z coordinate of 0 feet. This is the maximum concentration profile in the x-y plane.

The lateral extent of the plume as defined by the 0.005 relative concentration contour is taken to define the lateral extent of the region of potentially contaminated discharging water from the Sanitary Landfill. This limit was chosen because it is a conservative limit to contamination with over 99 % of the potential contaminant mass falling within this contour. Also, the concentration of 0.005 C/Co is sufficiently small so that no COC would occur at concentrations exceeding MCLs outside of this contour. Figure 9.3 shows the extent of the 0.005 relative concentration contour superimposed upon the Sanitary Landfill local model area. Also shown on the figure are the wetland boundary conditions as well as UTRC.

By superimposing the potentially contaminated groundwater discharge area defined by the 0.005 relative concentration contour on the steady-state calibrated two-dimensional areal flow model grid, estimates of contaminant discharge area

and volumetric flux can be determined. The potentially contaminated discharge area in the wetlands is estimated to be $5.88 \times 10^6 \text{ ft}^2$. The area within UTRC which is potentially within the contaminant plume is $2.6 \times 10^5 \text{ ft}^2$. The total potential discharging area for a contaminant plume is $6.14 \times 10^6 \text{ ft}^2$. The volumetric flux of potentially contaminated groundwater in the wetlands is $88,869 \text{ ft}^3/\text{day}$ (1.03 cfs) and the volumetric flux of potentially contaminated groundwater flowing directly into UTRC is $45,871 \text{ ft}^3/\text{day}$ (0.53 cfs) as calculated by the calibrated areal flow model. The total volumetric flux of potentially contaminated groundwater from both the wetlands and UTRC is $134,740 \text{ ft}^3/\text{day}$ (1.56 cfs) as calculated by the areal flow model. The maximum length across the potentially contaminated groundwater discharge area measured parallel to flow is approximately 3,240 ft. The maximum diagonal distance measured across the potentially contaminated groundwater discharge area is 3,650 ft.

9.2 Groundwater to Surface Water Dilution Factor

The groundwater and surface water flowing down gradient from the Sanitary Landfill ultimately discharge into Upper Three Runs Creek (UTRC). Based upon maximum POE concentrations in Chapter 8, one can project maximum COC concentrations in UTRC assuming instantaneous dilution. The assumption of instantaneous dilution assumes that COC concentrations entering the surface water body (i.e., UTRC) will mix uniformly and very quickly with the UTRC volumetric flow volume. Because the surface water body volumetric flux is significantly greater than the potentially contaminated surface and groundwater flowing from the Sanitary Landfill, the concentrations of COCs as measured in the surface water will be significantly decreased even when not considering attenuation processes such as volatilization and degradation which will also occur.

Assuming instantaneous dilution, a simple groundwater to surface water concentration reduction factor can be calculated which one can divide the groundwater maximum concentration by to receive a perfectly mixed surface water

concentration in UTRC some distance down gradient of the Sanitary Landfill discharge area. The groundwater to surface water dilution factor can be defined as the UTRC volumetric flow rate measured immediately up gradient of the Sanitary Landfill discharge area divided by the volumetric flow rate of contaminated groundwater discharge to the wetlands and UTRC.

The total potentially contaminated groundwater volumetric flux entering UTRC from the potential contaminant plume emanating from the Sanitary Landfill was calculated above from the calibrated areal flow model. The estimate is equal to 1.56 cfs. To be conservative, the volumetric flow rate measured in UTRC upstream of the Sanitary Landfill contributing zone should be a minimum flow rate. Gaging Station C, (see Figure 1.2) is located just upstream of the potential contaminant plume discharge area as defined above. The 7-day, 10-year (7Q10) low flow rate in UTRC measured at gaging Station C is 97.3 cfs. By dividing the UTRC low flow rate by the volumetric flow rate contributing from the potential Sanitary Landfill contaminant discharge area ($97.3/1.56$) one arrives at a groundwater to surface water dilution factor of 62.

$$Z = \left(\frac{Q_{\text{SLF recharge}}}{Q_{\text{AQUIFER}}} \right) * b$$

Figure 9.1 Schematic Representation of Sanitary Landfill Source Area and Equivalent Source Area for Analytic Solution

Figure 9.2 Relative Concentration Distribution for Analytic Calculation

Figure 9.3 Delineation of Potential Wetland Discharge Area for Contaminated Groundwater

Table 9.1 Input Parameters Used to Analytically Estimate Lateral Extent of Contamination

PARAMETER	INPUT VALUE
Longitudinal Dispersivity	100 ft
Transverse Y Dispersivity	10 ft
Transverse Z Dispersivity	1 ft
Initial Relative Concentration	1.0
Y Source Dimension	1220 ft
Z Source Dimension	26.3 ft
Average Seepage Velocity	1.24 ft/day

10.0 STUDY SUMMARY AND CONCLUSIONS

This study documented by this report had several objectives. The broad objective of the modeling study was to characterize and quantify the dominant groundwater flow and transport processes acting at and in the vicinity of the Sanitary Landfill. More specifically, there were also several more focused objectives. The first of these specific objectives was to develop a local-scale flow model for the Sanitary Landfill vicinity which was capable of examining effects upon the local groundwater regime resulting from various capping scenarios. A second objective of this study was to develop a flow and transport model for the Sanitary Landfill vicinity to provide supporting transport calculations for the Sanitary Landfill Alternative Concentration Limit (ACL) Demonstrations. A brief summary of these studies and the conclusions from them will be described in the following Section 10.1. Following this will be Section 10.2 which will discuss data needs for the Sanitary Landfill vicinity which could help to improve future modeling tasks.

10.1 Summary and Conclusions

A two-dimensional areal flow model was constructed for the Sanitary Landfill and vicinity using MODFLOW and the model was calibrated to steady-state conditions. The local-scale flow model reproduced the selected performance measures within the performance measure uncertainties prescribed before calibration. The calibration did not require heterogeneous application of medium hydraulic properties or recharge. Based upon the lack of field-scale knowledge regarding transmissivity and recharge, the homogeneous model is believed to be consistent with the current understanding of the hydrogeologic conditions at the Sanitary Landfill. The steady-state calibrated hydraulic conductivity and recharge values for the areal model were 35 ft/day and 15 in/yr, respectively. These values fall within the potential range of values expected for these parameters in the Sanitary Landfill vicinity. A sensitivity analysis found that an infinite selection of hydraulic conductivities and recharge rates could match the observed heads

adequately. However, based upon a realistic range of potential recharge rates, the hydraulic conductivity of Aquifer Unit I/IIC could potentially range by a factor of 2. A comparison to potential flow gains to UTRC from the local-scale model was found to be consistent with estimates between stream flow gages and also consistent with the A/M regional model (Beaudoin, 1991). The requirement for an appropriate agreement with potential flow gains to UTRC provided a further constraint on the model by which to calibrate.

A vertical cross sectional flow and transport model was also developed for the Sanitary Landfill vicinity to provide a means to examine flow paths in the vertical dimension and to support ACL Demonstrations through transport calculations for selected Sanitary Landfill COCs. SWIFT II was selected as the model to use because it provided the ability to model chain-decay processes as well as handle spatial and temporal variations in contaminant source loading. The vertical flow model was based upon the calibrated areal model and both the hydraulic conductivity and recharge calibrated values were adopted for consistency. Because characterization of the COC plumes with monitoring wells away from the Sanitary Landfill did not exist, the transport model was not able to be calibrated through traditional methods of history matching.

