Appendix 2.4B—Groundwater Model Development & Analysis

Prepared by
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June 2008

**Southern Nuclear Operating Company
Vogtle Early Site Permit Application**

GROUNDWATER MODEL
DEVELOPMENT & ANALYSIS

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Abbreviations

ft	feet
ft/day	feet per day
ft/ft	feet per foot
ft^2/day	square feet per day
in/yr	inches per year
bgs	below ground surface
msl	mean sea level

Acronyms

AMG	Alebraic Multigrid
BBM	Blue Bluff Marl
ESP	Early Site Permit
GMG	Geometric Multigrid
NRC	Nuclear Regulatory Commission
PA	Protected Area
PCG	Preconditioned Conjugate-Gradient
SER	Safety Evaluation Report
SIP	Strongly Implicit Procedure
SOR	Slice-Successive Overrelaxation
SSAR	Site Safety Analysis Report
FSAR	Final Safety Analysis Report
USGS	United States Geological Survey
VEGP	Vogtle Electrical Generating Plant
WHS	Waterloo Hydrogeologic Services

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EXECUTIVE SUMMARY

A groundwater flow model of the VEGP site has been developed in support of Early Site Application for Units 3 & 4. The model is two dimensional in the horizontal plane, using a single layer to describe the Water Table aquifer. The model was developed using the code MODFLOW 2000 developed by the U.S. Geological Survey, as it is implemented in the user interface environment Visual MODFLOW developed by Schlumberger Water Services.

The computer files of an initial version of this model were submitted to the NRC in January 2008. Upon review of these files the NRC provided written comments on the model, which were further discussed in meetings between the NRC and SNC and Bechtel on April 8-9, 2008.

NRC's comments concerned the accurate representation of the top of Blue Bluff Marl (BBM), which may control the location of the groundwater divide near the site of Units 3 & 4; spatial trends in residual errors; model mass balance errors; and the inadequate documentation of the model. The present report documents the groundwater model and discusses new, not previously presented work, which addresses NRC's comments.

The top of the BBM in the model was defined based on reinterpretation of all available data, including historic data and data collected during the site investigations for the ESP/COL investigations. The mapped BBM outcrop along the bluff near the Savannah River and to the south of the VEGP site was included in the model. In addition, a walk down over the area of the groundwater model was conducted to identify areas of groundwater discharge near the outcrop of the BBM.

Spatial trends in residual errors that existed in early simulations with the model were eliminated. Spatially variable groundwater recharge based on the vegetation cover, soil conditions and ground surface slope was introduced in the model. Several plausible hydraulic conductivity zonations and values were tested.

In all simulations the mass balance discrepancy between inflow and outflow was reduced to less than one percent. This was achieved through the use of smaller head convergence criteria and by adjusting other parameters of the numerical solver used by the model.

The present report documents the calibration process in detail describing the steps towards improved agreement between computed and observed water levels and the rationale for all assumptions made along the way.

The calibrated model was used to simulate post-construction conditions, accounting for changes in the topography at the site of Units 3 & 4, the presence of backfill material in the area of the new structures, and changes in ground water recharge. Particle tracking was performed to identify the groundwater pathways of potential liquid effluent releases from the power block area.

The conclusion drawn from all simulations with several alternative plausible models was that all groundwater pathways from the power block lead to Mallard Pond north of Units 3 & 4. Under all conditions the groundwater divide remains south of the power block area.

1.   OBJECTIVE AND SCOPE

The objective of this report is to document the development, calibration and use of a groundwater flow model for the Water Table aquifer at the Vogtle site. The model was developed in support of the responses to Open Items 2.4-2, 2.4-3, and 2.4-4 as identified in the NRC Safety Evaluation Report (SER) with Open Items for the Vogtle ESP application. These Open Items are listed in Attachment 1.

The model was developed in November 2007 and documented in Bechtel calculation 25144-K-003, Rev 0. A two-dimensional single layer groundwater model was used to simulate groundwater flow in the Water Table aquifer. A brief description of the model was presented in the SSAR Section 2.4.12 of the Vogtle ESP application, Revision 3, November 2007. At NRC's request the electronic files for the three simulated cases discussed in SSAR Section 2.4.12 were submitted to the NRC. These were the files included in the electronic Attachment 4 to Bechtel calculation 25144-K-003, Rev 0. However, Bechtel calculation 25144-K-003, Rev 0 itself was not reviewed by the NRC.

Upon review of the electronic computer files for the groundwater flow model, the NRC provided specific comments on the model, which are reproduced in Attachment 2. These comments were discussed in a meeting between the NRC and SNC and Bechtel at NRC's headquarters in Rockville on April 8, 2008 and in a second meeting in Bechtel's Frederick, Maryland office on April 9, 2008.

In the April 9 meeting the NRC stressed that the analysis with the model should include the plausible alternative conceptual models that do not violate the data and emphasized the need for thorough documentation of the process followed to arrive at the calibrated model used to predict future groundwater conditions and pathways of potential effluent releases. The present report was prepared in response to these comments.

2.   AQUIFIER DESCRIPTION & AVAILABLE DATA

2.1   Site Overview

The 3,169 acre VEGP site is located on a bluff on the southwest side of the Savannah River in eastern Burke County, Georgia, within the Coastal Plain physiographic province (see Figure 1). The finished grade level elevation of the proposed AP1000 VEGP Units 3 & 4 will approximately 220 ft msl.   The bottom of the foundation slab for the safety-related AP1000 containment structure will be at 39.5 ft below grade level, i.e. at elevation 180.5 ft msl. In constructing the new units, the site will be excavated approximately 80 ft to 90 ft below existing grade to remove the in-situ soil down to the principal bearing strata, the Blue Bluff Marl (BBM). The in-situ soil will be replaced with Category 1 and 2 structural fill material. Foundations for the new units will be poured on this new fill material and the fill material will be placed around the structures and continue up to the finished grade elevation of 220 ft msl.

Figure 2 shows the site and the location of the existing Units 1 & 2, as well as the planned Units 3 & 4, along with the observation wells monitored during the ESP groundwater investigation.

2.2   Hydrostratigraphy

The VEGP site and associated groundwater is located within the Coastal Plain physiographic province. There are three underlying aquifers at the VEGP site, the Cretaceous, the Tertiary, and the Water table (or Upper Three Runs) aquifer, all being part of the Southeastern Coastal Plain aquifer system.

- Water Table Aquifer: This is the unconfined aquifer that forms the first water-bearing zone encountered beneath the VEGP site. It consists of the Barnwell Group and includes the discontinuous deposits of the Utley limestone. The saturated interval within the Barnwell Group is commonly referred to as the Water Table aquifer which is also known as the Upper Three Runs aquifer.

- Tertiary Aquifer: The most productive aquifer at the VEGP site consists of the Congaree and Still Branch Formations, which are hydraulically connected and are referred to as the Tertiary aquifer. The overlying Lisbon Formation, containing the Blue Bluff Marl, acts as a confining layer. Recharge to the Tertiary aquifer is primarily by infiltration of rainfall in its outcrop area, which is a belt 20 to 60 miles wide extending northeastward across central Georgia and into portions of Alabama to the west and South Carolina to the east. Discharge from the Tertiary aquifer occurs from pumping, from natural springs in areas where topography is lower than the piezometric level of the aquifer, and from subaqueous outcrops that are presumed to occur offshore. Discharge also occurs to the Savannah River where the river has completely eroded the Blue Bluff Marl confining layer allowing discharge from the aquifer to the river bed.

● <u>Cretaceous Aquifer:</u> The Cretaceous aquifer is the lowermost aquifer at the VEGP site that overlies the impermeable bedrock. This aquifer comprises the Cape Fear Formation, Pio-Nono Formation/unnamed sands, Gaillard Formation/Black Creek Formation, and Steel Creek Formation. The Cretaceous aquifer system is under confined conditions for most of its areal extent. Recharge to the Cretaceous aquifer system is primarily by direct infiltration of rainfall in its outcrop area, located north of the VEGP site in a 10- to 30-mile-wide belt extending from Augusta, Georgia, northeastward across South Carolina to near the state line separating North and South Carolina. Discharge of the Cretaceous aquifer system is primarily from subaqueous exposures of the aquifer that are presumed to occur along the Continental Shelf. Other discharge sources are to the Savannah River and by pumping.

The ground surface contour map at the VEGP site is shown on Figure 2 (Ref. 12).

2.3   The Water Table Aquifer

The Water Table aquifer consists of the undifferentiated sands, clays, and silts of the Barnwell Group, and discontinuous deposits of the Utley limestone. The Utley limestone member of the Barnwell Group consists of sand, clay, and silt with carbonate-rich layers. The Utley limestone was found to be present in 36 percent of the borings completed for the ESP and COL subsurface investigations. It tends to be present in the power block area for VEGP Units 3 & 4 and the area to the north towards Mallard Pond and south towards the VEGP Units 3 & 4 cooling towers. Where present the base of the Utley Limestone ranges in elevation from approximately 96 ft msl to 152 ft msl. Figure 3 shows isopachs of the Utley limestone. The limestone is absent along the flanks of this feature and increases in thickness to a maximum of approximately 25 to 38 ft along its axis. Its total thickness varies considerably, and it is absent in some places within its general area of extent.

Figure 4 shows a hydrogeologic section showing the elevation, thickness, and description of subsurface materials of the Water Table aquifer at the VEGP site. This figure conceptually represents a cross-section of the single layer groundwater model developed for the VEGP site for this report (see Section 4).

The base of the aquifer is the top of the BBM. Figure 5 shows elevation contours for the top of the BBM based on the interpretation of data from 182 borings that have penetrated the top of the BBM within the VEGP site. The contour map for the top of the BBM was developed from borehole data generated from VEGP Units 1 & 2, and VEGP Units 3 & 4 ESP and COL hydrogeologic subsurface investigations. Figure 5 also shows the outcrop of the BBM based on the site geologic map presented in the ESP, reproduced in Figure 6, and the exposure of the Lisbon Formation as shown in the geologic map prepared by Huddlestun and Summerour (Ref. 8), reproduced in Figure 7. The outcrop of the BBM defines the horizontal extent of the Water Table aquifer.

2.4   Groundwater Flow Conditions

Groundwater level data from June 2005 through July 2007 for the Water Table aquifer that were used in the ESP application are summarized in Table 1. Figure 8 shows the hydrographs for all the monitoring wells. As can be seen in this figure the water level data show practically no seasonal variability.

The groundwater elevation data summarized in Table 1 were used to develop groundwater surface elevation contour maps for the Water Table aquifer on a quarterly basis. These maps are presented in Figure 9 through Figure 14 for June 2005 through November 2006. For each quarter, the spatial trend in the piezometric surface is similar, with elevations ranging from a high of approximately El. 165 ft msl in the vicinity of well OW-1013 to a low of less than El. 135 ft msl at well OW-1005. The groundwater surface contour maps indicate that horizontal groundwater flow across the VEGP site is in a north-northwest direction toward Mallard Pond (also known as Mathes Pond), which is a local discharge point for the shallow groundwater flowing beneath the VEGP site. As can be seen in Figure 9 through Figure 14 the water table contours change very little around the year. The horizontal hydraulic gradient across the site for the Water Table aquifer remains practically constant, as can be seen in the in the six water table contour maps shown in Figure 9 through Figure 14 and is approximately 0.014 ft/ft.

A feature of interest for the analysis of the pathway of potential liquid effluent releases is the location of the groundwater divide which separates the area where groundwater flow is primarily to the north from the area where groundwater flow is to the south. As can be seen in Figure 9 through Figure 14 the divide passes close to the south end of location of the cooling towers of Units 3 & 4. Its location does not seem to change around the year.

2.5   Surface Water Features of Interest

There are several small ponds in the vicinity of Units 3 & 4 (see Figure 15). To the north there is Mallard Pond with a water surface elevation of about 111 ft msl. Four relatively large ponds are located to the south of Units 3 & 4. Debris Basin 1, with a water surface elevation of about 135 ft msl, captures runoff from a drainage area of about 200 acres. The Met Tower pond, with water surface elevation of about 221 ft msl, captures runoff from a 50-acre drainage area. The Upper Debris Basin 2 receives runoff from the Units 3 & 4 power block area. It stores runoff until the water surface elevation reaches the invert elevation of 148.5 ft msl of three 36-inch diameter pipes and then discharges into the Lower Debris Basin. Finally, the Lower Debris Basin impounds water at elevation of about 138.5 ft msl. A minimal flow is maintained from the discharge pipe. The drainage area of the Lower Debris Basin is about 680 acres.

Table 1   Monthly Groundwater Level Elevations in Water Table Aquifer

Well No.	142	179	802A	803A	804	805A	806B	808	809	LT-1B	LT-7A	LT-12	LT-13	OW-1003	OW-1005	OW-1006	OW-1007	OW-1009	OW-1010	OW-1012	OW-1013	OW-1015
Jun 05	154.4	147.4	157.9	160.0	163.7	158.5	155.6	158.9	152.8	154.9	154.4	158.2	156.1	155.9	133.0	147.7	151.8	162.4	163.1	161.8	165.0	159.6
Jul 05	154.4	148.4	157.9	159.9	163.6	158.6	155.7	159.1	152.7	154.8	154.2	157.9	155.9	155.9	132.7	147.5	151.7	162.4	163.3	161.9	165.0	159.6
Aug 05	154.5	148.4	158.1	160.2	163.9	158.8	155.8	159.4	152.8	155.0	154.3	158.1	156.1	156.1	132.9	147.6	151.8	162.7	163.6	162.1	165.3	159.8
Sep 05	154.6	148.7	158.2	160.3	164.1	159.0	155.9	159.6	152.9	155.2	154.5	158.2	156.3	156.3	133.0	147.6	151.6	162.9	163.8	162.1	165.5	159.9
Oct 05	154.8	148.7	158.3	160.4	164.2	159.1	156.0	159.5	153.0	155.2	154.5	158.3	156.3	156.2	132.7	147.5	151.5	163.0	163.8	162.0	165.5	160.0
Nov 05	154.7	148.8	158.3	160.5	164.2	159.1	156.0	159.4	153.0	155.2	154.5	158.3	156.4	156.4	132.7	147.2	151.2	163.0	163.8	161.8	165.4	160.0
Dec 05	154.6	148.5	158.3	160.4	164.1	159.1	155.9	159.2	153.0	155.1	154.3	158.2	156.2	156.3	132.5	147.2	151.1	162.9	163.6	161.7	165.2	159.8
Jan 06	154.7	148.6	158.3	160.4	164.1	158.9	156.0	159.0	153.1	155.2	154.6	158.5	156.4	156.3	132.7	147.4	151.4	162.9	163.6	161.8	165.3	159.8
Feb 06	154.8	148.6	158.4	160.5	164.2	158.9	156.0	159.2	153.2	155.5	154.8	158.7	156.7	156.4	133.0	147.4	151.5	163.0	163.6	161.9	165.5	159.8
Mar 06	154.7	148.7	158.2	160.5	164.3	159.0	156.0	159.2	153.2	155.3	154.6	158.5	156.4	156.4	133.1	147.4	151.5	163.0	163.6	161.8	165.3	159.9
Apr 06	154.6	148.7	158.2	160.3	164.1	158.8	155.9	159.0	153.1	155.2	154.6	158.5	156.3	156.3	133.1	147.4	151.2	162.9	163.4	161.7	165.2	159.8
May 06	154.6	148.8	158.1	160.2	164.0	158.8	155.8	158.5	153.0	155.2	154.5	158.5	156.3	157.2	133.2	147.1	151.1	162.8	163.3	161.5	165.1	159.7
Jun 06	154.5	148.8	158.0	160.1	163.9	158.6	155.7	158.8	153.0	155.0	154.4	158.2	156.2	156.2	133.1	147.1	151.0	162.7	163.1	161.4	165.0	159.6
Jul 06	154.4	148.6	157.9	160.0	163.7	158.5	155.7	158.7	152.9	155.0	154.3	158.2	156.1	156.0	132.9	146.9	150.8	162.5	162.9	161.2	164.8	159.5
Aug 06	154.4	148.8	157.9	159.9	163.7	158.5	155.6	158.7	152.9	155.0	154.3	158.2	156.1	156.0	132.8	146.8	150.5	162.4	162.8	161.0	164.7	159.4
Nov 06	154.2	148.8	157.6	159.6	162.8	158.2	155.4	158.4	152.7	154.8	154.3	158.1	155.9	155.9	132.5	146.5	150.1	162.2	162.5	160.5	164.3	159.1
Dec 06	154.0	148.8	157.4	159.5	163.2	158.0	155.2	158.4	152.6	154.6	154.0	157.8	155.8	155.7	132.4	146.3	149.9	162.0	162.3	160.3	164.0	158.8
Jan 07	154.0	148.6	157.2	159.3	163.0	158.8	155.1	158.0	152.6	154.5	154.0	157.8	155.6	155.6	132.3	146.3	150.1	161.7	162.2	160.2	163.8	158.6
Feb 07	154.0	148.9	157.2	159.3	163.0	157.7	155.1	158.0	152.7	154.3	153.7	157.5	155.4	155.9	132.5	146.5	150.2	161.9	162.2	160.3	163.9	158.6
Mar 07	153.9	148.5	157.7	159.3	163.1	157.7	155.1	158.2	152.6	154.4	153.7	157.6	155.6	155.7	132.3	146.1	150.3	161.8	162.4	160.2	163.8	158.5
Apr 07	153.8	148.5	156.9	158.9	162.5	157.4	154.9	158.0	152.4	154.3	153.7	157.5	155.3	155.5	132.2	146.0	150.1	161.7	162.5	160.2	163.7	158.2
May 07	153.6	148.4	156.8	158.8	162.6	157.3	154.7	158.1	152.3	154.0	153.2	157.0	155.1	155.3	132.1	145.6	150.0	161.5	162.3	160.0	163.5	158.1
Jun 07	153.6	148.4	156.8	158.8	162.7	157.3	154.7	158.0	152.3	154.0	153.4	157.2	155.1	155.4	132.0	145.7	149.9	161.5	162.3	160.0	163.5	158.0
Jul 07	153.6	148.4	156.8	158.8	162.8	157.3	154.7	157.9	152.3	153.9	153.3	157.1	155.1	155.3	132.1	145.6	149.7	161.4	161.9	159.8	163.4	157.9

Groundwater level data are from SSAR Table 2.4.12-1.
No groundwater data were measured in September and October of 2006.

2.6   Net Infiltration

The net infiltration, or groundwater recharge, accounts for the rate of net gain of the groundwater system resulting from surface infiltration.   The presence of porous surface sands and the moderate topographic relief in the VEGP site area suggest that a significant fraction of the precipitation infiltrates the ground.

Although a water budget for the VEGP site has not been established, recharge and discharge rates in the region have been estimated on a basin-wide basis by different investigators. For example, in 1997 Clarke and West (Ref. 4, p. 93) estimated the long-term average groundwater recharge in the nearby Savannah River basin to be 14.5 in/yr, of which 6.8 inches is to the local flow system, 5.8 inches is to the intermediate flow system, and 1.9 inches is to the regional flow system.   The local flow system described by Clarke and West (Ref. 4) is characterized by relatively shallow and short flow paths. This system in the Savannah River basin is primarily the Upper Three Runs aquifer, which is equivalent to the Water Table aquifer at the VEGP site. Therefore, it is reasonable to assume that the recharge to the Water Table aquifer is about the same as that estimated for the local flow system described by Clarke and West, i.e. of the order of 6.8 in/yr.

Mean-annual precipitation in the basin ranges from 44 to 48 inches. Considering the distribution of precipitation in the Savannah River basin (see Figure 16) the rate of recharge to the Water Table aquifer at the VEGP site may be a little higher than the estimated average value (6.8 in/yr) over the entire basin. The estimate of the total rate of recharge provided in Ref. 4 (14.5 in/yr) is consistent with the groundwater recharge estimates obtained from several studies at the Savannah River Site, which are summarized in Table 2. It is also noted that the recharge at the VEGP site may be lower than that at the Savannah River Site because of greater topographic relief, which produces more runoff. The Savannah River Site is flatter than the VEGP site.

The rate of groundwater recharge within the model domain is expected to vary depending on the characteristics of the ground surface.   Five different recharge zones are defined listed in approximate order of decreasing recharge:

- Open areas on mild slopes.
- Forested areas on mild slopes.
- Open areas on steep slopes (greater than 10 percent).
- Forested areas on steep slopes (greater than 10 percent).
- Buildings, paved areas and areas covered with slabs

A direct estimation of the recharge rate for each of these zones requires systematic data for the water budget in the surface layer influenced by evapotranspiration. Because such data are not available the recharge rate in each of these zones is treated as a calibration parameter.

Table 2   Estimated Recharge Values at Different Parts of the Savannah River Site

Modeled Savannah River Site Area	Global Recharge Values (in/yr)	Report Title	Report Authors	Date	Document Number
RRSB	15	Groundwater Flow and Contaminant Transport Modeling in Support of the RRSB Operable Unit (U)	INTERA	March 2003	WSRC-RP-2002-4081
CMP Pits	10 12	Groundwater Modeling for the Chemicals, Metals, and Pesticides Pits (U)	G.W. Council, GeoTrans L.M. Grogin	October 2002	WSRC-RP-2002-4195
D Area	8 15	Flow and Transport Modeling for D-Area Groundwater (U)	K.E.Brewer C.S. Sochor	October 2002	WSRC-RP-2002-4166
Southern Sector	9	Groundwater Modeling for the Southern Sector of A/M Area (U)	GeoTrans	November 2001	WSRC-RP-2001-4254
CBRP	12.5	Groundwater Modeling for the C-Area Burning/Rubble Pit (U)	GeoTrans	October 2001	WSRC-TR-2001-00298
CRGW	12.5	Groundwater Transport Modeling for Southern TCE and Tritium Plumes in the C-Area Groundwater Operable Unit (U)	T.L.Fogle K.E. Brewer	June 2001	WSRC-TR-2001-00206
CRSB/ CBRP	11.6	Hydrogeological Analysis and Groundwater Flow for C-Reactor Area with Contaminant Transport for C-Reactor Seepage Basins (CRSB) and C-Area Burning Rubble Pit (CBRP) (U)	G.P. Flach; M.K. Harris; R.A. Hiergesell; A.D. Smits	December 1999	WSRC-TR-99-00310
LBRP	12.5	Groundwater Flow and Transport Modeling for the L-Area Burning/Rubble Pit (131-L), L-Area Rubble Pile (131-3L), and L-Area Gas Cylinder Disposal Facility (131-2L) Savannah River Site, Aiken, South Carolina (U)	D.C. Noffsinger	November 1999	WSRC-RP-99-4154
MCB/MBP	15	Groundwater Flow and Transport Modeling for the Miscellaneous Chemical Basin and Metals Burning Pit (731-4A/5A) (U)	D.C. Noffsinger	March 1999	WSRC-RP-98-4167
CKLP	12.5	Hydrogeological and Groundwater Flow Model for C, K, L, and P Reactor Areas, Savannah River Site, Aiken, SC (U)	G.P. Flach; M.K. Harris; R.A. Hiergesell; A.D. Smits	September 1998	WSRC-TR-98-00285
KBRP	10 to 17	Groundwater Flow and Solute Transport Modeling Report K-Area Burning Rubble Pit and Rubble Pile	GeoTrans	June 1998	WSRC-RP-98-5052

2.7　Hydraulic Conductivity

Hydraulic conductivity tests for VEGP Units 1 & 2 performed in the Barnwell sands and the Utley limestone are described in the VEGP Final Safety Analysis Report (FSAR) for Units 1 & 2 (Ref. 5). These tests consisted of two in-situ constant head tests and three laboratory tests on undisturbed samples of the Barnwell sands. The results of the hydraulic tests are presented in FSAR (Ref. 5) Table 2.4.12-12 and are summarized in Table 3. As can be seen in Table 3, the measured hydraulic conductivity values range from 0.03 ft/day to 0.8 ft/day.

Hydraulic conductivity tests performed in the Utley limestone consisted of five pumping tests, seven falling head and four constant head tests. The results are presented in FSAR Table 2.4.12-13 and are summarized in Table 3. The pumping test yielded hydraulic conductivity values ranging from 8.9 ft/day to 343 ft/day, with a geometric mean of 60.2 ft/day. The falling head and constant head tests yielded hydraulic conductivity values ranging from 0.38 ft/day to 16 ft/day. The geometric mean of the 7 falling head tests was 1.7 ft/day. The geometric mean of 4 constant head tests was 2.8 ft/day. It is noted that the hydraulic estimates from the pumping tests are more than an order of magnitude higher than those estimated from the slug tests. Also, comparing hydraulic conductivity values obtained with the same method we find that the hydraulic conductivity of the Utley limestone (2.8 ft/day, the geometric mean of 4 constant head tests) is a little less than five times that of the Barnwell sands (0.6 ft/day, the geometric mean of 2 constant head tests).

The hydraulic conductivity of the structural fill for the VEGP Units 1 & 2 was determined from slug tests (see Table 2.4.12-15 in Ref. 5). The results of these tests are presented in Table 4, which shows that the measured hydraulic conductivity values of the fill ranged from 1.32 to 3.34 ft/day, with a geometric mean of 2.32 ft/day.

For the VEGP Units 3 & 4 sites, hydraulic conductivity values for the Water Table aquifer were determined from slug tests performed in the groundwater observation wells installed at the site. The results of these tests are presented in Table 5. Table 5 shows that the wells are screened in portions of the Barnwell sands and Utley limestone with hydraulic conductivity values ranging from 0.12 to 2.65 ft/day, and a geometric mean of 0.5 ft/day. This value is of the same order of magnitude as the geometric mean of the hydraulic conductivity estimated from the falling and constant head tests for Units 1 & 2 (1.7 and 2.8 ft/day for wells screened in the Utley limestone, and 0.6 ft day for the Barnwell sands). Figure 17 shows the spatial distribution of the hydraulic conductivity values estimated from the hydraulic tests for Units 3 & 4. As can be seen from this figure there is no distinct pattern in the distribution of the hydraulic conductivity values.

Table 3   Hydraulic Conductivity for Water Table Aquifer - VEGP Units 1 & 2 Site

Well No.	Test Interval (ft bgs)	Hydraulic Conductivity (ft/day)	Geometric Mean (ft/day)
Utley Limestone			
		Pumping Test Results	60.2
1A	56 – 78	39	
1B	68 – 78	343	
1C	56 – 80	55	
1D	56 – 80	121	
2A	62 – 85	8.9	
		Falling Head Test Results	1.7
W-1	65 – 80	16	
1A	63 – 78	1.6	
W-2	69 – 85	2.7	
2A	70 – 85	0.26	
2B	69 – 84	1	
2C	65 – 85	0.38	
2D	70 – 85	5.7	
		Constant Head Test Results	2.8
1A	56 – 78	0.44	
2A	56 – 85	8.8	
2B	56 – 84	4.9	
2D	56 – 85	3.3	
Barnwell Sands			
		Constant Head Test Results	0.6
183	50 – 60	0.5	
184	53 – 63	0.7	
		Laboratory Test Results	0.1
107A	13.8 – 14.4	0.8	
	34 – 36	0.03	
	62.5 – 63	0.08	

Source: FSAR Table 2.4.12-12 and Table 2.4.12-13.

Table 4   Hydraulic Conductivity for Fill Materials - VEGP Units 1 & 2 Site

Well ID.	Hydraulic Conductivity	
	(ft/year)	(ft/day)
LT-1B	1220	3.34
LT-7A	750	2.05
LT-12	480	1.32
LT-13	1180	3.23
Geometric Mean (ft/day)		2.32

Source: FSAR Table 2.4.12-15.

Table 5   Hydraulic Conductivity Values for Water Table Aquifer - VEGP Units 3 & 4 Site

Observation Well No.	Test Interval (ft bgs)	Hydraulic Conductivity (ft/day)
OW-1003	72 – 91	0.12
OW-1005	143 - 169	0.32
OW-1006	113 - 136	1.4
OW-1007	99 – 120	2.65
OW-1009	81 – 98	1.1
OW-1010	70 – 92	0.18
OW-1012	71 – 94	0.39
OW-1013	81 – 104	0.38
OW-1015	90 – 120	0.44
Geometric Mean (ft/day)		0.50

Source: Vogtle ESP Application, SSAR Table 2.4.12-3

3. THE GROUNDWATER MODEL

3.1 The Conceptual Hydrogeologic Model

Based on the aquifer description presented in Section 2 the Water Table aquifer was conceptualized as a single hydrogeologic unit. No distinction is made between the Barnwell sands and the Utley limestone. The aquifer is described as a single material. Because of the low permeability of the BBM, there is negligible vertical flow from the Water Table aquifer to the underlying Tertiary aquifer. Therefore, the flow through the aquifer can be described as two-dimensional in the horizontal plane. The BBM is treated as impermeable. The top of the BBM forms the bottom of the aquifer.

The model domain was selected in such a manner as to minimize the impact of assumptions regarding boundary conditions on predictions in the area of Units 3 & 4 and their vicinity. The boundaries of the model domain were placed where reasonable assumptions regarding local conditions could be made. Figure 18 shows the selected model domain. It extends about 1 mile to the north, 2 miles to the south and about 1 mile to the west of Units 3 & 4. To the east the model extends to the outcrop of the BBM at the bluff along the Savannah River.

The northern boundary and a little more than the northern half of the western boundary of the model coincides with the surface water divide. It is assumed that in this area the groundwater divide coincides with the surface water divide. Therefore, this boundary is treated as a no-flow boundary.

The northeastern, southeastern and part of the southwestern boundary of the model is defined by the outcrop of the BBM, which represents the horizontal extent of the Water Table aquifer, and where the aquifer discharges. This interpretation is supported by observations of small springs and seeps along the bluff. The zone of groundwater discharge near the outcrop of the BBM is described in the model by a series of drains. Mallard Pond was treated as a constant head area.

3.2 The Numerical Model

3.2.1 The Numerical Code

The conceptual hydrogeologic model was implemented in a two-dimensional, single layer numerical groundwater model using the code MODFLOW 2000 (Ref. 7). MODFLOW solves the three-dimensional ground-water flow equation using a finite-difference method. It has been widely used in the industry since its development and release by the U.S. Geological Survey in 1984.

From its inception MODFLOW had a modular structure that allowed the incorporation of additional modules and packages to solve other equations that are often needed to handle specific groundwater problems (Ref. 8). Over the years several such modules and packages

have been added to the original code. MODFLOW 2000 is major revision of the code that expanded upon the modularization approach that was originally included in MODFLOW.

To facilitate the development of the present model the user interface Visual MODFLOW (Ref. 9) was used. Visual MODFLOW was developed by Waterloo Hydrogeologic Software (WHS), which is now part Schlumberger.

3.2.2   The Numerical Solver

Visual MODFLOW includes several different solvers for the numerical solution of the groundwater flow equations. They include the Preconditioned Conjugate-Gradient (PCG), the Strongly Implicit Procedure (SIP) package, the Slice-Successive Overrelaxation (SOR), the Waterloo Hydrogeologic Services (WHS), the Algebraic Multigrid Method (AMG) and the Geometric Multigrid Solver (GMG) package. After several tests it was determined the WHS solver produced converged solutions in most cases, while most of the other solvers did not. A brief description of the method used by each of the solvers is given in Ref. 9.

It was also found that for many combinations of parameters the iterative solution did not converge. To achieve convergence it was necessary to adjust the numerical parameters that affect the solver. A parameter in the WHS solver that was adjusted during several iterations was the "damping factor," which is used to reduce or "dampen" the head change calculated between successive outer iterations. As stated in Ref. 9 (page 294) for most well posed groundwater flow problems, a dampening factor of one can be used. However it was found that in this particular problem a much smaller dampening factor must be used. In some cases it was necessary to use a value as low as 0.1 or 0.05 in order to obtain a converged solution. The effect of reducing the dampening factor is to slow down the convergence speed and increase the number of required outer iterations. In some cases more than 10,000 iterations were needed for convergence.

Another numerical parameter that affects the obtained solution is the head change criterion. This is based on the maximum change between iterations at any cell. A quite small head change criterion was needed in most cases in order to obtain a mass balance discrepancy less than one percent. The default value for the head change criterion used in Visual MODFLOW is 0.01. In most simulations presented in this report a value of 0.005 was used.

3.2.3   The Numerical Grid

Figure 18 shows the rectangular area covered by the numerical grid of the model. Figure 19 shows the numerical grid and the boundary conditions used in the model. The grid spacing in the area surrounding existing Units 1 & 2 and the planned Units 3 & 4 is 100 ft by 100 ft. In areas away from these areas of interest the grid is coarser. The largest grid size used in the model is 200 ft by 200 ft. Figure 19 shows also the active cells of the model that represent the model domain described in Section 3.1. Grid cells outside this area are inactive. The model covers an

area of approximately 6 square miles. Figure 20 shows several physical features included in the model.

3.2.4 The Vertical Extent of the Model

Vertically the model is bounded by the ground surface at its top and the top of the BBM at its bottom. The topography used in the model is based on the LIDAR data for the area covered by the aerial survey conducted as part of the ESP for Units 3 & 4 (Ref. 12), and on USGS Digital Elevation Model (DEM) data for the rest of the model domain (Ref. 13). Figure 21 shows the bottom surface of the model domain as described in the model.

3.2.5 Types of Boundary Conditions Used in the Model

As explained in Section 3.1, the boundaries of the model domain were selected to coincide with key physical features that allow the definition of boundary conditions. Four different types of flow boundary conditions were used for the development of the model: drain, constant head, recharge and no flow boundaries. A brief description of these four conditions as they are defined and used in MODFLOW is provided below:

- Drain Boundary: The drain boundary condition in MODFLOW is designed to simulate the features that remove water from the aquifer at a rate equal to the product of the conductance of the drain and the difference between the head in the aquifer and a given level associated with the drain. Drain boundaries are used to simulate the effect of agricultural drains or seepage faces where groundwater discharges to the surface. The latter can happen along steep slopes or escarpments. In such cases the drain elevation corresponds to the ground surface elevation. When the water level reaches the ground surface elevation it is removed by the drain boundary. The drain has no effect if the head in the aquifer falls below the fixed elevation of the drain. The conductance of drains used to represent a seepage face is proportional to the area of the drain cells, and depends on the materials near the seepage face that may affect discharge conditions. In general the conductance of drain cells is treated as a calibration parameter.