The areal steady-state calibrated flow model was used to quantify the areal effects of capping of the Sanitary Landfill on the local groundwater system. Two different capping scenarios were examined based upon the area of the Sanitary Landfill considered to be capped. For these two scenarios a range of cap efficiencies were simulated. The cap efficiencies were 100%, 95%, 90%, and 85%. The maximum effects upon the flow regime were from simulations with Scenario 1 capping (largest capped area) and a cap efficiency of 100% (i.e., zero simulated infiltration). The effects of all capping scenarios examined were minimal under steady-state conditions with head declines in the vicinity of the Sanitary Landfill never exceeding 1 foot. The transient simulation of head decreases showed the head surface would approach a new approximate steady-state condition within

about two years. This calculation assumes no storage capacity within the unsaturated zone. The velocity field is increased up gradient of the area of maximum head decline and is decreased in the area down gradient of the area of maximum head decline. The maximum groundwater velocity decrease was slightly over 5% and increase was 2.5% for Scenario 1 capping with an efficiency of 100% as compared to no-cap conditions. The maximum increase in particle travel time to the seepage line from the Sanitary Landfill was 4.5% for Scenario 1 capping with an efficiency of 100% as compared to no-cap conditions. The maximum increase in particle travel time to UTRC from the Sanitary Landfill was 1.9% for Scenario 1 capping with an efficiency of 100% as compared to no-cap conditions. Likewise, the maximum change in flow out of the modeled region was 1.4% for Scenario 1 capping with an efficiency of 100% as compared to no-cap conditions. The vertical flow and transport model was used to transport a generic tracer from the Sanitary Landfill to the seepage line under no-cap conditions and Scenario 1 and Scenario 2 cap areas under cap efficiencies ranging from 100 to 85% as above. The cap proved it could effectively act to reduce saturated zone concentrations in the vicinity of the Sanitary Landfill in the long term. For Scenario 1 capping, the seepage line steady-state concentration decreased compared to no-cap steady-state concentrations from 100% to 48% for cap efficiencies of 100% to 85%.

Transport calculations in support of the ACL Demonstrations for the Sanitary Landfill were performed for 16 COCs. The vertical cross-sectional flow and transport model was used to perform transport calculations between the POC, the POE, and UTRC. The vertical cross-sectional model conservatively calculates down-gradient concentrations because it models maximum concentrations along the centerline of the plume assuming no lateral spreading. The majority of the COCs were organic constituents which are reported to undergo abiotic and abiotic degradation in groundwater environments. The attenuating processes of retardation and degradation were considered in the transport calculations. Retardation coefficients and degradation half lives were estimated based upon the literature and geochemical conditions present at the Sanitary Landfill. Two

scenarios for source loading were developed, a best-estimate and a maximum, both of which were conservative. For these two source scenarios, transport calculations were performed under no-cap conditions and Scenario 1 cap conditions assuming a 97% cap efficiency. Calculations were performed starting at the beginning of the Sanitary Landfill life (1974) through a thirty year assumed institutional control period ending at year 2025. The cap was assumed to be put in place in the middle of 1995.

The input concentration for the transport calculations was conservatively held constant at the maximum observed concentration of a particular COC at the POC wells from 1984 through the fourth quarter of 1991. Maximum concentrations simulated for the POE exceeded MCLs for three constituents for the no-cap best-estimate source scenario. These COCs were trichloroethylene, chloroethylene, and tritium. Both tritium and trichloroethylene were only slightly elevated over MCL. Maximum concentrations at the POE for Scenario 1 cap conditions for the conservative best-estimate source scenario were essentially the same as for no-cap conditions and the same three COCs exceeded MCLs. The steady-state concentrations simulated at the POE at the end of institutional control for the Scenario 1 cap conditions assuming the best-estimate source scenario were all below MCL with the exception of chloroethylene. Maximum concentrations simulated at the POE were controlled by the assumed source area and the stability of the constituent. For conservative constituents such as lead and arsenic, attenuation of concentrations between the Sanitary Landfill and the POE was relatively small because of the close proximity of the POE to the Sanitary Landfill (i.e., within 1 to 2 source dimensions) and the constant mass flux boundary condition. Concentrations simulated at the POC, POE and UTRC were related through the calculation of attenuation factors between the POC and the POE and the POC and UTRC.

The definition of the lateral extent of contamination and the potential discharge area and volumetric flux where Sanitary Landfill contaminated

groundwater discharges to the surface is required for the calculation of a groundwater to surface water dilution factor for the estimation of COC concentrations in UTRC. The area of contaminant discharge to the wetlands and the volumetric flux rate of contaminants to that area are also required by the risk assessors to define conditions for inhalation exposure risk scenarios. Because the vertical cross-sectional model does not consider lateral spreading, the Extended Pulse (Domenico and Robbins, 1985) closed-form advection-dispersion analytical model was used to determine an estimate of lateral extent of contamination. The lateral extent of contamination in the wetlands and UTRC was used in conjunction with the areal flow model to quantify a potential contaminated groundwater discharge rate to that area. The potentially contaminated discharge area in the wetlands was estimated to be $5.88 \times 10^6 \text{ ft}^2$ with a steady-state volumetric flux of $88,869 \text{ ft}^3/\text{day}$ (1.03 cfs). The area within UTRC which was estimated to be potentially within the contaminant plume is $2.6 \times 10^5 \text{ ft}^2$ with a steady-state volumetric flux of $45,871 \text{ ft}^3/\text{day}$ (0.53 cfs). The 7Q10 flow rate measured just upstream from the potential contaminant discharge area at UTRC at Station C is $8.4 \times 10^6 \text{ ft}^3/\text{day}$ (97.3 cfs). Assuming instantaneous dilution, a simple groundwater to surface water concentration reduction factor was calculated for UTRC and was equal to 62.

10.2 Data Needs in the Sanitary Landfill Vicinity

Several assumptions were made during the development of the Sanitary Landfill-local scale flow and transport models. Many of these assumptions were driven by the lack of characterization data available and the uncertainty of the data available. This is not to say that the characterization data presently available for the Sanitary Landfill is not good nor sufficient to study the Sanitary Landfill flow system. Obviously this report stands in contradiction to that premise. However, uncertainties in characterization data result in uncertainties in model conceptualization and results. Provided this fact, additional data is often desirable where hydrogeologic flow models are poorly constrained systems as witnessed by

the areal flow model sensitivity analysis (see Section 4.2.6). In the following paragraphs, data needs are identified which could help to characterize the Sanitary Landfill flow and transport system and ultimately result in improved and better constrained future flow and transport model results.

- Water level measurements to the north of the Sanitary Landfill would help considerable in prescribing boundary conditions on the northern up-gradient boundary of the Sanitary Landfill local-scale flow model. These water level measurements would greatly improve head control for the regional A/M Area flow model also. Water level measurements would also be helpful for areas to the east and west of the Sanitary Landfill. Head control in areas not considered wetlands is currently limited to the Sanitary Landfill and its very close proximity.
- The addition of a monitoring well network in areas down gradient of the Sanitary Landfill would greatly improve the understanding of the extent of contamination. Wells currently are in close proximity of the Sanitary Landfill. To fully characterize the extent of contamination there is a need for additional wells down gradient of the Sanitary Landfill within the wetlands monitoring both shallow and deeper portions of Aquifer Unit I/IIC.
- Continued sampling of the seepage line is recommended. In addition, more extensive surface water sampling stations between the present ones and UTRC are recommended in areas down gradient of the Sanitary Landfill.
- A geochemical evaluation of the groundwater below the Sanitary Landfill and along the potential flow paths as defined in this report would be very helpful in determining appropriate transport parameter values such as distribution coefficient and degradation rates.

Evaluation of dissolved organic carbon (DOC) and the groundwater redox state is crucial to an understanding of contaminant transport at the Sanitary Landfill. This analysis will also provide necessary information to describe the fate of metals.

- Currently a USGS UTRC gaging station (Station C) is located at the upstream model boundary of the Sanitary Landfill local-scale model. However, the next gage (Gage A) is located several miles downstream which makes estimation of the gain in UTRC from the Sanitary Landfill model region uncertain. The establishment of a UTRC gage at the downstream extent of UTRC in either the local-scale Sanitary Landfill model or the regional A/M model region would help constrain discharge estimates to UTRC and subsequently improve model reliability and estimates of the groundwater to surface water dilution factor.
- Aquifer tests would be very helpful in determining aquifer hydraulic conductivity over a scale more representative of the scale of flow evaluation. Slug tests provide an important information source but stress a small volume of the medium and are typically prone to boundary effects caused by the borehole. pumping tests and interference tests would provide some information on large scale heterogeneity, anisotropy, and storage parameters. It would also be highly useful to perform a pumping test designed to estimate the vertical conductivity of the Confining Unit I/IIB-I/IIC.
- A tracer test in the vicinity would provide in-situ estimates of effective porosity and dispersion parameters. An ambient tracer test or a borehole dilution test would provide direct estimates of groundwater seepage velocity. This level of information would be extremely beneficial for comparison to current calculated estimates of

groundwater velocity and transit time between the Sanitary Landfill and the wetlands or UTRC.

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APPENDIX A:

Maximum Concentration Distributions for SLF COCs

DESCRIPTION OF CONCENTRATION DATA PLOTS

In an effort to provide insight into the development of ground-water contamination at the SLF, the database was searched to find the date of the maximum recorded concentration for each COC for all wells. Bubble plots were one technique used to visualize this data. In addition, the maximum observed concentration for each COC as well as the quarter/year which it was recorded were posted at well locations to allow quick analysis. Appendix A contains bubble plots for all wells screened across the water table and maps of posted values for level B, C, and D wells.

The bubble plots show symbols only for wells in which a COC concentration above detection had been recorded in the database provided to INTERA. If no symbol exists at a particular well, then there was no recorded detection at that well. The size of the symbol at a particular well is proportional to the magnitude of the COC concentration at that well as compared to the magnitude of the COC concentration at all other wells. The minimum symbol size is 0.05 inches, which correlates to the smallest maximum COC concentration for all the wells screened at that interval. The maximum symbol size is 0.40 inches, which corresponds to the largest maximum COC concentration for all the wells screened at that interval.

The bubble plots are useful for quickly evaluating the historic maximum COC concentrations. In addition, they indicate areas in which COC have never been detected. The posted maximum values along with the associated dates provide insight into the temporal changes in maximum COC concentrations.

D-ZONE WELLS: 1,1,1-Trichloroethane

D-ZONE WELLS: 1,1,1-Trichloroethane

D-ZONE WELLS: Tetrachloroethylene

D-ZONE WELLS: Tetrachloroethylene

D-ZONE WELLS: Trichloroethylene

D-ZONE WELLS: trans-1,2-Dichloroethene

The map displays the SRS Plant site boundary and internal data points. The vertical axis represents SRS Plant Northing (FT) from 82500 to 86500, and the horizontal axis represents SRS Plant Easting (FT) from 44500 to 46500. The boundary is defined by a series of points with dates and values. Data points are scattered within the boundary, including dates like 1/91, 3/87, 3/88, 3/90, and values like 77.0, 320.0, 91.0, 754.0, 23.0, 647.0, 54.0, 104.0, and 100.0. A north arrow is shown in the upper right corner, indicating the orientation of the map.

D-ZONE WELLS: Chloroethylene (Vinyl chloride)

D-ZONE WELLS: Chloroethylene (Vinyl chloride)

D-ZONE WELLS: Dichlorodifluoromethane

D-ZONE WELLS: Dichlorodifluoromethane

D-ZONE WELLS: Trichlorofluoromethane

D-ZONE WELLS: Trichlorofluoromethane

D-ZONE WELLS: Dichloromethane

D-ZONE WELLS: Dichloromethane

D-ZONE WELLS: 1,4-Dichlorobenzene

D-ZONE WELLS: 1,4-Dichlorobenzene

D-ZONE WELLS: Chlorobenzene

The map displays the SRS Plant North and Easting coordinates. The vertical axis represents SRS Plant North (FT) from 82500 to 86500, and the horizontal axis represents SRS Plant Easting (FT) from 44500 to 46500. A large, irregular polygon outlines the plant boundary. Numerous points are plotted within and around this boundary, each labeled with a date and a numerical value. The dates include 1/82, 1/91, 4/81, and 3/91. The numerical values include <5.0, 52.0, 79.0, 8.1, 1.5, and <10.0. A compass rose in the upper right corner indicates 'Plant North' and 'True North'.

D-ZONE WELLS: Benzene

D-ZONE WELLS: Benzene

D-ZONE WELLS: Ethylbenzene

D-ZONE WELLS: Ethylbenzene

D-ZONE WELLS: Xylenes

[illegible]

D-ZONE WELLS: Arsenic

The map displays the SRS Plant layout with Northing and Easting coordinates. The Northing axis ranges from 82500 to 86500, and the Easting axis ranges from 44500 to 46500. A complex polygon represents the plant boundary. Inside and around this boundary, numerous points are plotted, each labeled with a date (e.g., 1/82, 2/91, 3/88) and a numerical value (e.g., <2.0, <5.0, 31.2, 37.6, 100.0). A compass rose in the upper right corner indicates 'Plant North' and 'True North'.

D-ZONE WELLS: Iron

D-ZONE WELLS: Iron

D-ZONE WELLS: Lead

D-ZONE WELLS: Lead

D-ZONE WELLS: Manganese

D-ZONE WELLS: Manganese

D-ZONE WELLS: Tritium

D-ZONE WELLS: Tritium

C-ZONE WELLS: 1,1,1-Trichloroethane

C-ZONE WELLS: Tetrachloroethylene

C-ZONE WELLS: Trichloroethylene

C-ZONE WELLS: trans-1,2-Dichloroethene

C-ZONE WELLS: Chloroethylene (Vinyl chloride)

C-ZONE WELLS: Dichlorodifluoromethane

C-ZONE WELLS: Trichlorofluoromethane

C-ZONE WELLS: Dichloromethane

C-ZONE WELLS: 1,4-Dichlorobenzene

C-ZONE WELLS: Chlorobenzene

C-ZONE WELLS: Benzene

C-ZONE WELLS: Ethylbenzene

C-ZONE WELLS: Xylenes

C-ZONE WELLS: Arsenic

C-ZONE WELLS: Iron

C-ZONE WELLS: Lead

C-ZONE WELLS: Manganese

C-ZONE WELLS: Tritium

B-ZONE WELLS: 1,1,1-Trichloroethane

B-ZONE WELLS: Tetrachloroethylene

B-ZONE WELLS: Trichloroethylene

B-ZONE WELLS: trans-1,2-Dichloroethene

B-ZONE WELLS: Chloroethylene (Vinyl chloride)

B-ZONE WELLS: Dichlorodifluoromethane

B-ZONE WELLS: Trichlorofluoromethane

B-ZONE WELLS: Dichloromethane

B-ZONE WELLS: 1,4-Dichlorobenzene

B-ZONE WELLS: Chlorobenzene

B-ZONE WELLS: Benzene

B-ZONE WELLS: Ethylbenzene

B-ZONE WELLS: Xylenes

B-ZONE WELLS: Arsenic

B-ZONE WELLS: Iron

B-ZONE WELLS: Lead

B-ZONE WELLS: Manganese

B-ZONE WELLS: Tritium

APPENDIX B:

MODFLOW Data Input for Steady-State Calibrated Areal Flow Model

BASIC PACKAGE

1 100 100 1 4
11 0 13 0 0 0 17 18 0 20 0 22 0 0 0 0 0 0

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[illegible]

B-6

[illegible]