- Constant Head Boundary: The constant head boundary condition is used to fix the head value in selected grid cells. The effect of the constant head condition is to provide a source of water entering the system, or a sink for water leaving the system, depending on the head conditions in the surrounding grid cells.

- Recharge Boundary: The recharge boundary condition is applied at the ground surface and is used to simulate the effect of groundwater recharge applied. Such recharge represents the net gain of the groundwater system as a result of deep infiltration resulting from precipitation, after the effect of evapotranspiration losses have taken into account. The recharge boundary condition can also used to describe artificial recharge or seepage from a pond.

● No Flow Boundary: This is the default boundary condition in MODFLOW when no other boundary condition is defined. It is used to describe no flow boundaries, such as the groundwater divide, or those resulting from impermeable neighboring materials.

3.2.6   The Numerical Solver

Visual MODFLOW offers the option of selecting from several built-in numerical solvers of the partial differential flow equations. Past experience with Visual MODFLOW has shown that the Waterloo Hydrogeologic Software (WHS) solver performs best in terms of numerical convergence.

The WHS solver was used for all solutions presented in this report. The WHS solver uses two convergence criteria, the head change between successive outer iterations and the residual criterion which is based on the change between successive inner iterations. The head change criterion used was 0.005 ft, and the residual change criterion was 0.001 ft.

3.3   Assumptions

The development of the model is based on the following assumptions.

3.3.1   Aquifer Materials

● Assumption: The Water Table aquifer can be described as a single hydrogeologic unit, without differentiating between the Barnwell sands and the Utley limestone.

Rationale: The Utley limestone is very discontinuous and absent in places. Where it exists it is found at the base of the Barnwell formation. The available data do not seem to support a clear horizontal and vertical delineation of the Utley limestone as a separate unit. Because of the uncertainty in defining zones of the Utley it is deemed more defensible to describe the aquifer in a simple manner, i.e. as consisting of a single material with hydraulic conductivity close to the geometric mean of the conductivity of the Utley and the Barnwell sands.

3.3.2   Flow Boundary Conditions

● Assumption: The northeastern, southeastern and part of the southwestern boundary of the model, defined by the outcrop of the BBM can be described by the drain boundary condition. The drain elevation at drain cells along this boundary is equal to the ground surface elevation. The conductance of these drain cells along the outcrop of the BBM is 80 ft^2/day.

Rationale: As discussed in Section 3.1, the outcrop of the BBM represents the edge of the Water Table aquifer along the bluff. Seeps and springs have been observed along the bluff. Therefore it is justified to use the drain boundary condition that allows the discharge of groundwater along a seepage face.

Setting the drain elevation equal to the ground surface is standard for the seepage faces (Ref. 9, page 244). The value of the conductance of the drain cells was determined by calibration.

● Assumption: Part of the southwest and northwest boundaries of the model can be treated as no-flow boundaries.

Rationale: This boundary coincides with the surface water divide. In most watersheds the surface water divide coincides with the groundwater divide.

● Assumption: The bottom of the aquifer can be treated as a no-flow boundary

Rationale: The BBM that forms the base of the Water Table aquifer has very low permeability (4.9×10^{-9} to 8.5×10^{-6} cm/s – SSAR page 2.4.12-21) and has an average thickness of 63 ft at the VEGP site (SSAR page 2.4.12-15). Therefore, it can be considered practically impermeable.

● Assumption: Mallard Pond is in hydraulic communication with the Water Table aquifer and can be represented by constant head cells. The constant head value used for Mallard Pond is El. 111 ft msl.

Rationale: The water surface of Mallard Pond is very close to the water table. Therefore, this pond communicates with the water table. The elevation of the water surface in Mallard Pond was obtained by surveying.

● Assumption: The Met Tower pond (see Figure 20) can be treated as a recharge area.

Rationale: The water surface elevation of Met Tower pond is at 221 ft msl, i.e. much higher than the water table in its vicinity. The maximum groundwater elevation recorded at the site of Units 3 & 4 is around 166 ft msl. The Met Tower pond is about half a mile from the site of the cooling towers of Units 3 & 4. The groundwater data collected at the site of Units 3 & 4 suggest that the groundwater divide is quite close to the south end of the cooling towers. Moving south of the divide groundwater levels should be lower. Therefore, the groundwater elevation under the Met Tower pond has to be lower than the maximum groundwater level at the site (166 ft msl). No data exist that would allow to directly estimate the recharge rate under this pond. It is assumed to be 20 in/yr, i.e. about two and a half times the average recharge area over the model domain. Because of the small area of the pond, this assumption is not deemed as having a major impact on the simulated water table at the site of Units 3 & 4. A sensitivity of the predicted water levels to the rate of recharge from this pond is presented in Subsection 4.4.8.2.

3.3.3   Groundwater Recharge

● Assumption: The distribution of groundwater recharge over the model domain can be described by the zones shown in Figure 22 and listed in Table 6:

Table 6   Zones of Different Recharge

Zone	Description	Recharge in/yr
1	Open areas on mild slopes.	8 to 12
2	Forested areas on mild slopes.	6 to 10
3	Open areas on steep slopes (> 10 percent).	6 to 10
4	Forested areas on steep slopes (> 10 percent).	4 to 8
5	Buildings, paved areas and areas covered with slabs	0

Rationale: The five zones shown in Figure 22 and listed in Table 6 were delineated based on the LIDAR topographic survey (Ref. 12), the USGS topographic map (Ref. 13), and observations during a site visit.

The recharge values shown given in Table 6 were determined through calibration. The relative magnitude of groundwater recharge is consistent with basic hydrologic principles. Areas with steep slopes have less recharge than areas on mild slopes and the same vegetation cover because of higher runoff. Forested areas have relatively lower recharge than open areas with the same ground surface slope, because of higher evapotranspiration and interception losses. Paved areas and areas covered with slabs or occupied by buildings have zero recharge. Precipitation falling on these areas runs off and is collected by the existing drainage system at the site.

On average, the aquifer recharge rate at the VEGP site is about the same as the long-term average groundwater recharge estimates provided by Clarke and West (Ref. 4) for the Savannah River basin.

3.3.4   Aquifer Bottom

● Assumption: The bottom of the Water Table aquifer is the top of the BBM, whose contours are shown in Figure 21. There is no leakage through the bottom of the aquifer.

Rationale: The BBM is continuous across the VEGP site and hydraulically isolates the Water Table aquifer from the underlying Tertiary aquifer. Its average thickness at the VEGP site is 63 ft. (SSAR page 2.4.12-15). Laboratory permeability tests were also conducted on core samples collected from the marl. Laboratory measurements ranged from 1.4×10^{-5} to 2.4×10^{-2} ft/day (4.9×10^{-9} to 8.5×10^{-6} cm/s) with a geometric mean of 1.3×10^{-3} ft/day (4.6×10^{-7} cm/s), indicating that the marl is practically impermeable (SSAR page 2.4.12-21).

Part of Figure 21 is based on the interpretation of data from 182 borings that penetrated the marl during the geologic and geotechnical investigations. This part is shown in Figure 5, which is reproduced from SSAR Figure 2.5.1-47.

In the part of the model domain that is beyond the area covered by Figure 5 the top of the BBM was extrapolated. For the purpose of this extrapolation it was assumed that beyond the area covered by the data from the geotechnical and geological investigations at the VEGP site, the top of the BBM gently slopes downwards.

3.3.5   Hydraulic Conductivity

● Assumption: A single value of the hydraulic conductivity can be used to characterize the part of the model domain where hydraulic tests were conducted.

Rationale: As discussed in Section 3.1, the distribution of the estimated hydraulic conductivity values estimated from the hydraulic tests conducted at the site does not suggest a distinct pattern that would provide the basis for the delineation of different hydraulic conductivity zones (see Figure 17). The single value of the hydraulic conductivity that characterizes most of the model domain was determined in the process of the calibration of the model.

● Assumption: The fill material that will be used for the construction of Units 3 & 4 is expected to be similar to that used for Units 1 & 2. The hydraulic conductivity of this material is expected to be of the order of 2 to 3 ft/day.

Rationale: It is expected that the construction of Units 3 & 4 will use fill materials from the same borrow areas that were used during the construction of Units 1 & 2. The geometric mean of the hydraulic conductivity values from different samples of this material reported in the FSAR for Units 1 & 2 (Ref. 5) was 2.3 ft/day. The maximum measured hydraulic conductivity was 3.3 ft/day.

● Assumption: A zone of relatively more permeable materials exists in the northern part of the model domain and especially between the site of the VEGP units and Mallard pond.

Rationale: The hypothesis about the existence of such a zone was introduced during the calibration of the model. It is supported by the geologic and geotechnical investigation which suggest that the Utley limestone is present in this area. The isopachs of the Utley limestone (see Figure 3) were used to delineate areas of higher hydraulic conductivity. More details on the delineation of the higher conductivity zone are discussed in Subsection 4.4.3.

● Assumption: Native materials within each hydraulic conductivity zone are assumed to be homogenous and horizontally isotropic.

Rationale: There is no evidence of anisotropy in the Water Table aquifer materials. General flow patterns are not affected much by local heterogeneities. Therefore, for the purpose of flow modeling aimed at predicting the general flow direction the effect of small scale heterogeneities can be ignored.

4. MODEL CALIBRATION

4.1 Calibration Target

The model was calibrated for the existing conditions at the VEGP site, with Units 1 & 2 in place, by comparing the model simulated groundwater head values with the observed groundwater levels. As discussed in Section 2.4 groundwater levels at the VEGP site have been monitored from June 2005 to July 2007 for this ESP application. The groundwater level data observed at 22 wells during the month of March 2006 were used as representative of the long-term to calibrate the model. Figure 12 shows the groundwater surface contour map prepared based on the March 2006 observed groundwater levels presented in Table 7.

Table 7   March 2006 Observed Groundwater Level used in Model Calibration

Well ID	Easting (ft)	Northing (ft)	Groundwater Level (ft msl)
142	622260.4	1143282.4	154.71
179	621778.7	1144061.2	148.72
802A	624195.0	1142201.7	158.23
803A	622896.0	1142085.4	160.45
804	622224.8	1141599.6	164.30
805A	624395.7	1141616.2	158.98
806B	623724.5	1143821.6	156.03
808	623297.7	1144624.3	159.15
809	621857.2	1143320.4	153.18
LT-1B	623301.3	1143390.5	155.28
LT-7A	623314.3	1143154.1	154.59
LT-12	623597.6	1142776.8	158.48
LT-13	624108.7	1143136.4	156.35
OW-1003	621884.3	1142864.1	156.43
OW-1005	620408.8	1144047.9	133.12
OW-1006	619179.7	1143817.9	147.37
OW-1007	619301.0	1142383.8	151.45
OW-1009	620888.6	1141891.6	163.01
OW-1010	620051.7	1140809.0	163.57
OW-1012	621045.9	1139969.5	161.80
OW-1013	621715.0	1140805.4	165.31
OW-1015	623086.3	1140550.6	159.89

During this monitoring period, the observed groundwater levels have exhibited little variability (see Figure 9 through Figure 14).

Even though the measured groundwater levels in March 2006 are higher than the average groundwater levels over the period of available data (June 2005 to July 2007), they are considered more representative of long-term groundwater conditions because 2007 was a very dry year.

4.2   Calibration Measures and Statistics

Several parameters providing different measures of the agreement between simulated and observed groundwater levels were used for the calibration of the model. These parameters are defined in terms of the calibration residuals of the water table level defined as the difference between calculated and observed results. The calibration residual, R_i, at a point i is defined as:

$$R_i = {}^{model}X_i - {}^{obs}X_i$$

where

${}^{model}X_i$ is the calculated water level at point *i*

${}^{obs}X_i$ is the observed water level at point

The residual mean, $\overline{R}$, is a measure of the average residual value and is defined by the equation:

$$\overline{R} = \frac{1}{n}\sum_{i=1}^{n} R_i$$

where n is the number of points where calculated and observed values are compared.

The absolute residual mean, $\left|\overline{R}\right|$, is a measure of the average absolute residual value and is defined as:

$$\left|\overline{R}\right| = \frac{1}{n}\sum_{i=1}^{n}\left|R_i\right|$$

The Root Mean Squared (*RMS*) residual is defined by:

$$RMS = \left[\frac{1}{n} \sum_{i=1}^{n} R_i^2 \right]^{1/2}$$

The Correlation Coefficient, $Cor\left[{}^{model}X, {}^{obs}X \right]$, is calculated as the covariance between the calculated values with the model and the observed water levels at selected points divided by the product of their standard deviations, i.e.:

$$Cor\left[{}^{model}X, {}^{obs}X \right] = \frac{Cov\left[{}^{model}X, {}^{obs}X \right]}{{}^{model}\sigma {}^{obs}\sigma}$$

where

$Cov\left[{}^{model}X, {}^{obs}X \right]$ is the covariance between the calculated and observed water levels

${}^{model}\sigma$ is the standard deviation of the calculated values with the model

${}^{obs}\sigma$ is the standard deviation of the observed values

The covariance is calculated using the following equation:

$$Cov\left[{}^{model}X, {}^{obs}X \right] = \frac{1}{n} \sum_{i=1}^{n} \left({}^{model}X_i - {}^{model}\overline{X} \right)\left({}^{obs}X_i - {}^{obs}\overline{X} \right)$$

where

$${}^{model}\overline{X} = \frac{1}{n} \sum_{i=1}^{n} {}^{model}X_i$$
is the mean of the water levels calculated with the model at n selected points

$${}^{obs}\overline{X} = \frac{1}{n} \sum_{i=1}^{n} {}^{obs}X_i$$
is the mean of the observed water levels at n selected points

The standard deviation of the water levels calculated with the model is calculated as:

$$^{model}\sigma = \left[\frac{1}{n}\sum_{i=1}^{n}\left(^{model}X_i - ^{model}\overline{X}\right)^2\right]^{1/2}$$

The standard deviation of the observed water levels is calculated as:

$$^{obs}\sigma = \left[\frac{1}{n}\sum_{i=1}^{n}\left(^{obs}X_i - ^{obs}\overline{X}\right)^2\right]^{1/2}$$

The standard error of the estimate (*SEE*) provides a measure of the variability of the residual around the expected residual value. It is given by the equation

$$SEE = \left[\frac{\frac{1}{n-1}\sum_{i=1}^{n}\left(R_i - \overline{R}\right)^2}{n}\right]^{1/2}$$

The normalized root mean squared (*NRMS*) is the *RMS* divided by the maximum difference in the observed head values. It is given by the following equation:

$$NRMS = \frac{RMS}{^{obs}X_{max} - ^{obs}X_{min}}$$

In addition to calculating the parameters described above for each calibration simulation, Visual MODFLOW provides also a plot of the simulated vs. the observed water level values, which provides a way of visualizing the agreement between model and measured values. An example of such a plot is given in Figure 24. The same figure shows also the range of calculated values for each observed value with 95 percent confidence that the simulation results will be acceptable for a given observed value. In a successful calibration the line representing the perfect match between modeled and observed values, i.e. the line along which the modeled values are equal to the observed values, should be within the 95% confidence interval. The plot of simulated vs. observed water levels shown in Figure 24 also shows the 95% interval, defined as the interval where 95% of the total number of data points are expected to occur.

Finally, additional measure of the adequacy of each run is the discrepancy between inflows and outflows from the model domain. To satisfy the overall mass balance, this discrepancy should be

zero. In practice though, this may not be possible. The aim in calibrating and developing the groundwater model for the VEGP site was to make the mass balance discrepancy as small as possible. The mass balance discrepancy, Md, is calculated using the following equation:

$$M_d = \frac{V_{in} - V_{out}}{\frac{1}{2}(V_{in} + V_{out})}$$

where

V_{in} is the total flow into the model domain

V_{out} is the total flow out of the model domain

Most of the calibration measures and statistics discussed above are reported for all the simulations leading to the calibration of the model presented in this report (see Table 8). In addition to these parameters, the maximum residual is also reported in this table.

4.3   Calibration Criteria

Using the calibration measures and statistics the following criteria were used for calibration of the model:

a. Root mean squared residual *RMS* < 3 ft

b. Normalized root mean squared residual *NRMS* < 10 percent

c. Absolute value of maximum residual < 6 ft

d. Mass balance discrepancy M_d < 1 percent

e. A simpler model that meets these criteria is preferable over a more complex model that also meets the same criteria.

4.4   Model Calibration Process

The primary calibration parameters were the hydraulic conductivity and the aquifer recharge rate. These two parameters were varied to achieve satisfactory agreement between simulated and observed water levels according to the calibration criteria stated in Section 4.3. The conductance of the drains, which has a smaller impact on the solution obtained with the model, was also varied. However, in the set of calibration runs presented here it was kept constant (equal to 80 ft^2/day) to better illustrate the impact of the major calibration parameters.

The calibration effort started with the simplest set of assumptions, a uniform hydraulic conductivity value over the entire model domain and a uniform recharge. Zones of different hydraulic conductivity and groundwater recharge zones were progressively introduced where their presence could be supported by local conditions and where it seemed to improve the calibration of the model. This process is described in Subsections 4.4.1 through 4.4.7, where

each conceptual configuration of different hydraulic conductivity and recharge zones is referred to as a different "model".

The calibration was achieved through a series of simulations using different values of the key parameters involved. An attempt to use the nonlinear parameter estimation and optimization package PEST (Parameter Estimation) built into Visual MODFLOW was not successful because the code presented serious convergence problems. As a result, a decision was made to calibrate the model using the conventional approach of trying different parameter values moving progressively towards the values that best satisfy the calibration criteria. The sequence of runs documenting the sequence of the model calibration process and the key parameters used in these runs are listed in Table 8.

Table 8   Summary of Simulated Cases During Calibration

Run	Hydraulic Conductivity [1]					Recharge [2]					Calibration statistics [3]						M_d
	K_1	K_2	K_3	K_4	K_5	R_1	R_2	R_3	R_4	R_5	MR	ARM	SEE	RMS	NRMS	CC	
	Ft/d	ft/d	ft/d	Ft/d	Ft/d	in/yr	in/yr	in/yr	in/yr	in/yr	ft	ft		ft	%		%
Model 1: Uniform Hydraulic Conductivity and Recharge																	
101	12					6.8					27.215	13.306	1.06	14.165	44.004	0.761	-0.01
102	15					6.8					22.331	8.854	1.013	9.997	31.056	0.816	-0.02
103	18					6.8					18.741	5.738	0.986	7.204	22.38	0.838	-0.02
104	21					6.8					15.964	4.263	0.969	5.433	16.876	0.843	-0.02
105	24					6.8					13.724	3.539	0.96	4.544	14.116	0.841	-0.02
106	27					6.8					11.867	3.464	0.954	4.4	13.669	0.835	-0.02
107	30					6.8					10.285	3.605	0.951	4.742	14.732	0.827	-0.02
108	27					6					-9.784	3.861	0.96	5.041	15.662	0.819	-0.02
109	27					7					12.389	3.449	0.953	4.369	13.572	0.838	-0.02
110	27					8					14.901	3.815	0.951	4.928	15.308	0.85	-0.01
Model 2: Uniform Hydraulic Conductivity, Variable Recharge																	
201						10	6	6	4	0	10.857	3.436	0.954	4.375	13.592	0.786	-0.02
202						10	5	6	4	0	9.155	3.726	0.98	4.683	14.549	0.755	-0.01
203						10	7	6	4	0	12.47	3.513	0.938	4.427	13.752	0.814	-0.02
204						10	6	6	3	0	10.455	3.432	0.953	4.382	13.614	0.782	-0.02
205	27					10	6	6	2	0	10.048	3.451	0.953	4.403	13.68	0.779	-0.01
206						10	6	6	5	0	11.254	3.465	0.956	4.382	13.612	0.789	-0.02
208						9	6	6	4	0	10.678	3.438	0.954	4.428	13.755	0.797	-0.02
209						11	6	6	4	0	11.82	3.604	0.958	4.481	13.921	0.782	-0.02
211						10	6	5	4	0	11.813	3.598	0.958	4.477	13.907	0.782	-0.02
212						10	6	7	4	0	11.828	3.611	0.958	4.485	13.934	0.781	-0.02

Table 8   Summary of Simulated Cases During Calibration (cont.)

Run	Hydraulic Conductivity [1]					Recharge [2]					Calibration statistics [3]						M_d
	K_1	K_2	K_3	K_4	K_5	R_1	R_2	R_3	R_4	R_5	MR	ARM	SEE	RMS	NRMS	CC	
	Ft/d	ft/d	ft/d	Ft/d	Ft/d	in/yr	in/yr	in/yr	in/yr	in/yr	ft	ft		ft	%		%
Model 3: Accounting for the thickness of the Utley limestone																	
301	27	15	30	60							12.021	4.705	0.934	5.55	17.241	0.769	-0.01
302	27	15	30	45							12.026	4.755	0.932	5.639	17.519	0.77	-0.01
303	27	15	15	60		10	6	6	4	0	15.052	5.574	0.919	6.733	20.915	0.783	-0.01
304	27	10	30	60							13.458	7.016	1.031	8.267	25.682	0.739	-0.02
305	27	20	30	60							11.244	3.649	0.924	4.463	13.865	0.784	-0.01
306	20	20	30	60							13.742	4.193	0.924	5.115	15.894	0.806	-0.02
Model 4: Simplified representation of the Utley limestone																	
401	27	35									8.86	3.483	0.933	4.457	13.845	0.789	-0.02
402	27	50									-9.951	4.037	0.923	5.12	15.907	0.783	-0.02
403	20	35				10	6	6	4	0	10.865	3.636	0.898	4.371	13.577	0.807	-0.02
404	15	35									12.808	4.969	0.874	5.897	18.32	0.813	-0.02
405	20	25									13.781	4.424	0.923	5.317	16.518	0.809	-0.01
Model 5: High Conductivity Zone Upstream of Mallard Pond																	
501	20	35	50								9.053	3.454	0.876	4.144	12.873	0.81	-0.02
502	15	35	50								10.791	4.711	0.856	5.458	16.954	0.815	-0.02
503	25	35	50								-7.966	3.341	0.9	4.235	13.157	0.799	-0.02
504	20	35	100			10	6	6	4	0	-6.55	3.238	0.851	3.907	12.136	0.813	-0.01
505	20	35	200								-7.428	3.22	0.854	3.932	12.216	0.815	-0.01
506	20	30	100								6.706	3.366	0.841	3.961	12.306	0.818	-0.01
507	20	40	100								-7.441	3.211	0.862	3.966	12.319	0.808	-0.01

Table 8   Summary of Simulated Cases During Calibration (cont.)

Run	Hydraulic Conductivity [1]					Recharge [2]					Calibration statistics [3]						M_d
	K_1	K_2	K_3	K_4	K_5	R_1	R_2	R_3	R_4	R_5	MR	ARM	SEE	RMS	NRMS	CC	
	Ft/d	ft/d	ft/d	Ft/d	Ft/d	in/yr	in/yr	in/yr	in/yr	in/yr	ft	ft		ft	%		%
Model 6: Low Conductivity Zone in the Southwestern Part of the Model Domain																	
601	20	25	50	10							7.619	3.535	0.738	4.064	12.625	0.863	-0.02
602	15	25	200	10							6.905	3.223	0.729	3.715	11.542	0.867	-0.02
603	20	25	200	8							6.259	2.963	0.723	3.477	10.802	0.869	-0.02
604	20	25	200	6							-6.534	2.805	0.719	3.339	10.372	0.871	-0.02
605	20	25	400	6							-6.993	2.745	0.717	3.286	10.207	0.871	-0.02
606	20	25	50	10							-7.418	2.694	0.716	3.303	10.262	0.872	-0.02
607	15	25	200	10		10	6	6	4	0	-7.624	2.74	0.729	3.356	10.425	0.867	-0.02
608	20	25	200	8							-6.537	2.748	0.709	3.264	10.14	0.875	-0.02
609	20	25	200	6							6.052	2.798	0.688	3.338	10.37	0.883	-0.02
610	20	25	400	6							-6.14	2.764	0.683	3.254	10.109	0.884	-0.02
611	20	25	50	10							6.032	2.944	0.676	3.487	10.865	0.889	-0.02
612	15	25	200	10							-7.043	2.742	0.707	3.253	10.106	0.882	-0.02
613	20	25	200	8							-7.428	2.733	0.727	3.332	10.353	0.88	-0.02
614	20	25	200	6							6.687	2.781	0.681	3.34	10.377	0.885	-0.02

Table 8   Summary of Simulated Cases During Calibration (cont.)

Run	Hydraulic Conductivity [1]					Recharge [2]					Calibration statistics [3]						M_d
	K_1	K_2	K_3	K_4	K_5	R_1	R_2	R_3	R_4	R_5	MR	ARM	SEE	RMS	NRMS	CC	
	Ft/d	ft/d	ft/d	Ft/d	Ft/d	in/yr	in/yr	in/yr	in/yr	in/yr	ft	ft		ft	%		%
Model 7: Simplified Version of Model 6																	
701	28	75	8								7.449	2.98	0.659	3.607	11.205	0.894	-0.02
702	30	75	8								6.771	2.728	0.669	3.277	10.179	0.89	-0.02
703	32	75	8								-6.322	2.626	0.679	3.139	9.752	0.886	-0.02
704	34	75	8								-7.189	2.585	0.689	3.169	9.844	0.882	-0.02
705	32	80	8								-6.419	2.624	0.679	3.13	9.723	0.886	-0.02
706	32	85	8			10	6	6	4	0	-6.509	2.622	0.679	3.124	9.704	0.886	-0.02
707	32	90	8								-6.592	2.621	0.679	3.12	9.692	0.886	-0.02
708	32	100	8								-6.741	2.617	0.68	3.118	9.686	0.885	-0.02
709	32	150	8								-7.259	2.607	0.689	3.164	9.828	0.884	-0.02
710	32	100	9								-6.942	2.632	0.691	3.169	9.845	0.881	-0.02
711	32	100	7								-6.521	2.676	0.673	3.12	9.692	0.888	-0.02

Table 8 Notes

[1] Hydraulic conductivity zones 1 and 2

K_1:   the entire model domain

Hydraulic conductivity zones for Model 3 (the colors next to each conductivity zone are those used in Figure 29)

- ☐ K_1:   Utley limestone < 10 ft thick and north of the area covered by the Utley limestone isopachs
- ▮ K_2:   south of the area covered by the Utley limestone isopachs
- ▮ K_3:   Utley limestone 10 to 20 ft thick
- ▮ K_4:   Utley limestone > 20 ft thick

Hydraulic conductivity zones for Model 4 (the colors next to each conductivity zone are those used in Figure 33)

- ☐ K_1:   area where the Utley limestone was not found to be present and area not covered by the Utley limestone isopachs
- ▮ K_2:   generalized area where the Utley limestone was detected

Hydraulic conductivity zones for Model 5 (the colors next to each conductivity zone are those used in Figure 37)

- ☐ K_1:   area where the Utley limestone was not found to be present and area not covered by the Utley limestone isopachs
- ▮ K_2:   generalized area where the Utley limestone was detected
- ▮ K_3:   high conductivity zone up gradient of Mallard Pond

Hydraulic conductivity zones for Model 6 (the colors next to each conductivity zone are those used in Figure 41)

- ☐ K_1:   area where the Utley limestone was not found to be present and area not covered by the Utley limestone isopachs
- ▮ K_2:   generalized area where the Utley limestone was detected
- ▮ K_3:   high conductivity zone up gradient of Mallard Pond
- ▮ K_4:   low conductivity zone in the southwestern part of the model domain

Hydraulic conductivity zones for Model 7 (the colors next to each conductivity zone are those used in Figure 45)

- ☐ K_1:   Recharge zones (the colors next to each recharge zone are those used in Figure 22) area not covered by the K_2 and K_3 zones
- ▮ K_2:   high conductivity zone up gradient of Mallard Pond
- ▮ K_3:   low conductivity zone in the southwestern part of the model domain

Table 8 Notes (cont.)

[2] Recharge zones (the colors next to each recharge zone are those used in Figure 22)

R_1: open areas with minimal vegetation on mild slopes

R_2: forested areas on mild slopes

R_3: open areas with minimal vegetation on steep slopes

R_4: forested areas on steep slopes

R_5: buildings, paved areas and areas covered with slabs

[3] MR: maximum residual

AMR: absolute residual mean

SEE: standard error estimate

RMS: root mean squared error

NRMS: normalize root mean squared error

CC: correlation coefficient

4.4.1 Model 1: Uniform Hydraulic Conductivity and Recharge

The first step in the calibration process was to use a single value for the hydraulic conductivity over the entire model domain. This value is indicated as K1 in Table 8. To simplify the calibration of the model the fill material for the Units 1 & 2 was ignored. As stated in Section 2.7, at the time of the construction of Units 1 & 2 four slug tests were conducted to determine the hydraulic conductivity of the backfill material used. These tests resulted in values ranging from 1.32 to 3.34 ft/day, with a geometric mean of 2.32 ft/day. In general, slug tests produce lower hydraulic conductivity values than those needed to describe large scale groundwater flows, of the order of the scale of the present groundwater flow model. Pumping tests provide more representative hydraulic conductivity values of average properties over larger scales than those tested with slug tests, and therefore better estimates of hydraulic conductivity for use in models of this scale. In the absence of pumping tests in the fill material, the large-scale hydraulic conductivity of this material is essentially unknown, with the slug tests providing only a lower bound of its value. Therefore, to remove the uncertainty associated with the conductivity of the fill it was decided not to include this material in the calibration runs. It is also noted that the extent of this material is relatively small. A sensitivity analysis using different values for the fill material for Units 1 & 2 showed that its impact on the simulated water levels at the observation well that were used for the calibration of the model is very small. This sensitivity analysis is presented in Subsection 4.4.8.1.

The values of the uniform hydraulic conductivity used for the native materials were in the range of 10 to 20 ft/day. These hydraulic conductivity values are greater than the geometric mean of the hydraulic conductivity values estimated from the slug tests at the site of Units 3 & 4 (0.5 ft/day). No pumping tests were conducted at this site. It is noted that at the nearby site of Units 1 & 2 the geometric mean of the hydraulic conductivity from the pumping test was 60 ft day, while that from slug tests was 0.6 ft/day for the Barnwell Sands and 2.32 ft day for tests in the Utley limestone. Considering that the pumping test values in this case were more than an order of magnitude higher than those from the slug tests, it is reasonable to expect the same would apply to the site of Units 3 & 4 and the rest of the model domain.

Also, a single value of groundwater recharge was used over the entire model domain (indicated as R1 in Table 8). The range of values used was between 6 and 8 inches per year, i.e. between about 13 and 17 percent of the mean annual precipitation at the site). Runs for different values of these parameters are presented in Table 8. Each combination of hydraulic conductivity and recharge is described by a different "run" number. Table 8 gives the input parameters and calibration statistics for runs 101 through 110.

Figure 23 shows the simulated head for the run with the smallest RMS residual (run 109). Figure 24 shows the simulated vs. observed water levels for the same run. Figure 25 shows the

distribution of the estimated residuals for this run. As can be seen in these Figures the RMS residual and the maximum residual (at well OW-1005) are very high.

4.4.2   Model 2: Uniform Hydraulic Conductivity, Variable Recharge

The next step in the calibration process was to vary the rate of groundwater recharge by dividing the area not covered by buildings, slabs or pavements into the four different zones described in Section 2.6. Several combinations of recharge values within the following ranges were tested:

- Open areas on mild slopes: R_1 = 10 to 12 in/yr

- Forested areas on mild slopes: R_2 = 5 to 7 in/yr

- Open areas on steep slopes (greater than 10 percent): R_3 = 5 to 7 in/yr

- Forested areas on steep slopes (greater than 10 percent): R_4 = 2 to 5 in/yr

In all cases the recharge in areas covered by buildings or slabs, or that are paved was set to zero (R5 = 0).

Table 8 gives the input parameters and calibration statistics for different combinations of these recharge zones and hydraulic conductivity zones. Figure 26 shows the simulated head for one of these simulations (run 201). Figure 27 shows the simulated vs. observed water levels for the same simulation. As in the cases of uniform recharge presented in Subsection 4.4.1, the RMS residual and the maximum residual (at well OW-1005) are very high suggesting that a variable recharge distribution by itself is not enough to improve the agreement between modeled and observed water levels. Figure 28 shows the distribution of the estimated residuals for this run.

4.4.3   Model 3: Accounting for the Thickness of the Utley Limestone

The next model tested was based on a non-uniform hydraulic conductivity distribution. As stated in Section 2.7 and shown in Figure 17 there is no distinct pattern in the spatial distribution of the hydraulic conductivity values estimated from the slug tests conducted at the site of Units 3 & 4. Based on the observation that in general the Utley limestone is more permeable than the Barnwell sands (Section 2.7), different hydraulic conductivity zones were defined based on the thickness of the Utley limestone. For this purpose the contours of the thickness of the Utley limestone shown in Figure 3 were used. Four conductivity zones were defined, listed below in order of increasing hydraulic conductivity

- Areas where the thickness Utley limestone is absent (K_1)

- Areas where the thickness Utley limestone is less than 10 ft thick (K_2)

- Areas where the thickness Utley limestone is between 10 and 20 ft (K_3)

- Areas where the thickness Utley limestone is greater than 20 ft (K_4)

As can be seen in Figure 3 the isopachs of the Utley limestone based on the data from the geotechnical and geological investigations at the site is smaller than the groundwater flow model

domain. To define hydraulic conductivity zone over the entire domain, it was assumed that south of the area covered by isopachs shown in Figure 3, the Utley limestone is absent. It was also assumed that north of the area covered by isopachs shown in Figure 3, the Utley limestone is present, but it is less than 10 ft thick. The hydraulic conductivity zones defined based on the available data and the last two assumptions are shown in Figure 29. Simulations with different values for the hydraulic conductivity of each zone were made. The groundwater recharge used in all simulations was the same, based on the five recharge zones discussed in Subsection 4.4.2. The recharge values that resulted in the closest agreement with the observed water levels in Subsection 4.4.2 were used, i.e. R1=10; R2=6; R3=6; R4=4; R5=0 ft/day.