B-8

0.17530E+03 0.17513E+03 0.17494E+03 0.17474E+03 0.17452E+03 0.17430E+03 0.17407E+03 0.17382E+03 0.17357E+03 0.17329E+03
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0.16809E+03 0.16722E+03 0.16631E+03 0.16538E+03 0.16445E+03 0.16353E+03 0.16261E+03 0.16168E+03 0.16074E+03 0.15977E+03
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0.12184E+03 0.12173E+03 0.12165E+03 0.12157E+03 0.12149E+03 0.12139E+03 0.12131E+03 0.12124E+03 0.12116E+03 0.12111E+03
0.12107E+03 0.12104E+03 0.12102E+03 0.12101E+03 0.12098E+03 0.12093E+03 0.12084E+03 0.12076E+03 0.12068E+03 0.12041E+03
0.12043E+03 0.12047E+03 0.12052E+03 0.12060E+03 0.12078E+03 0.99999E+02 0.99999E+02 0.99999E+02 0.99999E+02 0.99999E+02
0.13185E+03 0.13172E+03 0.13178E+03 0.13184E+03 0.13189E+03 0.13193E+03 0.13197E+03 0.13201E+03 0.13204E+03 0.13206E+03
0.13208E+03 0.13209E+03 0.13210E+03 0.13210E+03 0.13208E+03 0.13208E+03 0.13206E+03 0.13204E+03 0.13200E+03 0.13198E+03
0.13191E+03 0.13186E+03 0.13179E+03 0.13172E+03 0.13163E+03 0.13154E+03 0.13144E+03 0.13132E+03 0.13120E+03 0.13108E+03
0.13091E+03 0.13075E+03 0.13058E+03 0.13039E+03 0.13018E+03 0.12998E+03 0.12972E+03 0.12945E+03 0.12918E+03 0.12885E+03
0.12851E+03 0.12814E+03 0.12772E+03 0.12729E+03 0.12678E+03 0.12628E+03 0.12578E+03 0.12551E+03 0.12535E+03 0.12520E+03
0.12504E+03 0.12485E+03 0.12467E+03 0.12451E+03 0.12435E+03 0.12421E+03 0.12406E+03 0.12391E+03 0.12374E+03 0.12355E+03
0.12334E+03 0.12312E+03 0.12288E+03 0.12263E+03 0.12242E+03 0.12222E+03 0.12202E+03 0.12182E+03 0.12162E+03 0.12144E+03
0.12128E+03 0.12119E+03 0.12112E+03 0.12108E+03 0.12098E+03 0.12082E+03 0.12064E+03 0.12078E+03 0.12067E+03 0.12058E+03
0.12048E+03 0.12048E+03 0.12045E+03 0.12048E+03 0.12048E+03 0.12044E+03 0.12038E+03 0.12022E+03 0.99999E+02 0.99999E+02
0.99999E+02 0.99999E+02 0.99999E+02 0.99999E+02 0.99999E+02 0.99999E+02 0.99999E+02 0.99999E+02 0.99999E+02 0.99999E+02
0.13076E+03 0.13063E+03 0.13060E+03 0.13065E+03 0.13101E+03 0.13108E+03 0.13110E+03 0.13114E+03 0.13117E+03 0.13120E+03
0.13122E+03 0.13123E+03 0.13124E+03 0.13125E+03 0.13124E+03 0.13123E+03 0.13122E+03 0.13120E+03 0.13117E+03 0.13113E+03
0.13108E+03 0.13103E+03 0.13097E+03 0.13090E+03 0.13082E+03 0.13074E+03 0.13064E+03 0.13053E+03 0.13042E+03 0.13029E+03
0.13015E+03 0.13000E+03 0.12983E+03 0.12965E+03 0.12945E+03 0.12923E+03 0.12900E+03 0.12875E+03 0.12847E+03 0.12817E+03
0.12784E+03 0.12748E+03 0.12708E+03 0.12664E+03 0.12612E+03 0.12582E+03 0.12549E+03 0.12523E+03 0.12503E+03 0.12484E+03
0.12465E+03 0.12444E+03 0.12425E+03 0.12403E+03 0.12387E+03 0.12372E+03 0.12358E+03 0.12343E+03 0.12327E+03 0.12310E+03
0.12290E+03 0.12269E+03 0.12248E+03 0.12222E+03 0.12200E+03 0.12178E+03 0.12155E+03 0.12132E+03 0.12110E+03 0.12090E+03

B-22

BLOCK-CENTERED FLOW PACKAGE

B-26

B-27

B-28

B-32

B-33

B-34

B-35

B-37

B-38

B-39

B-41

GENERAL HEAD BOUNDARY PACKAGE

B-42

236	50			
236				
1	39	100	150.00	5.00
1	40	100	149.00	5.00
1	41	100	148.00	5.00
1	42	100	147.00	5.00
1	43	100	146.00	5.00
1	44	100	145.00	5.00
1	45	100	144.00	5.00
1	46	100	143.00	5.00
1	47	100	142.00	5.00
1	48	100	141.00	5.00
1	49	100	140.00	5.00
1	50	100	139.00	10.00
1	51	100	138.00	10.00
1	52	100	137.00	10.00
1	53	100	136.00	10.00
1	54	100	135.00	10.00
1	55	100	134.00	10.00
1	56	100	133.00	10.00
1	57	100	132.00	10.00
1	58	100	131.00	10.00
1	59	100	130.00	10.00
1	60	100	129.00	10.00
1	61	100	128.00	10.00
1	62	100	127.00	10.00
1	63	100	126.00	10.00
1	64	100	125.00	10.00
1	65	100	124.40	10.00
1	66	100	123.90	10.00
1	67	100	123.50	50.00
1	68	100	123.10	50.00
1	69	100	122.70	100.00
1	70	100	122.30	100.00
1	71	100	121.90	100.00
1	72	100	121.50	100.00
1	73	100	121.30	100.00
1	74	100	121.00	500.00
1	74	99	120.63	500.00
1	75	98	120.86	500.00
1	76	97	120.79	500.00
1	77	97	120.72	500.00
1	78	97	120.65	1000.00
1	79	97	120.57	1000.00
1	80	97	120.50	1000.00
1	80	96	120.43	1000.00
1	81	95	120.36	1000.00
1	81	94	120.29	5000.00
1	81	93	120.22	5000.00
1	81	92	120.15	5000.00
1	81	91	120.08	5000.00
1	81	90	120.01	5000.00
1	81	89	119.94	5000.00
1	82	88	119.86	5000.00
1	83	87	119.79	5000.00
1	83	86	119.72	5000.00
1	83	85	119.65	5000.00
1	83	84	119.58	5000.00
1	83	83	119.51	5000.00
1	83	82	119.44	5000.00
1	83	81	119.37	5000.00
1	84	80	119.30	5000.00
1	84	79	119.23	5000.00
1	85	78	119.16	5000.00
1	85	77	119.09	5000.00

1	85	76	119.02	5000.00
1	86	75	118.95	5000.00
1	86	74	118.88	5000.00
1	86	73	118.80	5000.00
1	85	72	118.73	5000.00
1	85	71	118.66	5000.00
1	86	70	118.59	5000.00
1	87	70	118.52	5000.00
1	86	69	118.45	5000.00
1	86	68	118.38	5000.00
1	86	67	118.31	5000.00
1	90	66	118.24	5000.00
1	90	65	118.17	5000.00
1	91	64	118.09	5000.00
1	92	63	118.02	5000.00
1	93	63	117.95	5000.00
1	94	63	117.88	5000.00
1	95	62	117.81	5000.00
1	95	61	117.74	5000.00
1	95	60	117.67	5000.00
1	96	59	117.60	5000.00
1	97	58	117.53	5000.00
1	98	57	117.46	5000.00
1	98	56	117.38	5000.00
1	98	55	117.31	5000.00
1	99	54	117.24	5000.00
1	99	53	117.17	5000.00
1	99	52	117.10	5000.00
1	99	51	117.03	5000.00
1	100	50	116.96	5000.00
1	100	49	116.89	5000.00
1	100	48	116.82	5000.00
1	100	47	116.75	5000.00
1	100	46	116.67	5000.00
1	100	45	116.60	5000.00
1	100	44	116.53	5000.00
1	100	43	116.46	5000.00
1	100	42	116.39	5000.00
1	100	41	116.32	5000.00
1	100	40	116.25	5000.00
1	100	39	116.18	5000.00
1	100	38	116.11	5000.00
1	100	37	116.04	5000.00
1	100	36	115.96	5000.00
1	100	35	115.89	5000.00
1	100	34	115.82	5000.00
1	100	33	115.75	5000.00
1	100	32	115.68	5000.00
1	100	31	115.50	5000.00
1	100	30	115.50	5000.00
1	100	29	115.50	5000.00
1	100	28	115.50	5000.00
1	100	27	115.50	5000.00
1	100	26	115.00	5000.00
1	100	25	115.00	5000.00
1	100	24	115.00	5000.00
1	100	23	115.00	5000.00
1	100	22	115.00	5000.00
1	100	21	115.00	5000.00
1	100	20	114.50	5000.00
1	100	19	114.50	5000.00
1	100	18	114.50	5000.00
1	100	17	114.50	5000.00
1	100	16	114.00	5000.00
1	100	15	114.00	5000.00