Table 8 gives the input parameters and calibration statistics for different combinations of values for the hydraulic conductivity zones defined in Model 3. Figure 30 shows the simulated head for the combination of hydraulic conductivity values that has the smallest RMS residual (run 305). Figure 31 shows the simulated vs. observed water levels for the same simulation. Figure 32 shows the distribution of the estimated residuals for this run. As can be seen in these Figures, Model 3 does not improve significantly the agreement between simulated and observed water levels. The model continues to significantly overestimate the water level at well OW-1005, and there seems to be a systematic error in the model, as suggested by all negative residuals in southern part of the area covered by data, and all positive residuals in the northern part of this area (see Figure 32).

4.4.4   Model 4: Simplified Representation of the Utley Limestone

A simplified representation of the general areas where the Utley limestone was found to be present was the basis for Model 4. The delineation of the area that was treated as a different hydraulic conductivity zone from the rest of the model is shown in Figure 33. Different combinations of the hydraulic conductivity of this zone (K2) and the rest of the model (K1) were tested. The results are shown in Table 8 which also gives the calibration statistics for these selected simulations with this model. Figure 34 shows the simulated head for the combination of hydraulic conductivity values for Model 4 that has the smallest RMS residual (run 411). Figure 35 shows the simulated vs. observed water levels for the same simulation. Figure 36 shows the distribution of the estimated residuals for this run.

As can be seen from Table 8, Figure 35 and Figure 36 the results of Model 4 are not very different for those obtained with Model 3 and they do not represent much of an improvement for the calibration of the model.

4.4.5   Model 5: High Conductivity Zone Upstream of Mallard Pond

All the simulations with four alternative models presented in Subsections 4.4.1 through 4.4.4 overestimate the water levels in the northern part of the site of Units 3 & 4 and especially at observation at observation well OW-1005. This suggests that the aquifer in this area may be

relatively more permeable. To test this hypothesis several simulations with a higher hydraulic conductivity zone between Mallard Pond and well OW-1005 were performed. Figure 37 shows the delineation of this high conductivity zone.

The calibration statistics for some of the combinations of hydraulic conductivity values tested are given in Table 8.   Figure 38 shows the simulated head for the combination of hydraulic conductivity values for Model 5 that has the smallest RMS residual (run 504). Figure 39 shows the simulated vs. observed water levels for the same simulation.   Figure 40 shows the distribution of the estimated residuals for this run.

As can be seen from Figure 39 and Figure 40 Model 5 improves significantly the agreement between simulated and the measured water levels at well OW-1005, but it does not improve the calibration of the model in the southern part of the model where the simulated heads for all cases are lower than what was measured.

It should also be noted that the high conductivity zone between Mallard Pond and well OW-1005 is in an area not covered by the borings drilled as part of the geological and geotechnical investigation. Therefore, there are no hydraulic test data to support the existence of such a zone.

4.4.6   Model 6: Low Conductivity Zone in the Southwestern Part of the Model Domain

Model 5 does not correctly describe groundwater levels in the southwestern part of the model domain.   To address this limitation of Model 5 a low conductivity zone is defined in the southwestern quarter of the model domain.   Figure 41 shows the delineation of this low conductivity zone together with the other conductivity zones used in the model. The northern boundary of this low conductivity zone coincides with the zero thickness contour of the Utley Limestone, providing evidence for the presence of a lower conductivity zone given the contrast in values between the Utley Limestone and Barnwell Sands as described in Section 2.7 and summarized in Table 3.

Different combinations of values for each of the hydraulic conductivity zones shown in Figure 41 were tested. Table 8 gives the calibration statistics for some of these combinations. Figure 42 shows the simulated head for the combination of hydraulic conductivity values for Model 6 that has the smallest RMS residual (run 612). Figure 43 shows the simulated vs. observed water levels for the same simulation. Figure 44 shows the distribution of the estimated residuals for this run.

4.4.7   Model 7: Simplified Version of Model 6

In the course of the tests with Model 6, it became apparent that satisfactory model calibration is achieved using similar hydraulic conductivity values for zones 1 and 2.   Based on this observation it was decided to introduce a simpler model, where these two zones are combined into a single zone. Figure 45 shows the delineation of the three zones used in Model 7, the high

conductivity zone between Mallard Pond and well OW-1005, the low conductivity zone in the southwestern quarter of the model domain, and the rest of the model domain. Table 8 gives the calibration statistics for selected combinations of the hydraulic conductivity of these three zones.

Figure 46 shows the simulated head for the combination of hydraulic conductivity values for Model 7 that has the smallest RMS residual (run 708). Figure 47 shows the simulated vs. observed water levels for the same simulation. Figure 48 shows the distribution of the estimated residuals for this run.

The solutions obtained with Model 6 and 7 are very similar and are the closest to the measured water levels. Overall, as can be seen from Figure 46 and Figure 47 and Table 8, Model 7 gives the best match with the data. One limitation is that the hydraulic conductivity zones, whose introduction optimizes the model calibration effort, are partially outside the area covered by the geotechnical and geological borings and hydraulic testing for VEGP. As noted in 4.4.6, the northern boundary of the low conductivity zone coincides with the absence of the Utley limestone. However, there is no field data to justify the eastern boundary of the low conductivity zone.

4.4.8 Sensitivity Analysis

4.4.8.1 Hydraulic Conductivity of the Backfill Material Around Units 1 & 2

As mentioned in Subsection 4.4.1, to simplify the calibration process, the backfill material around Units 1 & 2 was ignored. To demonstrate the impact of this assumption, a sensitivity simulation was performed with Model 7, i.e. the model that has the best agreement with the water level measurements. In this simulation the fill material was treated as a separate hydraulic conductivity zone, with a hydraulic conductivity value of 3.3 ft/day, i.e. the highest estimated value from the four slug tests conducted in the fill material during construction of Units 1 & 2 (Figure 49). Figure 50 shows the simulated water levels from this simulation. A comparison of Figure 50 and Figure 46 suggests that the effect of changing the hydraulic conductivity in the backfilled area of Units 1 & 2 by an order of magnitude has very small effect on water levels. The same can be said about the basic calibration statistics. Table 9 gives the calibration statistics for the two values of the conductivity of the fill differing by an order of magnitude. Figure 51 shows the simulated vs. observed water levels for the same simulation. Figure 52 shows the distribution of the estimated residuals for this run.

Table 9   Sensitivity of Calibration Statistics to Units 1 & 2 Backfill

Hydraulic conductivity of Units 1 & 2 backfill	Calibration Statistics						
	MR	ARM	SEE	RMS	NRMS	CC	M_d
Ft/day	ft	ft		ft	%		%
3.3	-6.68	2.708	0.694	3.191	9.912	0.88	-0.02
32[1]	-6.741	2.617	0.68	3.118	9.686	0.885	-0.02

(1)   This is the value for the native materials in the area of Units 1 & 2 that gave the best agreement between modeled and observed values (see Table 8).

4.4.8.2   Rate of Recharge at the Met Tower Pond

The rate of groundwater recharge from the Met Tower pond is unknown. In all calibration runs presented in Subsections 4.4.1 through 4.4.7 the rate of recharge from this pond was assumed to be 20 in/yr. Two sensitivity simulations were performed, one with a rate of recharge equal to that used in the area surrounding the pond, i.e. 6 in/yr, and another with double the rate used in most simulations, i.e. 40 in/yr.

Figure 53 shows the simulated water levels using 6 in/yr for the rate of recharge from the Met Tower pond, and Figure 54 shows the simulated vs. observed water levels for the same simulation. Figure 55 shows the distribution of the estimated residuals for this run. Figure 56 shows the simulated water levels obtained using 40 in/yr and Figure 57 shows the simulated vs. observed water levels for the same simulation. Figure 58 shows the distribution of the estimated residuals for this run. A comparison of Figure 53 (6 in/yr), Figure 46 (20 in/yr) and Figure 56 (40 in/yr) suggests that the effect of the assumed rate of groundwater recharge form the Met Tower pond is very local, with higher rates producing slightly higher water levels under this pond. The effect of the rate of recharge from this pond on the location of the groundwater divide is negligible. The effect of the recharge rate on the agreement between modeled and observed water levels is also negligible. Table 10 gives the calibration statistics for the three runs with the three different recharge rates used.

Table 10   Sensitivity of calibration statistics to the rate of recharge at the Met Tower Pond

Recharge at Met Tower pond	Calibration Statistics						
	MR	ARM	SEE	RMS	NRMS	CC	M_d
in/yr	ft	Ft		Ft	%		%
6	-6.834	2.601	0.682	3.128	9.717	0.884	-0.02
20	-6.741	2.617	0.68	3.118	9.686	0.885	-0.02
40	-6.608	2.64	0.676	3.111	9.664	0.887	-0.02

4.4.8.3   Constant Head Condition at the Upper Debris Basin 2

The Upper Debris Basin 2 was not accounted for directly in all the calibration simulations presented in Subsections 4.4.1 through 4.4.7. An alternative assumption regarding this pond is to treat it as a constant head area. A special simulation was conducted for this purpose, where the area of this pond was defined as a constant head area at 148.5 ft. msl. Figure 59 shows the simulated water levels under this assumption. As can be seen by comparing Figure 46 and Figure 59 treating the Upper Debris Basin 2 as a constant head area has practically no effect on the simulated water levels in the area of Units 3 & 4 and on the location of the groundwater divide. Table 11 gives the calibration statistics for this simulation. Figure 60 shows the simulated vs. observed water levels for the same simulation. Figure 61 shows the distribution of the estimated residuals for this run.

Table 11   Sensitivity of Calibration Statistics to Constant Head Boundary Condition at Upper Debris Basin 2

Condition at pond	Calibration Statistics						
	MR	ARM	SEE	RMS	NRMS	CC	M_d
	ft	ft		Ft	%		%
Constant head at el. 148.5 ft	-7.101	2.688	0.704	3.228	10.027	0.977	-0.02
No special condition set for the pond	-6.741	2.617	0.68	3.118	9.686	0.885	-0.02

4.4.8.4   Additional Areas of Zero Recharge

During the calibration runs, the parking lot at Units 1&2 and a number of small structures were not represented as paved areas. To assess the effect of these additional areas of zero recharge, the recharge zone map was modified and a sensitivity analysis conducted. Figure 62 shows the simulated water levels under this assumption. As can be seen by comparing Figure 46 and Figure 62, incorporating these additional paved areas has practically no effect on the simulated water levels in the area of Units 3 & 4 and on the location of the groundwater divide. Table 12 gives the calibration statistics for this simulation. Figure 63 shows the simulated vs. observed water levels for the same simulation. Figure 64 shows the distribution of the estimated residuals for this run.

Table 12   Sensitivity of Calibration Statistics to Additional Paved Areas (Parking Lot for Units 1&2 and a Number of Small Structures)

Recharge Zones	Calibration Statistics						
	MR	ARM	SEE	RMS	NRMS	CC	M_d
	ft	ft		Ft	%		%
Added parking lot at Units 1&2 and a number of small structures	-7.079	2.546	0.676	3.108	9.656	0.887	-0.02
Non-zero recharge applied to Units 1 and 2 parking lots and small structures	-6.741	2.617	0.68	3.118	9.686	0.885	-0.02

4.5   Validation of the Groundwater Model

4.5.1   Validation Using Stream Flow Data

To further establish the validity of the calibrated groundwater model, the groundwater discharge to the creek draining Mallard Pond was compared with stream flow measurements in the creek. Stream flow data had been collected in June and July 1985 in support of the construction of Units 1 & 2. The estimated flows at three points on the creek of Mallard Pond are given in Table 13. The locations of the stream flow measurements given in Table 13 are shown in Figure 65.

Table 13   Measured Stream Flows in the Stream of Mallard Pond in June-July 1985

Station	Location	Flow (gpm)	
		Measured	Estimated
2	Mallard Pond Drain	335	320
3	100 ft downstream from drain	220	346
5	300 ft downstream from drain	600	373

Model 7 was used to estimate the total groundwater discharge into Mallard Pond and the downstream reaches of the creek that are fed by Mallard Pond. Mallard Pond is represented by constant-head cells in the model, while the creek downstream of Mallard Pond is represented by drain cells (Figure 66). To estimate the groundwater discharge to different parts of the creek, the surrounding area was divided into three zones (Figure 67), which were used in a Zone Budget simulation. Zone 1 is defined by the constant-head cells representing Mallard Pond, Zone 2 is the drain cells between Stations 2 and 3 in Figure 65, and Zone 3 is the drain cells between Stations 3 and 5 in Figure 65.

The estimated cumulative groundwater discharges from the model into the creek are given in Table 13. These discharges represent the average base flow in the creek. As can be seen, the

agreement between these estimated and measured discharge is very good. With the exception of Station 3, the estimated groundwater discharges are lower than the stream flows, which should be expected because the stream flows may include some surface water runoff.

It is important to recognize the limitations of this comparison, which should be seen only as "an order of magnitude" confirmation of the predictions of the model. A detailed comparison of groundwater discharges with stream flows would require several stream flow measurements over a period of time to establish the central tendency and variability of the base flow in the stream. In addition there are limitations in the accuracy of the method used to estimate the stream flow, and it is difficult to assess what portion of the measured stream flow is attributable to groundwater discharge and what part to surface runoff.

4.5.2 Validation Using 1971 Groundwater Level Data Prior to Construction of Units 1 & 2

Another independent check was conducted by modifying the model to represent the site as it existed prior to the construction of Units 1 & 2. This model was then run to steady state and compared to groundwater levels measured in November 1971.

The main change to the setup of the model was the redistribution of recharge, particularly at the location of buildings and paved areas. These areas were changed from zero recharge zones to either that of open area on mild slopes or forested area on steep slopes, depending on the surrounding land usage. No effort was made redistribute the recharge based on pre-construction topography or landcover. It was assumed that the Met Tower pond was not present prior to Units 1 & 2, while Mallard Pond remained as a constant head boundary condition. Changes in the topography were not accounted for in this simulation. The November 1971 water levels used were obtained from Drawing No. AX6DD239 of the FSAR (Ref. 5) and are presented in Table 14.

Table 14   Groundwater Levels Prior to Construction of Units 1 & 2

Well	Plant		State		Elev (ft msl)
	Northing (ft)	**Easting (ft)**	**Northing (ft)**	**Easting (ft)**	
42D	8,403	9,571	1,143,403	623,571	157
124	6,896	9,527	1,141,896	623,527	162
129	8,856	9,576	1,143,856	623,576	153
140	7,846	8,702	1,142,846	622,702	159
141	7,860	8,293	1,142,860	622,293	156
142	8,283	8,262	1,143,282	622,260	153
143	8,283	8,738	1,143,283	622,738	153
176	7,117	11,423	1,142,117	625,423	159
177	8,560	10,865	1,143,560	624,865	161
178	9,958	8,994	1,144,958	622,994	158
179	9,059	7,779	1,144,061	621,779	157

Figure 68 shows the simulated head for this model. Figure 69 shows the simulated vs. observed water levels for the same simulation. Figure 70 shows the distribution of the estimated residuals for this run. Perusal of these figures demonstrates that the model fairly represents the 1971 groundwater level data, especially considering the gross assumptions made regarding the pre-construction recharge. It is expected that a better fit of the modeled vs. observed water levels could be obtained through a more accurate representation of the pre-construction topography and spatial distribution of recharge. However, this additional validation step was not deemed necessary as there would be no effect on the post-construction simulations for Units 3 & 4.

5.   POST-CONSTRUCTION SIMULATIONS

For the construction of Units 3 & 4 the existing site must be graded to create a flat pad for the planned footprint of the new units. Units 3 & 4 will have a finished grade level elevation of approximately 220 ft msl. The bottom of the foundation slab for the safety-related containment buildings will be at elevation 180.5 ft msl. During the construction of the new units the site will be excavated to remove the in-situ soil down to the principal bearing strata, the Blue Bluff Marl. The elevation of the top of the Blue Bluff Marl at the site ranges from 120 ft to 140 ft msl. The in-situ soil will be replaced with Category 1 and 2 structural fill material. Foundations for the new units will be poured on this new fill material.

5.1   Post-Construction Groundwater Simulations

Groundwater flow simulations for post-construction conditions were performed with the best calibrated model, which was simulation 708 from Model 7. For the simulation of post-construction conditions three modifications were made to this model:

a. The topography used in the model was modified to reflect the final grading of the site after the completion of the construction of Units 3 & 4.

b. A new hydraulic conductivity zone was introduced to describe the backfill material in the area around the power block of Units 3 & 4.

c. The rate of groundwater recharge in the area affected by the construction of units 3 & 4 was changed to reflect post-construction conditions.

The hydraulic conductivity of the fill material for the construction of Units 3 & 4 is assumed to be the same as that of the fill material used for Units 1 & 2. As discussed in Section 2.7, the geometric mean of four slug tests conducted in the structural fill material for Units 1 & 2 was 2.3 ft/day. The hydraulic conductivity values from these tests ranged from 1.3 to 3.3 ft/day. As a conservative assumption it is assumed that the hydraulic conductivity of the fill material is equal to the maximum measured value, i.e. 3.3 ft/day. Figure 71 shows the hydraulic conductivity zones used in the post-construction simulations. This is the same as Figure 45 with the addition of a new zone for the backfill material in the area of the power block of Units 1 & 2 and Units 3 & 4.