1	100	14	114.00	5000.00
1	100	13	114.00	5000.00
1	100	12	114.00	5000.00
1	100	11	114.00	5000.00
1	100	10	114.00	5000.00
1	100	9	114.00	5000.00
1	100	8	114.00	5000.00
1	100	7	113.50	5000.00
1	100	6	113.50	5000.00
1	100	5	113.50	5000.00
1	100	4	113.50	5000.00
1	100	3	113.50	5000.00
1	100	2	113.50	5000.00
1	100	1	113.50	5000.00
1	6	1	171.52	5000.00
1	7	1	171.44	5000.00
1	8	1	171.31	5000.00
1	9	1	171.15	5000.00
1	10	1	170.97	5000.00
1	11	1	170.78	5000.00
1	12	1	170.54	5000.00
1	13	1	170.30	5000.00
1	14	1	170.04	5000.00
1	15	1	169.77	5000.00
1	16	1	169.49	5000.00
1	17	1	169.20	5000.00
1	18	1	168.90	5000.00
1	19	1	168.58	5000.00
1	20	1	168.25	5000.00
1	21	1	167.92	5000.00
1	22	1	167.57	5000.00
1	23	1	167.21	5000.00
1	24	1	166.85	5000.00
1	25	1	166.47	5000.00
1	26	1	166.08	5000.00
1	27	1	165.69	5000.00
1	28	1	165.28	5000.00
1	29	1	164.87	5000.00
1	30	1	164.44	5000.00
1	31	1	164.01	5000.00
1	32	1	163.57	5000.00
1	33	1	163.12	5000.00
1	34	1	162.66	5000.00
1	35	1	162.20	5000.00
1	36	1	161.73	5000.00
1	37	1	161.25	5000.00
1	38	1	160.77	5000.00
1	39	1	160.28	5000.00
1	40	1	159.78	5000.00
1	41	1	159.28	5000.00
1	42	1	158.78	5000.00
1	43	1	158.24	5000.00
1	44	1	157.71	5000.00
1	45	1	157.17	5000.00
1	46	1	156.63	5000.00
1	47	1	156.07	5000.00
1	48	1	155.51	5000.00
1	49	1	154.94	5000.00
1	50	1	154.37	5000.00
1	51	1	153.78	5000.00
1	52	1	153.19	5000.00
1	53	1	152.59	5000.00
1	54	1	151.99	5000.00
1	55	1	151.38	5000.00
1	56	1	150.77	5000.00

1	57	1	150.15	5000.00
1	58	1	149.52	5000.00
1	59	1	148.88	5000.00
1	60	1	148.23	5000.00
1	61	1	147.58	5000.00
1	62	1	146.92	5000.00
1	63	1	146.24	5000.00
1	64	1	145.56	5000.00
1	65	1	144.87	5000.00
1	66	1	144.17	5000.00
1	67	1	143.47	5000.00
1	68	1	142.75	5000.00
1	69	1	142.02	5000.00
1	70	1	141.28	5000.00
1	71	1	140.53	5000.00
1	72	1	139.77	5000.00
1	73	1	139.00	5000.00
1	74	1	138.22	5000.00
1	75	1	137.43	5000.00
1	76	1	136.64	5000.00
1	77	1	135.83	5000.00
1	78	1	135.02	5000.00
1	79	1	134.19	5000.00
1	80	1	133.34	5000.00
1	81	1	132.49	5000.00
1	82	1	131.61	5000.00
1	83	1	130.72	5000.00
1	84	1	129.82	5000.00
1	85	1	128.90	5000.00
1	86	1	127.98	5000.00
1	87	1	127.05	5000.00
1	88	1	126.09	5000.00
1	89	1	125.11	5000.00
1	90	1	124.11	5000.00
1	91	1	123.09	5000.00
1	92	1	122.05	5000.00
1	93	1	120.98	5000.00
1	94	1	119.89	5000.00
1	95	1	118.78	5000.00
1	96	1	117.61	5000.00
1	97	1	116.45	5000.00
1	98	1	115.27	5000.00
1	99	1	114.06	5000.00

DRAIN PACKAGE

B-47

1363	50		
1363			
1	75	86	121.000 4.000e+02
1	75	87	121.000 4.000e+02
1	76	85	121.000 4.000e+02
1	76	86	121.000 4.000e+02
1	77	85	121.000 4.000e+02
1	77	86	121.000 4.000e+02
1	78	85	121.000 4.000e+02
1	78	86	121.000 4.000e+02
1	79	85	121.000 4.000e+02
1	79	86	121.000 4.000e+02
1	79	91	121.000 4.000e+02
1	79	92	121.000 4.000e+02
1	79	93	121.000 4.000e+02
1	79	94	121.000 4.000e+02
1	80	91	121.000 4.000e+02
1	80	92	121.000 4.000e+02
1	80	93	121.000 4.000e+02
1	80	94	121.000 4.000e+02
1	80	95	121.000 4.000e+02
1	80	87	121.000 4.000e+02
1	80	88	121.000 4.000e+02
1	81	87	121.000 4.000e+02
1	81	88	121.000 4.000e+02
1	80	89	121.000 4.000e+02
1	80	90	121.000 4.000e+02
1	82	81	121.000 4.000e+02
1	82	82	121.000 4.000e+02
1	82	83	121.000 4.000e+02
1	82	84	121.000 4.000e+02
1	82	85	121.000 4.000e+02
1	82	86	121.000 4.000e+02
1	82	87	121.000 4.000e+02
1	81	84	121.000 4.000e+02
1	81	85	121.000 4.000e+02
1	81	86	121.000 4.000e+02
1	80	81	122.000 4.000e+02
1	80	82	122.000 4.000e+02
1	80	83	122.000 4.000e+02
1	81	81	122.000 4.000e+02
1	81	82	122.000 4.000e+02
1	81	83	122.000 4.000e+02
1	79	84	122.000 4.000e+02
1	79	85	122.000 4.000e+02
1	79	86	122.000 4.000e+02
1	80	84	122.000 4.000e+02
1	80	85	122.000 4.000e+02
1	80	86	122.000 4.000e+02
1	78	87	122.000 4.000e+02
1	78	88	122.000 4.000e+02
1	78	89	122.000 4.000e+02
1	78	90	122.000 4.000e+02
1	79	87	122.000 4.000e+02
1	79	88	122.000 4.000e+02
1	79	89	122.000 4.000e+02
1	79	90	122.000 4.000e+02
1	77	91	122.000 4.000e+02
1	77	92	122.000 4.000e+02
1	77	93	122.000 4.000e+02
1	77	94	122.000 4.000e+02
1	78	91	122.000 4.000e+02
1	78	92	122.000 4.000e+02
1	78	93	122.000 4.000e+02
1	78	94	122.000 4.000e+02

1	75	94	122.000 4.000e+02
1	76	94	122.000 4.000e+02
1	75	95	122.000 4.000e+02
1	75	91	123.000 4.000e+02
1	75	92	123.000 4.000e+02
1	75	93	123.000 4.000e+02
1	76	91	123.000 4.000e+02
1	76	92	123.000 4.000e+02
1	76	93	123.000 4.000e+02
1	76	87	123.000 4.000e+02
1	76	88	123.000 4.000e+02
1	76	89	123.000 4.000e+02
1	76	90	123.000 4.000e+02
1	77	87	123.000 4.000e+02
1	77	88	123.000 4.000e+02
1	77	89	123.000 4.000e+02
1	77	90	123.000 4.000e+02
1	77	84	123.000 4.000e+02
1	77	85	123.000 4.000e+02
1	77	86	123.000 4.000e+02
1	78	84	123.000 4.000e+02
1	78	85	123.000 4.000e+02
1	78	86	123.000 4.000e+02
1	78	81	123.000 4.000e+02
1	78	82	123.000 4.000e+02
1	78	83	123.000 4.000e+02
1	79	81	123.000 4.000e+02
1	79	82	123.000 4.000e+02
1	79	83	123.000 4.000e+02
1	76	81	124.000 4.000e+02
1	76	82	124.000 4.000e+02
1	76	83	124.000 4.000e+02
1	77	81	124.000 4.000e+02
1	77	82	124.000 4.000e+02
1	77	83	124.000 4.000e+02
1	75	84	124.000 4.000e+02
1	75	85	124.000 4.000e+02
1	75	86	124.000 4.000e+02
1	76	84	124.000 4.000e+02
1	76	85	124.000 4.000e+02
1	76	86	124.000 4.000e+02
1	75	87	124.000 4.000e+02
1	75	88	124.000 4.000e+02
1	75	89	124.000 4.000e+02
1	75	90	124.000 4.000e+02
1	83	77	121.000 4.000e+02
1	83	78	121.000 4.000e+02
1	84	77	121.000 4.000e+02
1	84	78	121.000 4.000e+02
1	82	79	121.000 4.000e+02
1	82	80	121.000 4.000e+02
1	83	79	121.000 4.000e+02
1	83	80	121.000 4.000e+02
1	84	73	121.000 4.000e+02
1	84	74	121.000 4.000e+02
1	84	75	121.000 4.000e+02
1	85	73	121.000 4.000e+02
1	85	74	121.000 4.000e+02
1	85	75	121.000 4.000e+02
1	83	76	121.000 4.000e+02
1	84	76	121.000 4.000e+02
1	84	70	121.000 4.000e+02
1	84	71	121.000 4.000e+02
1	84	72	121.000 4.000e+02
1	85	69	120.000 4.000e+02

1	85	70	120.000 4.000e+02
1	86	68	120.000 4.000e+02
1	86	69	120.000 4.000e+02
1	87	68	120.000 4.000e+02
1	87	69	120.000 4.000e+02
1	87	68	120.000 4.000e+02
1	87	67	120.000 4.000e+02
1	88	66	120.000 4.000e+02
1	88	67	120.000 4.000e+02
1	88	65	120.000 4.000e+02
1	89	64	120.000 4.000e+02
1	89	65	120.000 4.000e+02
1	89	66	120.000 4.000e+02
1	90	63	120.000 4.000e+02
1	90	64	120.000 4.000e+02
1	91	62	120.000 4.000e+02
1	91	63	120.000 4.000e+02
1	81	76	121.000 4.000e+02
1	81	77	121.000 4.000e+02
1	81	78	121.000 4.000e+02
1	82	76	121.000 4.000e+02
1	82	77	121.000 4.000e+02
1	82	78	121.000 4.000e+02
1	80	79	121.000 4.000e+02
1	80	80	121.000 4.000e+02
1	81	79	121.000 4.000e+02
1	81	80	121.000 4.000e+02
1	82	70	121.000 4.000e+02
1	82	71	121.000 4.000e+02
1	82	72	121.000 4.000e+02
1	82	73	121.000 4.000e+02
1	82	74	121.000 4.000e+02
1	82	75	121.000 4.000e+02
1	83	70	121.000 4.000e+02
1	83	71	121.000 4.000e+02
1	83	72	121.000 4.000e+02
1	83	73	121.000 4.000e+02
1	83	74	121.000 4.000e+02
1	83	75	121.000 4.000e+02
1	83	68	121.000 4.000e+02
1	83	69	121.000 4.000e+02
1	84	68	121.000 4.000e+02
1	84	69	121.000 4.000e+02
1	85	68	121.000 4.000e+02
1	85	67	121.000 4.000e+02
1	85	68	121.000 4.000e+02
1	86	65	121.000 4.000e+02
1	86	66	121.000 4.000e+02
1	86	67	121.000 4.000e+02
1	87	63	121.000 4.000e+02
1	87	64	121.000 4.000e+02
1	87	65	121.000 4.000e+02
1	88	62	121.000 4.000e+02
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1	90	62	121.000 4.000e+02
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1	83	53	124.000	4.000e+02
1	84	52	124.000	4.000e+02
1	84	53	124.000	4.000e+02
1	85	52	124.000	4.000e+02
1	85	53	124.000	4.000e+02
1	80	55	125.000	4.000e+02
1	80	56	125.000	4.000e+02
1	80	57	125.000	4.000e+02
1	80	58	125.000	4.000e+02
1	80	59	125.000	4.000e+02
1	81	55	125.000	4.000e+02
1	81	56	125.000	4.000e+02
1	81	57	125.000	4.000e+02
1	81	58	125.000	4.000e+02
1	81	59	125.000	4.000e+02
1	78	55	126.000	4.000e+02
1	78	56	126.000	4.000e+02
1	78	57	126.000	4.000e+02
1	78	58	126.000	4.000e+02
1	78	59	126.000	4.000e+02
1	79	55	126.000	4.000e+02
1	79	56	126.000	4.000e+02
1	79	57	126.000	4.000e+02
1	79	58	126.000	4.000e+02
1	79	59	126.000	4.000e+02
1	81	50	125.000	4.000e+02
1	81	51	125.000	4.000e+02
1	81	52	125.000	4.000e+02
1	81	53	125.000	4.000e+02
1	81	54	125.000	4.000e+02
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1	82	51	125.000	4.000e+02
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1	82	49	126.000	4.000e+02
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1	84	49	126.000	4.000e+02
1	85	49	126.000	4.000e+02
1	99	33	117.000	4.000e+02
1	99	34	117.000	4.000e+02
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1	99	36	117.000	4.000e+02
1	99	37	117.000	4.000e+02
1	99	38	117.000	4.000e+02
1	99	39	117.000	4.000e+02
1	99	40	117.000	4.000e+02
1	99	28	116.000	4.000e+02
1	99	29	116.000	4.000e+02
1	99	30	116.000	4.000e+02

1	98	31	118.000 4.000e+02
1	98	32	118.000 4.000e+02
1	98	34	118.000 4.000e+02
1	98	35	118.000 4.000e+02
1	98	36	118.000 4.000e+02
1	98	37	118.000 4.000e+02
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1	98	27	118.000 4.000e+02
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1	98	1	118.000 4.000e+02
1	98	2	118.000 4.000e+02
1	98	3	118.000 4.000e+02
1	98	4	118.000 4.000e+02

1	96	5	118.000 4.000e+02
1	96	6	118.000 4.000e+02
1	96	7	118.000 4.000e+02
1	96	8	118.000 4.000e+02
1	74	84	124.000 4.000e+02
1	74	82	124.000 4.000e+02
1	74	83	124.000 4.000e+02
1	74	84	124.000 4.000e+02
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1	74	87	124.000 4.000e+02
1	74	88	124.000 4.000e+02
1	74	89	124.000 4.000e+02
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1	74	93	124.000 4.000e+02
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1	96	42	121.000 4.000e+02
1	93	42	121.000 4.000e+02
1	93	43	121.000 4.000e+02
1	93	44	121.000 4.000e+02
1	93	45	121.000 4.000e+02
1	93	46	121.000 4.000e+02
1	93	47	121.000 4.000e+02
1	91	43	122.000 4.000e+02

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1	91	45	122,000	4.000e+02
1	91	46	122,000	4.000e+02
1	91	47	122,000	4.000e+02
1	92	43	122,000	4.000e+02
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1	92	45	122,000	4.000e+02
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1	76	56	127,000	4.000e+02
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1	72	61	127,000	4.000e+02
1	73	58	127,000	4.000e+02
1	73	59	127,000	4.000e+02
1	73	60	127,000	4.000e+02
1	73	61	127,000	4.000e+02
1	74	57	127,000	4.000e+02
1	74	58	127,000	4.000e+02