The hydraulic conductivity of the native materials and other parameters used in the model were kept equal to the values that produced the best agreement between computed and observed water levels (see Section 4.4). Figure 72 illustrates the model grid overlaid on the excavation plan. It can be seen from this figure that the smallest footprint of the excavation (at the base) was used to define the areas of backfill. Using this footprint enables a conservative determination of the travel time. The reason behind this is because the backfill has the lowest hydraulic conductivity of all materials between the release point and its discharge point, and hence travel through the backfill will dominate the total travel time.

Figure 73 shows the recharge zones used to simulate post-construction conditions. Over most of the model domain these zones are the same as those used for the calibration of the model under present conditions (see Figure 22), but differ at the site of Units 3 & 4, where zero recharge was defined in all paved areas or areas of buildings, and where local changes in vegetation cover and ground surface slope due to grading are expected. Figure 74 shows the simulated water levels obtained under these conditions. Figure 75 shows the simulated vs. observed water levels for the same simulation. Figure 76 shows the distribution of the estimated residuals for this run.

5.1.1   Release of Particles From Circle of Radius of 775ft

To assess the pathways of potential effluent releases particle tracking was performed with the model using 30 particles evenly distributed along the periphery of a 775-ft radius circle centered between Units and 3 & 4 and encompassing the entire power block area (see Figure 77). The 775-ft radius circle encompassing the power block used is that shown in SSAR Figure 1-4. The coordinates of the center of this circle are: Easting 621,446 and Northing 1,142,882 (in the State Plane Grid coordinate system). Results of the simulation show that all particles on the circle, which originate from the power block area, discharge to Mallard Pond as illustrated in Figure 77.

5.1.2   Particle Release From Auxiliary Building of Unit 4

To supplement the particle tracking exercise, an additional model run was conducted with the release of a single particle from underneath the auxiliary building of Unit 4. The particle was released at the location where it had the shortest pathway to travel through the fill. The purpose of this was to estimate a conservative travel time through the low hydraulic conductivity fill. The purpose of this model run was to determine the travel time of the particle through each of the subsurface materials between Unit 4 and its discharge point at Mallard Pond. The results of this model run demonstrate that the total travel time for the particle is 6.7 years, with the travel time in each material presented in Table 15. The pathway of the particle to its discharge point at Mallard Pond is illustrated in Figure 78.

Table 15   Travel Time for Single Particle Released from Auxiliary Building of Unit 4

Travel Time (years)			
Fill (3.3 ft/day)	Barnwell Sands/Utley Limestone (32 ft/day)	High Conductivity Zone above Mallard Pond (100 ft/day)	Total Pathway
2.4	3.2	1.1	6.7

6. CONCLUSIONS

A two-dimensional, horizontal plane model was developed to simulate groundwater flow under present and post-construction conditions at the VEGP site. The model was developed using all available historic data and data collected in support of the ESP/COL application.

Several alternative plausible conceptual models for the distribution of the hydraulic properties of the native materials were evaluated and tested during the calibration of the model. The calibration process is presented in seven steps, each of which is referred as a different "model." The final calibration of the model met all calibration criteria. All solutions obtained with the model and presented in this report had less than one percent mass balance discrepancy between inflow and outflow.

The calibrated model was used to simulate post-construction conditions, accounting for changes in the topography at the site of Units 3 & 4, the presence of backfill material in the area of the new structures, and changes in ground water recharge. Particle tracking was performed to identify the groundwater pathways of potential liquid effluent releases from the power block area.

Special attention was paid to the location of the groundwater divide in the vicinity of the power block. The models that produced the best agreement with the measured data have the groundwater divide south of the cooling towers.

All simulations with different alternative models suggest that the groundwater divide is to the south of the power block area of Units 3 & 4 and that any effluent releases to the groundwater from within the power block area will move northward and end up in Mallard Pond.

7. REFERENCES

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9 Waterloo Hydrogeologic Inc., <u>Visual MODFLOW Professional v.4.2, User's Manual</u>, 2006.

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11 Drawings:
23162-000-EZ-3610-00001 00C, Units 3 & 4 - 500KV and 230KV Conceptual Transmission Lines, 11/2/2007, SV0-0000-E2-2005 V1.0
23162-000-P1-0010-00001 00E, Conceptual Plot Plan, 9/28/2007, SV0 0000 X2 2001 V5.0
23162-000-P1-0010-00002 00E, Conceptual General Arrangement, 9/28/2007, SV0 0000 X2 2001 V5.0

12 LIDAR Survey, Metro Engineering and Surveying Company. Drawing Number M-197, Dated 8/11/06.

13 Burke County, USGS Digital Elevation Map

Figure 1:   VEGP Site Location [Figure 2.5.1-1]
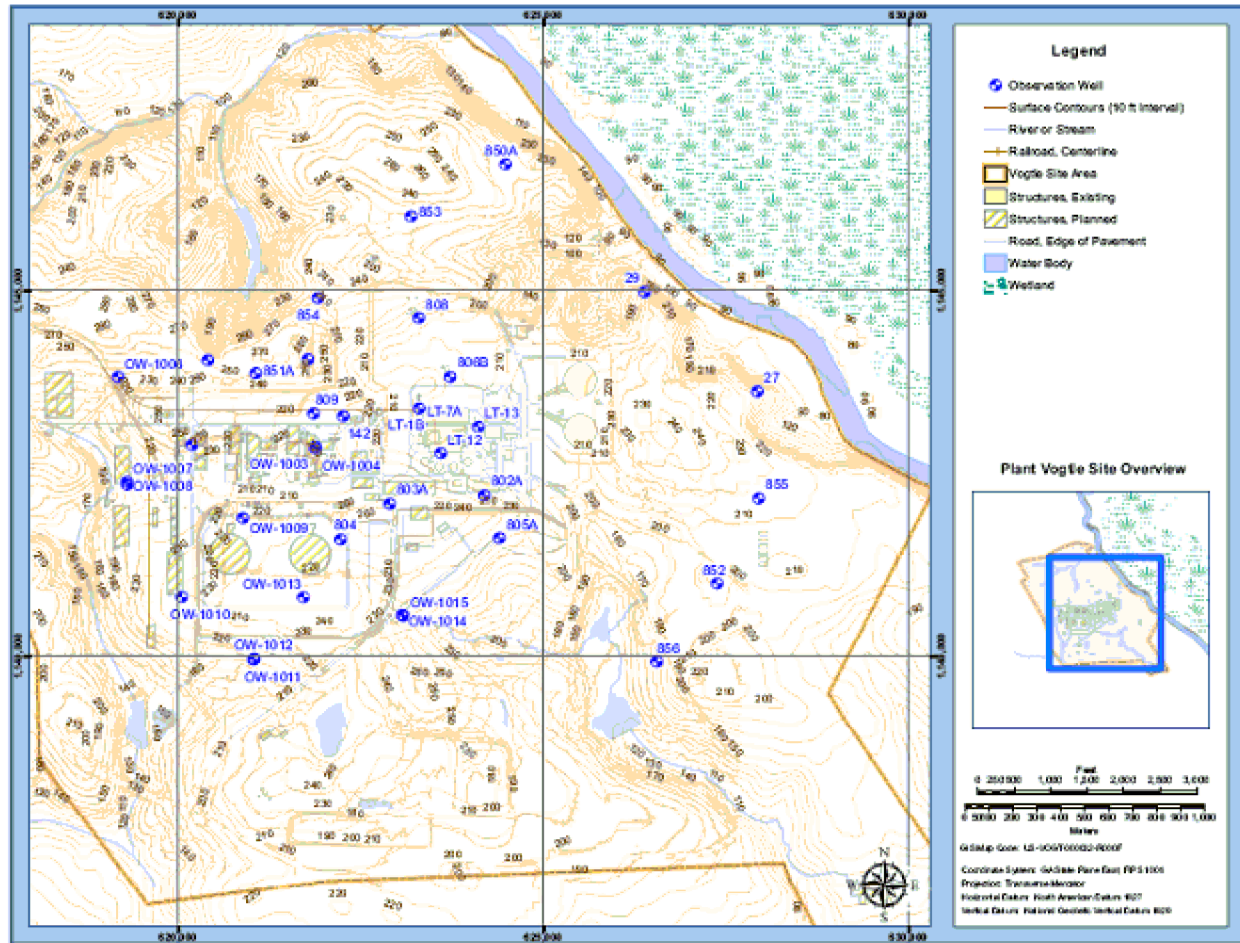
Figure 2:   Site Map Showing the Existing and the Proposed VEGP Units [Figure 2.4.12-3]
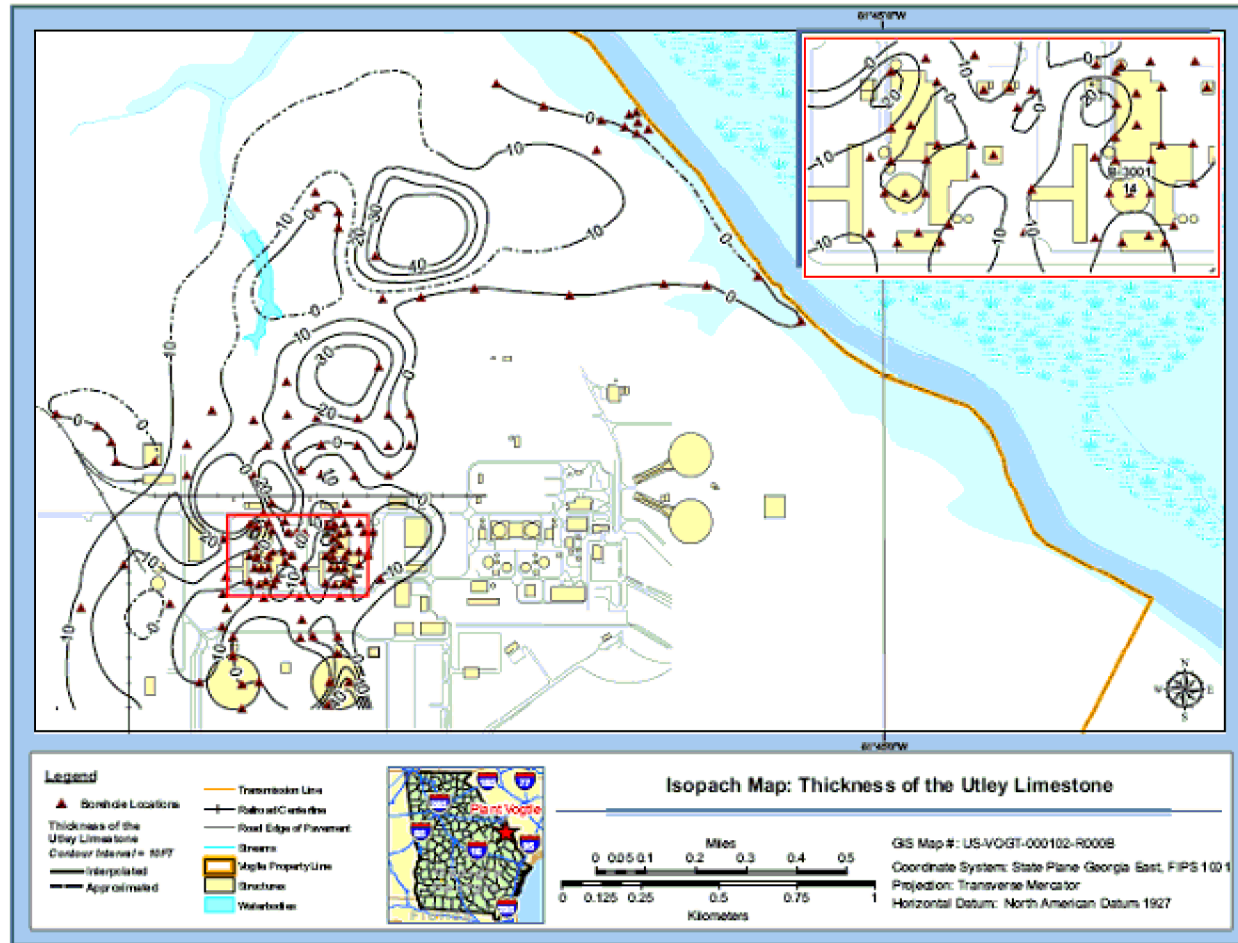
Figure 3: Isopachs of the Utley Limestone [Ref. 5, Figure 2.5.1-53]
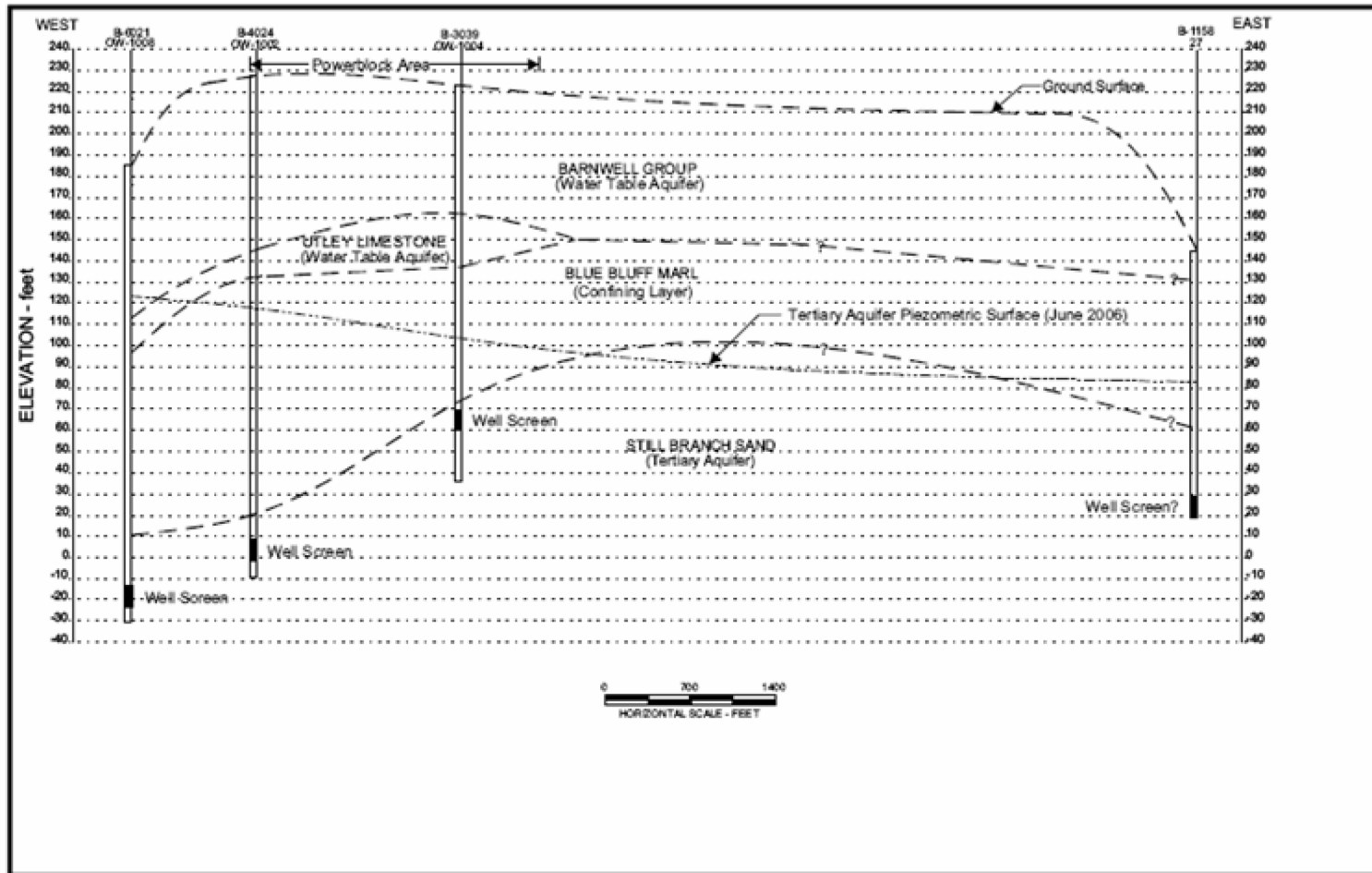
Figure 4: Hydrogeologic Cross-Section of the Water Table Aquifer at the VEGP Site [Ref. 5 Figure 2.4.12-2A]
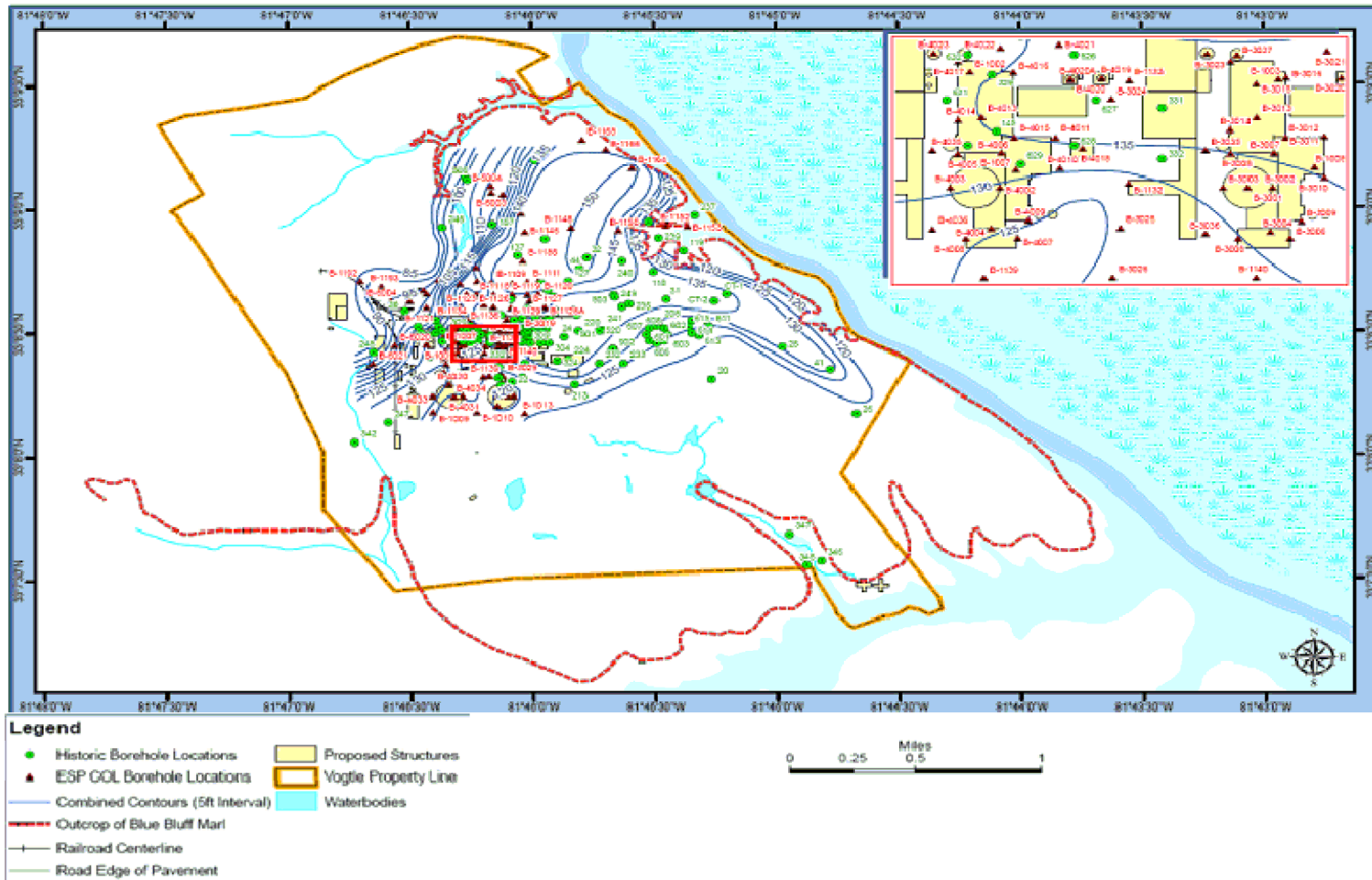
Figure 5:   Outcrop and Contours of the Top of the Blue Bluff Marl at the VEGP Site

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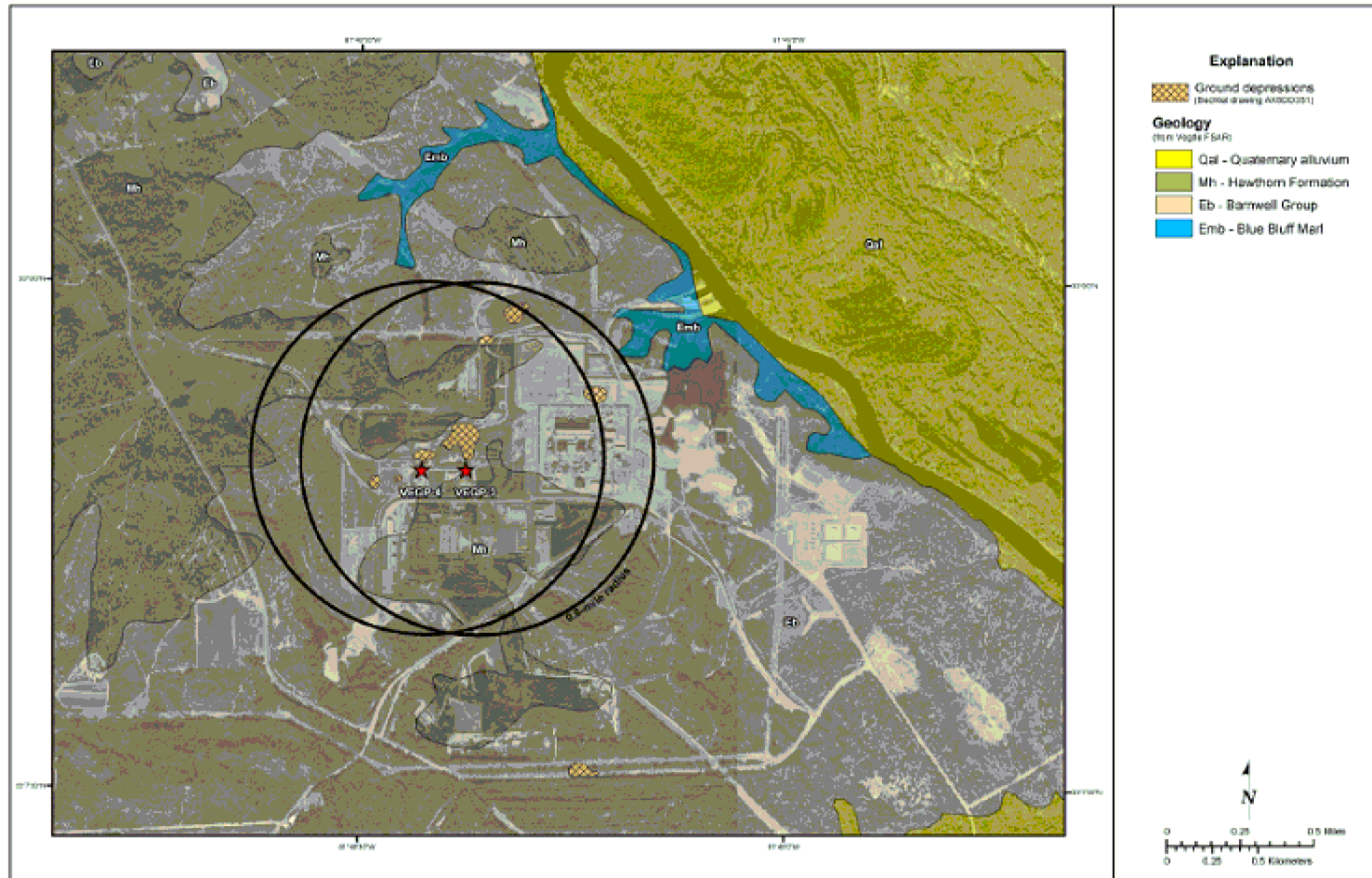
Figure 6: Site Geologic Map Showing the Outcrop of the Blue Bluff Marl [Figure 2.5.1-31]
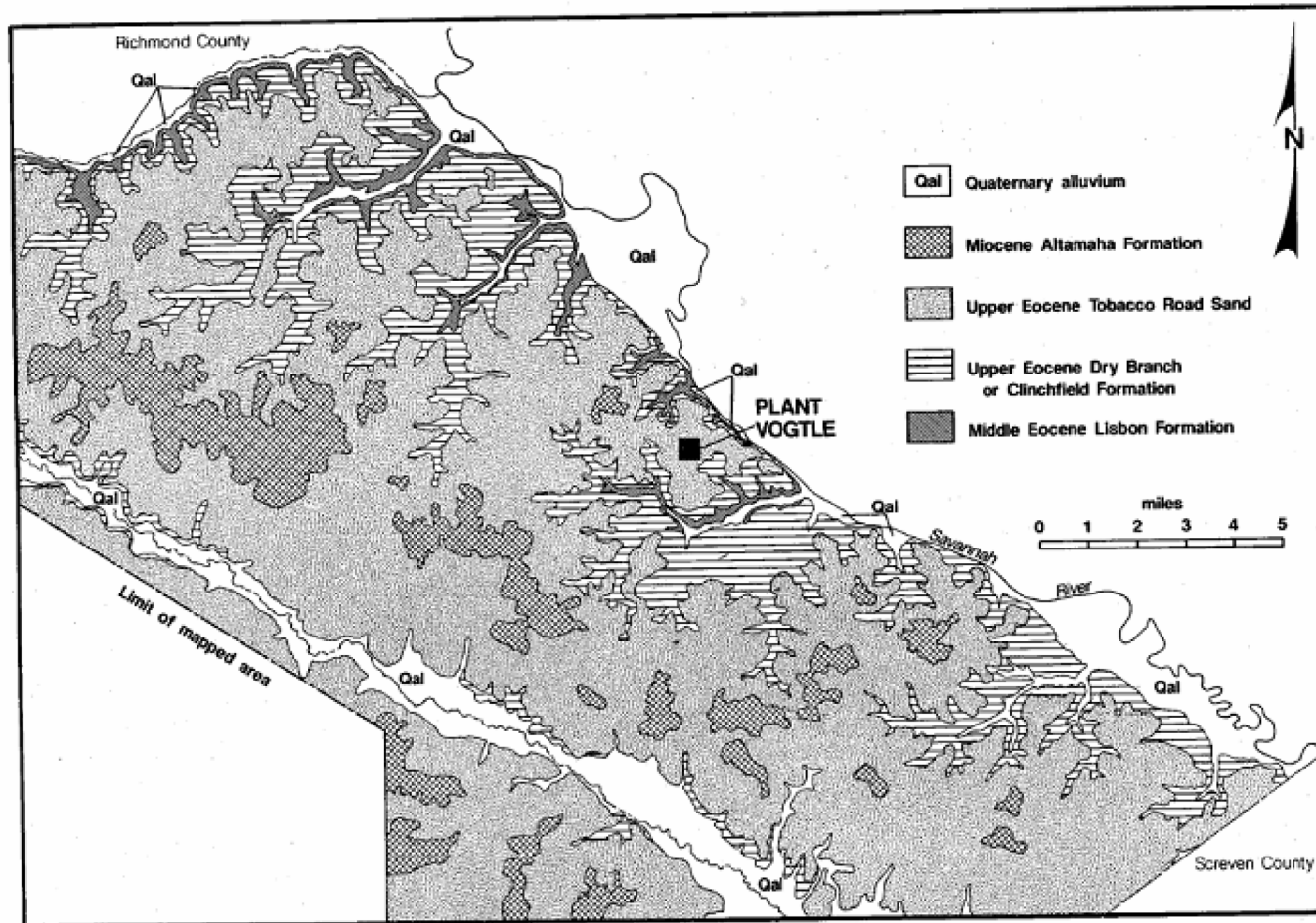
Figure 7: Geologic Map of Barnwell Group Exposures in Eastern Burke County [from Ref. 6]
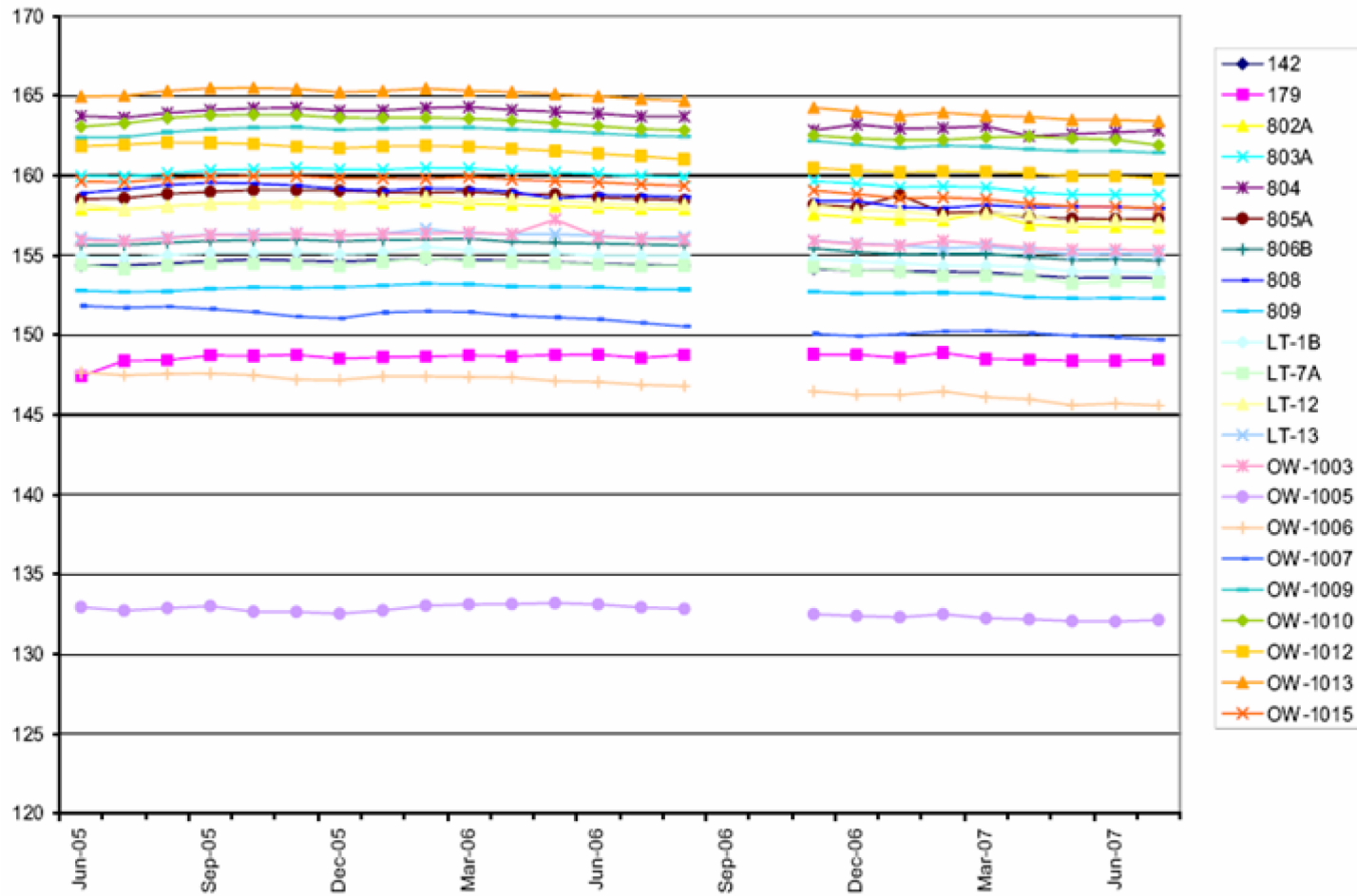
Figure 8:   Water Table Aquifer: June 2005 – July 2007 Hydrographs [Ref. 5, Figure 2.4.12-23]
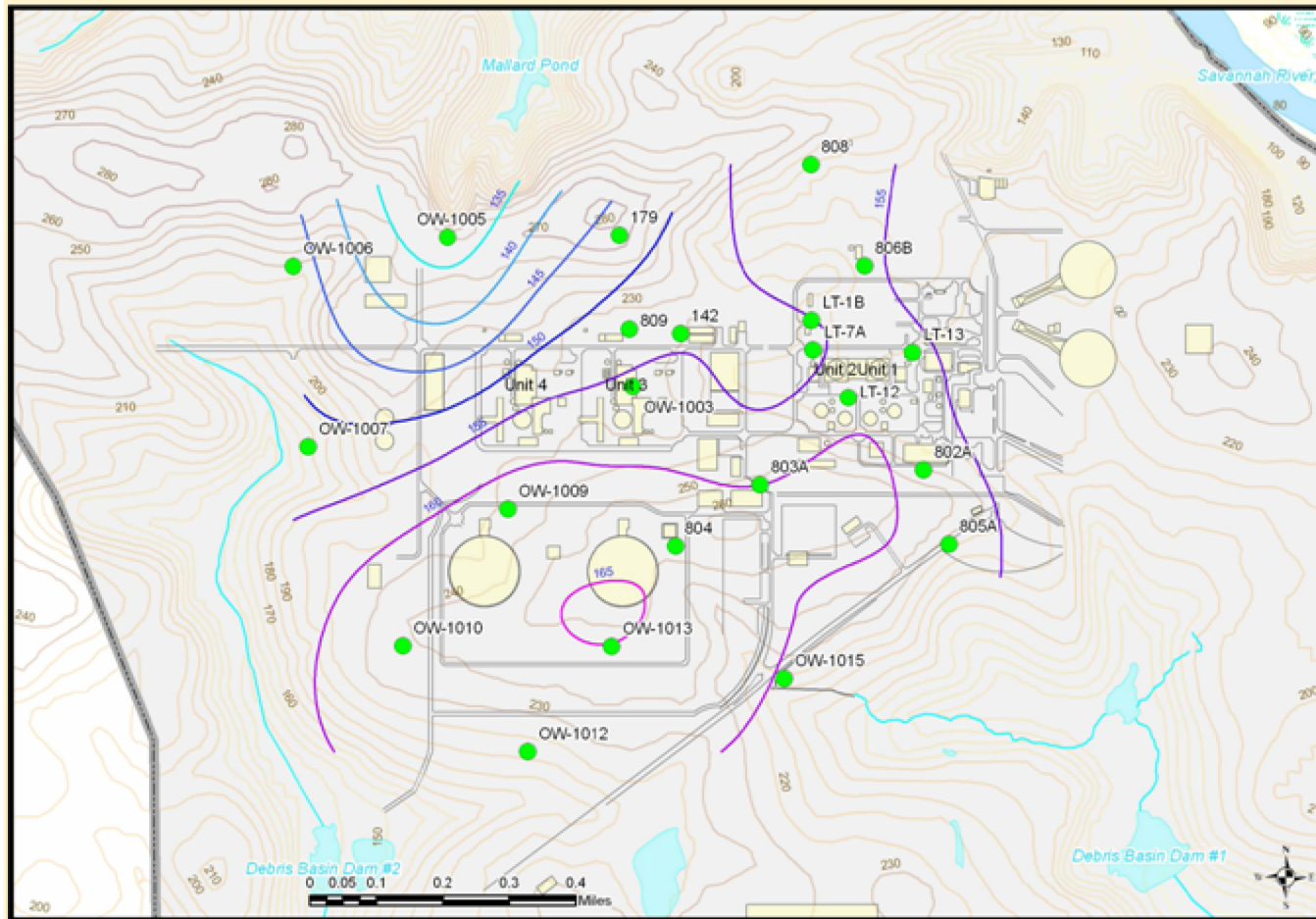
Figure 9: Water Table Aquifer: Piezometric Contour Map for June 2005 [Ref. 5, Figure 2.4.12-7]