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1	74	60	127.000 4.000e+02
1	69	59	126.000 2.000e+02
1	69	60	126.000 2.000e+02
1	69	61	126.000 2.000e+02
1	69	62	126.000 2.000e+02
1	69	63	126.000 2.000e+02
1	69	64	126.000 2.000e+02
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1	70	60	126.000 2.000e+02
1	70	61	126.000 2.000e+02
1	70	62	126.000 2.000e+02
1	70	63	126.000 2.000e+02
1	70	64	126.000 2.000e+02
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1	71	60	126.000 2.000e+02
1	71	61	126.000 2.000e+02
1	71	62	126.000 2.000e+02
1	71	63	126.000 2.000e+02
1	71	64	126.000 2.000e+02
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1	66	61	126.000 2.000e+02
1	66	62	126.000 2.000e+02
1	66	63	126.000 2.000e+02
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1	67	61	126.000 2.000e+02
1	67	62	126.000 2.000e+02
1	67	63	126.000 2.000e+02
1	67	64	126.000 2.000e+02
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1	68	61	126.000 2.000e+02
1	68	62	126.000 2.000e+02
1	68	63	126.000 2.000e+02
1	68	64	126.000 2.000e+02
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1	67	66	126.000 2.000e+02
1	67	67	126.000 2.000e+02
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1	68	66	126.000 2.000e+02
1	68	67	126.000 2.000e+02
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1	69	66	126.000 2.000e+02
1	69	67	126.000 2.000e+02
1	67	68	127.000 2.000e+02
1	67	69	127.000 2.000e+02
1	68	68	127.000 2.000e+02
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1	68	70	127.000 2.000e+02
1	69	68	127.000 2.000e+02
1	69	69	127.000 2.000e+02
1	69	70	127.000 2.000e+02
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1	75	82	124.000 2.000e+02
1	75	83	124.000 2.000e+02
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1	70	72	127.000 2.000e+02
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1	74	79	124.000 2.000e+02
1	74	80	124.000 2.000e+02
1	74	81	124.000 2.000e+02
1	66	59	126.000 2.000e+02
1	67	59	126.000 2.000e+02
1	68	59	126.000 2.000e+02
1	65	61	126.000 2.000e+02
1	65	62	126.000 2.000e+02

1	65	63	128.000 2.000e+02
1	65	64	128.000 2.000e+02
1	65	65	128.000 2.000e+02
1	66	64	128.000 2.000e+02
1	66	65	128.000 2.000e+02
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1	64	61	128.000 2.000e+02
1	64	62	128.000 2.000e+02
1	64	63	128.000 2.000e+02
1	64	64	128.000 2.000e+02
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1	63	60	130.000 2.000e+02
1	63	61	130.000 2.000e+02
1	63	62	130.000 2.000e+02
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1	23	100	157.000 5.000e+03
1	15	100	158.000 5.000e+03
1	16	100	158.000 5.000e+03
1	17	100	158.000 5.000e+03
1	18	100	158.000 5.000e+03

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1	5	100	159,000	5,000e+03
1	6	100	159,000	5,000e+03
1	7	100	159,000	5,000e+03
1	8	100	159,000	5,000e+03
1	9	100	159,000	5,000e+03
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1	59	82	138,000	3,000e+02
1	59	83	138,000	3,000e+02
1	60	81	138,000	3,000e+02
1	60	82	138,000	3,000e+02
1	60	83	138,000	3,000e+02
1	61	82	134,000	3,000e+02
1	61	83	134,000	3,000e+02
1	61	84	134,000	3,000e+02
1	62	82	134,000	3,000e+02
1	62	83	134,000	3,000e+02
1	62	84	134,000	3,000e+02
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1	66	87	128,000	3,000e+02
1	66	88	128,000	3,000e+02
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1	65	87	129,000	3,000e+02
1	65	88	129,000	3,000e+02
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1	64	86	135,000	3,000e+02
1	64	87	135,000	3,000e+02
1	64	88	133,000	3,000e+02
1	64	89	133,000	3,000e+02
1	64	70	133,000	3,000e+02
1	64	71	133,000	3,000e+02
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1	73	78	125,000	8,000e+02
1	73	79	125,000	8,000e+02
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1	73	82	125,000	8,000e+02
1	73	83	125,000	8,000e+02
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1	73	85	125,000	8,000e+02
1	73	86	125,000	8,000e+02
1	73	87	125,000	8,000e+02
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1	73	91	125,000	8,000e+02
1	73	92	125,000	8,000e+02
1	73	93	125,000	8,000e+02
1	73	94	125,000	8,000e+02
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1	72	77	128,000	8,000e+02
1	72	78	128,000	8,000e+02
1	72	79	128,000	8,000e+02
1	72	80	128,000	8,000e+02

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1	72	82	128.000 8.000e+02
1	72	83	128.000 8.000e+02
1	72	84	128.000 8.000e+02
1	72	85	128.000 8.000e+02
1	72	86	128.000 8.000e+02
1	72	87	128.000 8.000e+02
1	72	88	128.000 8.000e+02
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1	72	91	128.000 8.000e+02
1	72	92	128.000 8.000e+02
1	72	93	128.000 8.000e+02
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1	69	72	127.000 8.000e+02
1	69	73	127.000 8.000e+02
1	69	74	127.000 8.000e+02
1	69	75	127.000 8.000e+02
1	69	76	127.000 8.000e+02
1	69	77	127.000 8.000e+02
1	70	73	127.000 8.000e+02
1	70	74	127.000 8.000e+02
1	70	75	127.000 8.000e+02
1	70	76	127.000 8.000e+02
1	70	77	127.000 8.000e+02
1	71	74	127.000 8.000e+02
1	71	75	127.000 8.000e+02
1	71	76	127.000 8.000e+02
1	71	77	127.000 8.000e+02
1	68	71	128.000 8.000e+02
1	68	72	128.000 8.000e+02
1	68	73	128.000 8.000e+02
1	68	74	128.000 8.000e+02
1	68	75	128.000 8.000e+02
1	68	76	128.000 8.000e+02
1	68	69	128.000 8.000e+02
1	68	70	128.000 8.000e+02
1	68	71	128.000 8.000e+02
1	67	70	128.000 8.000e+02
1	67	71	128.000 8.000e+02
1	65	69	130.000 8.000e+02
1	65	70	130.000 8.000e+02
1	65	71	130.000 8.000e+02
1	67	72	129.000 8.000e+02
1	67	73	129.000 8.000e+02
1	67	74	129.000 8.000e+02
1	67	75	129.000 8.000e+02
1	67	76	129.000 8.000e+02
1	67	77	129.000 8.000e+02
1	68	77	129.000 8.000e+02
1	66	72	130.000 8.000e+02
1	66	73	130.000 8.000e+02
1	66	74	130.000 8.000e+02
1	66	75	130.000 8.000e+02
1	66	76	130.000 8.000e+02
1	66	77	130.000 8.000e+02
1	66	78	130.000 8.000e+02
1	67	78	130.000 8.000e+02
1	68	78	130.000 8.000e+02
1	69	78	130.000 8.000e+02
1	71	78	127.000 8.000e+02
1	71	79	127.000 8.000e+02
1	71	80	127.000 8.000e+02
1	71	81	127.000 8.000e+02
1	71	82	127.000 8.000e+02

1	71	83	127.000	8.000e+02
1	71	84	127.000	8.000e+02
1	71	85	127.000	8.000e+02
1	71	86	127.000	8.000e+02
1	70	87	127.000	8.000e+02
1	70	88	127.000	8.000e+02
1	70	89	127.000	8.000e+02
1	70	90	127.000	8.000e+02
1	70	91	127.000	8.000e+02
1	70	92	127.000	8.000e+02
1	70	93	127.000	8.000e+02
1	71	87	127.000	8.000e+02
1	71	88	127.000	8.000e+02
1	71	89	127.000	8.000e+02
1	71	90	127.000	8.000e+02
1	71	91	127.000	8.000e+02
1	71	92	127.000	8.000e+02
1	71	93	127.000	8.000e+02
1	70	78	128.000	8.000e+02
1	70	79	128.000	8.000e+02
1	70	80	128.000	8.000e+02
1	70	81	128.000	8.000e+02
1	70	82	128.000	8.000e+02
1	70	83	128.000	8.000e+02
1	70	84	128.000	8.000e+02
1	70	85	128.000	8.000e+02
1	70	86	128.000	8.000e+02
1	64	72	132.000	8.000e+02
1	64	73	132.000	8.000e+02
1	64	74	132.000	8.000e+02
1	64	75	132.000	8.000e+02
1	64	76	132.000	8.000e+02
1	64	77	132.000	8.000e+02
1	64	78	132.000	8.000e+02
1	65	72	132.000	8.000e+02
1	65	73	132.000	8.000e+02
1	65	74	132.000	8.000e+02
1	65	75	132.000	8.000e+02
1	65	76	132.000	8.000e+02
1	65	77	132.000	8.000e+02
1	65	78	132.000	8.000e+02
1	63	73	133.000	8.000e+02
1	63	74	133.000	8.000e+02
1	63	75	133.000	8.000e+02
1	63	76	133.000	8.000e+02
1	63	77	133.000	8.000e+02
1	63	78	133.000	8.000e+02
1	62	74	134.000	8.000e+02
1	62	75	134.000	8.000e+02
1	62	76	134.000	8.000e+02
1	62	77	134.000	8.000e+02
1	62	78	134.000	8.000e+02
1	61	75	135.000	8.000e+02
1	61	76	135.000	8.000e+02
1	61	77	135.000	8.000e+02
1	61	78	135.000	8.000e+02
1	60	77	136.000	8.000e+02
1	60	78	136.000	8.000e+02
1	60	79	136.000	8.000e+02
1	60	80	136.000	8.000e+02
1	60	81	136.000	8.000e+02
1	60	82	136.000	8.000e+02
1	61	79	135.000	8.000e+02
1	61	80	135.000	8.000e+02
1	61	81	135.000	8.000e+02

1	81	82	135.000 8.000e+02
1	81	83	135.000 8.000e+02
1	81	84	135.000 8.000e+02
1	82	79	135.000 8.000e+02
1	82	80	135.000 8.000e+02
1	82	81	135.000 8.000e+02
1	82	82	135.000 8.000e+02
1	82	83	135.000 8.000e+02
1	82	84	135.000 8.000e+02
1	88	79	130.000 8.000e+02
1	88	80	130.000 8.000e+02
1	88	81	130.000 8.000e+02
1	88	82	130.000 8.000e+02
1	88	83	130.000 8.000e+02
1	88	84	130.000 8.000e+02
1	88	85	130.000 8.000e+02
1	89	79	130.000 8.000e+02
1	89	80	130.000 8.000e+02
1	89	81	130.000 8.000e+02
1	89	79	135.000 8.000e+02
1	89	80	135.000 8.000e+02
1	89	81	135.000 8.000e+02
1	89	82	135.000 8.000e+02
1	89	83	135.000 8.000e+02
1	89	84	135.000 8.000e+02
1	89	85	135.000 8.000e+02
1	87	79	135.000 8.000e+02
1	87	80	135.000 8.000e+02
1	87	81	135.000 8.000e+02
1	87	82	135.000 8.000e+02
1	87	83	135.000 8.000e+02
1	87	84	135.000 8.000e+02
1	87	85	135.000 8.000e+02
1	85	79	132.000 8.000e+02
1	85	80	132.000 8.000e+02
1	85	81	132.000 8.000e+02
1	85	82	132.000 8.000e+02
1	85	83	132.000 8.000e+02
1	85	84	132.000 8.000e+02
1	85	85	132.000 8.000e+02
1	84	79	133.000 8.000e+02
1	84	80	133.000 8.000e+02
1	84	81	133.000 8.000e+02
1	84	82	133.000 8.000e+02
1	84	83	133.000 8.000e+02
1	84	84	133.000 8.000e+02
1	84	85	133.000 8.000e+02
1	83	79	134.000 8.000e+02
1	83	80	134.000 8.000e+02
1	83	81	134.000 8.000e+02
1	83	82	134.000 8.000e+02
1	83	83	134.000 8.000e+02
1	83	84	134.000 8.000e+02
1	83	85	134.000 8.000e+02
1	89	82	129.000 8.000e+02
1	89	83	129.000 8.000e+02
1	89	84	129.000 8.000e+02
1	89	85	129.000 8.000e+02
1	89	86	129.000 8.000e+02
1	89	87	129.000 8.000e+02
1	89	88	129.000 8.000e+02
1	89	89	129.000 8.000e+02
1	89	90	129.000 8.000e+02
1	88	86	130.000 8.000e+02
1	88	87	130.000 8.000e+02

1	86	88	130.000 8.000e+02
1	86	89	130.000 8.000e+02
1	86	86	135.000 8.000e+02
1	86	87	135.000 8.000e+02
1	87	86	135.000 8.000e+02
1	87	87	135.000 8.000e+02
1	83	85	134.000 8.000e+02
1	83	86	134.000 8.000e+02
1	57	59	140.000 8.000e+02
1	57	60	140.000 8.000e+02
1	57	61	140.000 0.000e+00
1	79	47	125.000 9.990e+02
1	79	48	125.000 9.990e+02
1	79	49	125.000 9.990e+02
1	80	47	125.000 9.990e+02
1	80	48	125.000 9.990e+02
1	80	49	125.000 9.990e+02
1	81	47	125.000 9.990e+02
1	81	48	125.000 9.990e+02
1	81	49	125.000 9.990e+02
1	82	47	125.000 9.990e+02
1	82	48	125.000 9.990e+02
1	81	46	126.000 9.990e+02
1	82	46	126.000 9.990e+02
1	83	46	126.000 9.990e+02
1	83	47	126.000 9.990e+02
1	84	46	126.000 9.990e+02
1	84	47	126.000 9.990e+02
1	85	46	126.000 9.990e+02
1	85	47	126.000 9.990e+02
1	83	45	125.000 9.990e+02
1	84	45	125.000 9.990e+02
1	85	45	125.000 9.990e+02
1	87	44	124.000 9.990e+02
1	87	45	124.000 9.990e+02
1	88	44	124.000 9.990e+02
1	88	45	124.000 9.990e+02
1	89	44	123.000 9.990e+02
1	89	45	123.000 9.990e+02
1	90	44	123.000 9.990e+02
1	90	45	123.000 9.990e+02
1	95	38	120.000 9.990e+02
1	95	39	120.000 9.990e+02
1	95	40	120.000 9.990e+02
1	96	38	120.000 9.990e+02
1	96	39	120.000 9.990e+02
1	96	40	120.000 9.990e+02
1	96	28	120.000 9.990e+02
1	96	29	120.000 9.990e+02
1	96	30	120.000 9.990e+02
1	96	31	120.000 9.990e+02
1	96	32	120.000 9.990e+02
1	96	33	120.000 9.990e+02
1	96	34	120.000 9.990e+02
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1	96	36	120.000 9.990e+02
1	96	37	120.000 9.990e+02
1	97	20	120.000 9.990e+02
1	97	21	120.000 9.990e+02
1	97	22	120.000 9.990e+02
1	97	23	120.000 9.990e+02
1	97	24	120.000 9.990e+02
1	97	25	120.000 9.990e+02
1	97	26	120.000 9.990e+02
1	97	27	120.000 9.990e+02

RECHARGE PACKAGE

1	0
1	0

03.42000-03(10E12.4) -1

OUTPUT CONTROL PACKAGE

-1	0	80	0
0	1	1	1
0	0	0	0

PCG SOLVER PACKAGE

100	100	1
0.01	0.1	1.00

2 1

END

DATE
FILMED

12 / 8 / 93