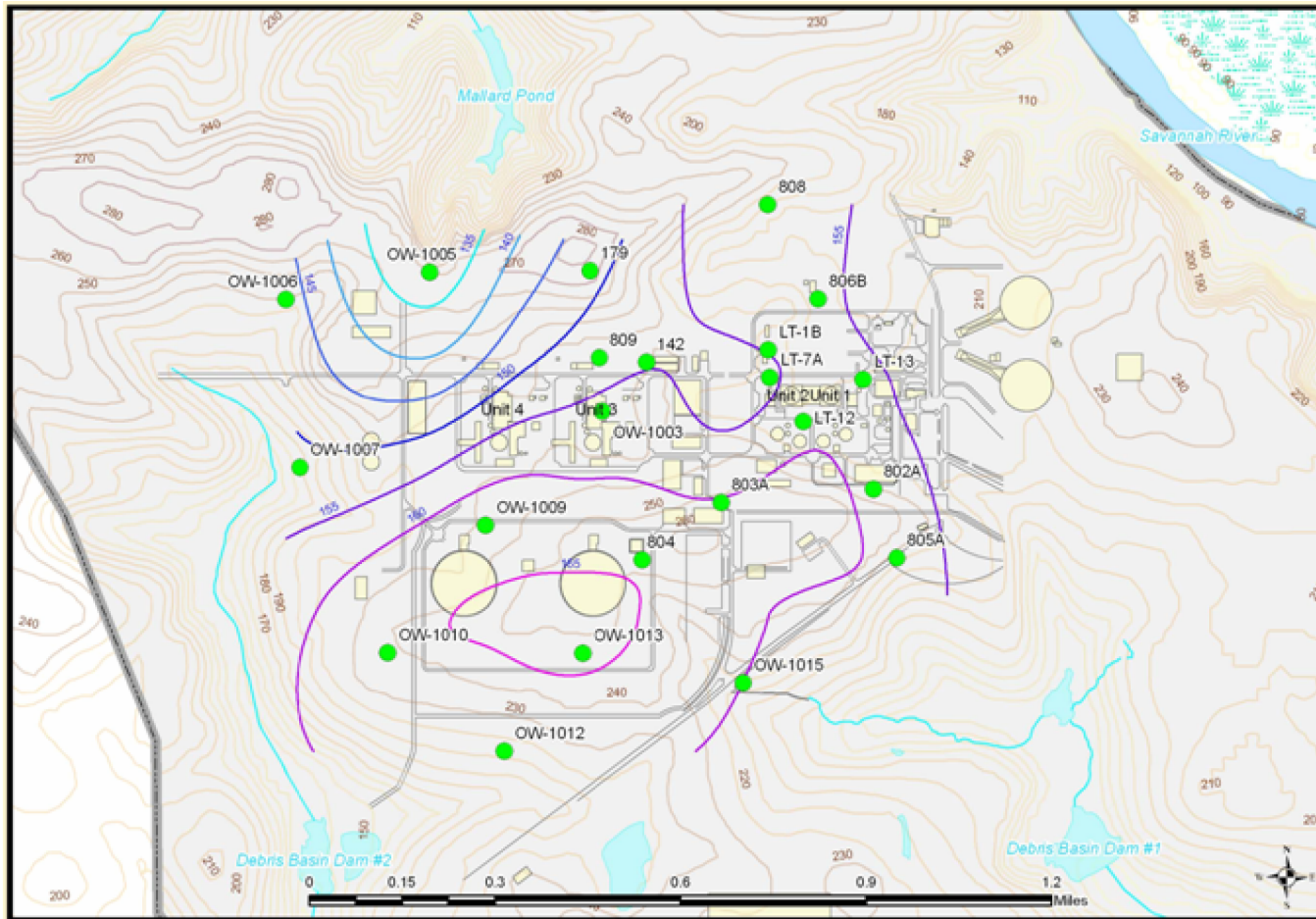
Figure 10: Water Table Aquifer: Piezometric Contour Map for October 2005 [Ref. 5, Figure 2.4.12-8]
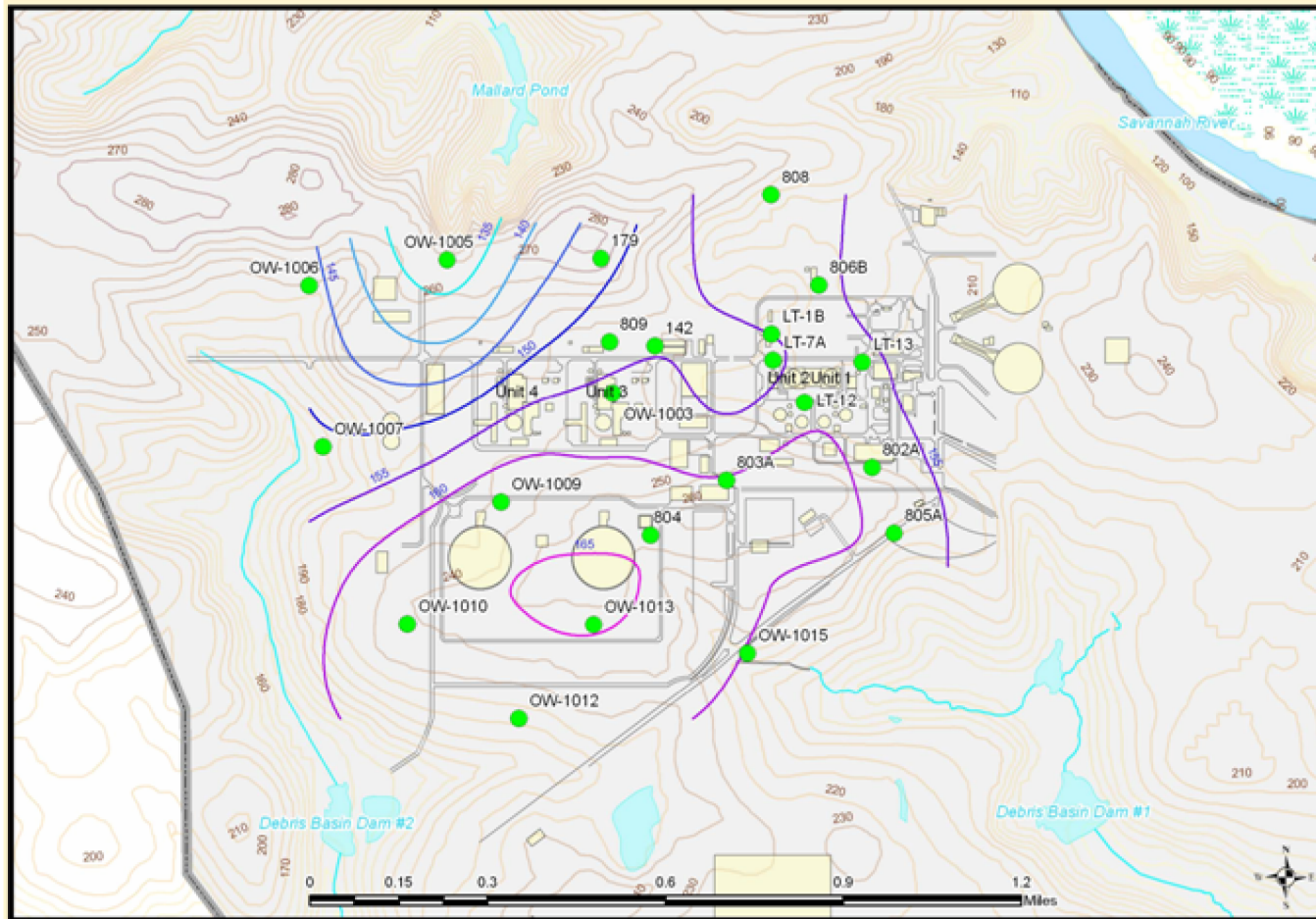
Figure 11: Water Table Aquifer: Piezometric Contour Map for December 2005 [Ref. 5, Figure 2.4.12-9]
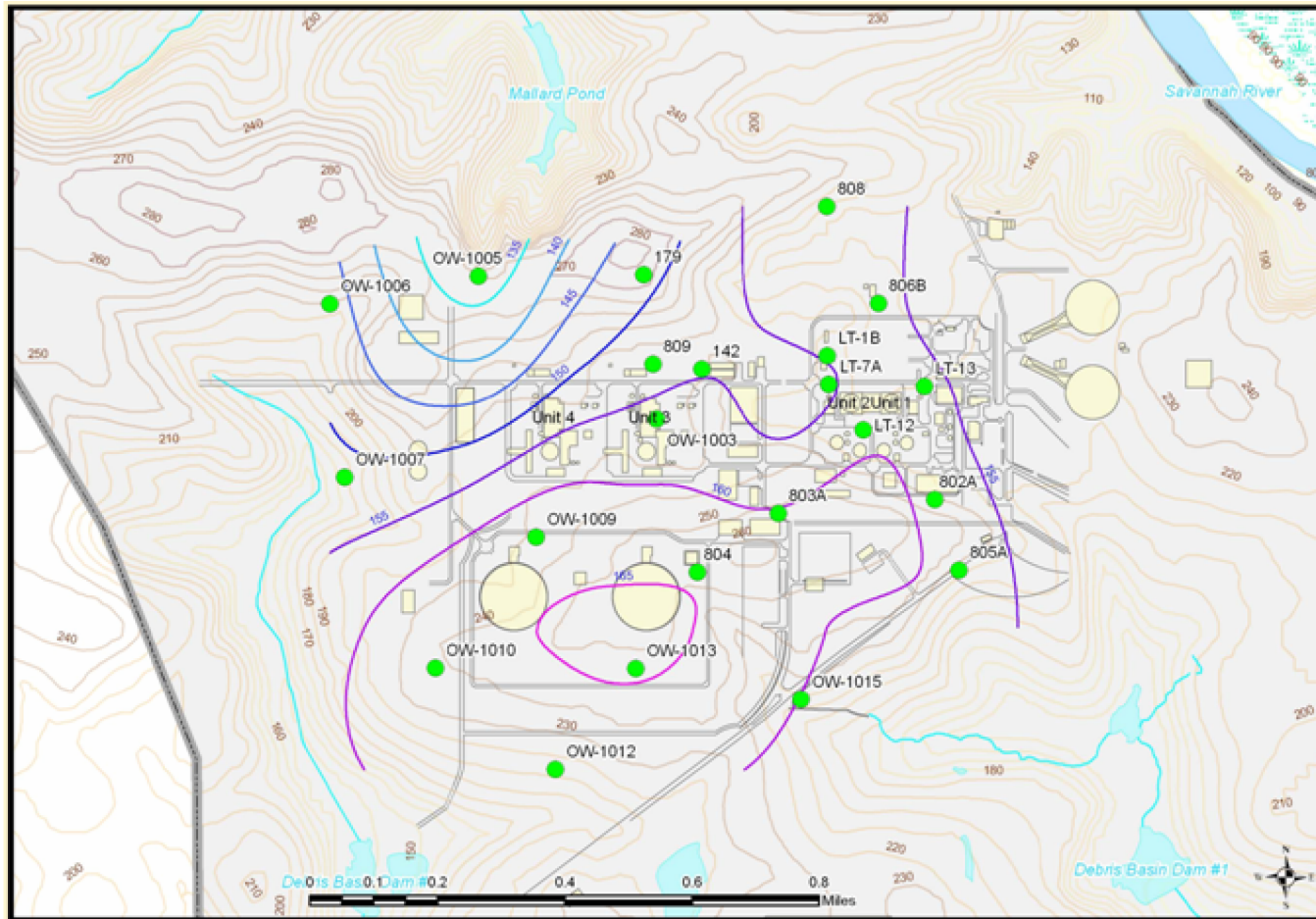
Figure 12: Water Table Aquifer: Piezometric Contour Map for March 2006 [Ref. 5, Figure 2.4.12-10]
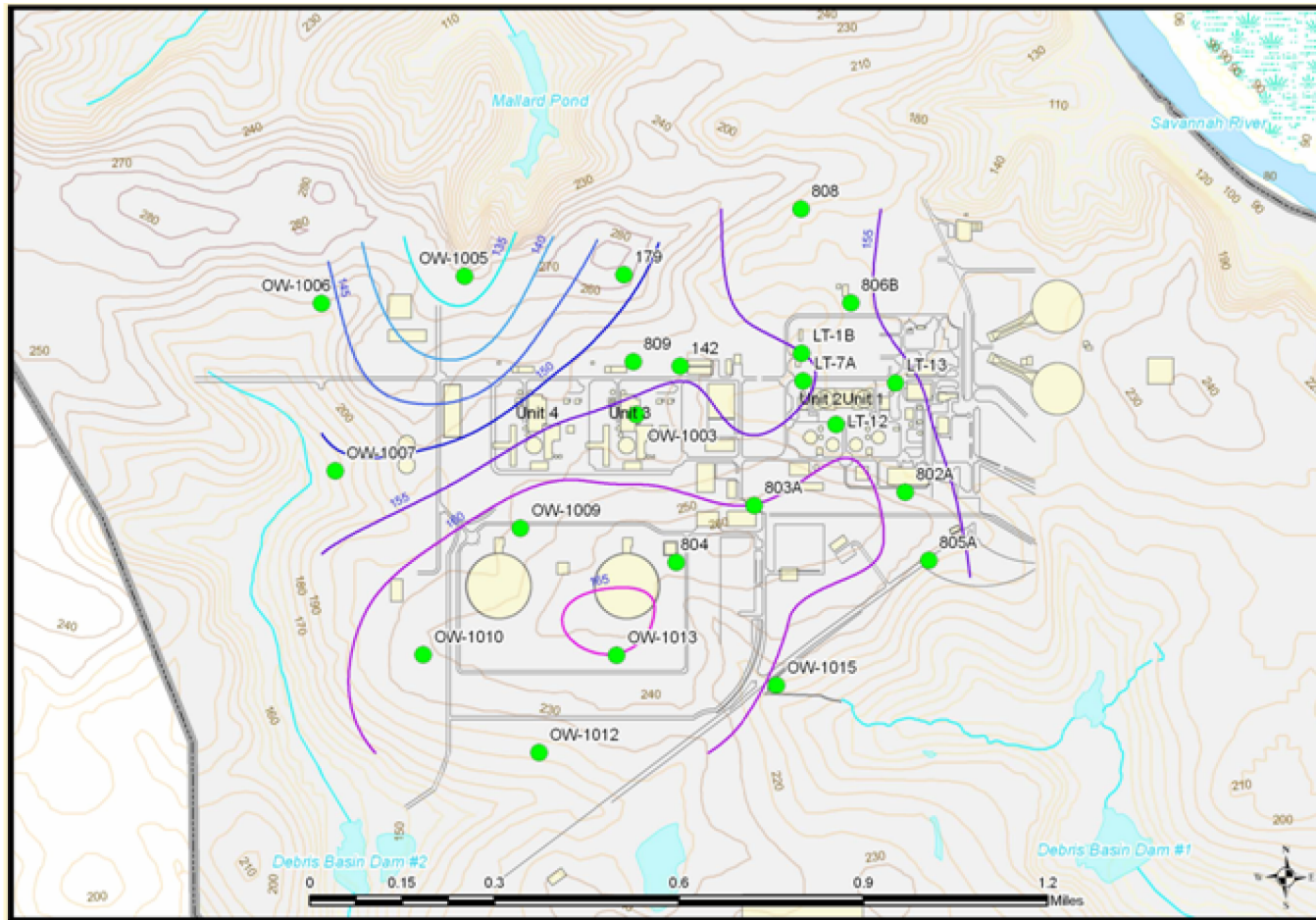
Figure 13:   Water Table Aquifer: Piezometric Contour Map for June 2006 [Ref. 5, Figure 2.4.12-11]
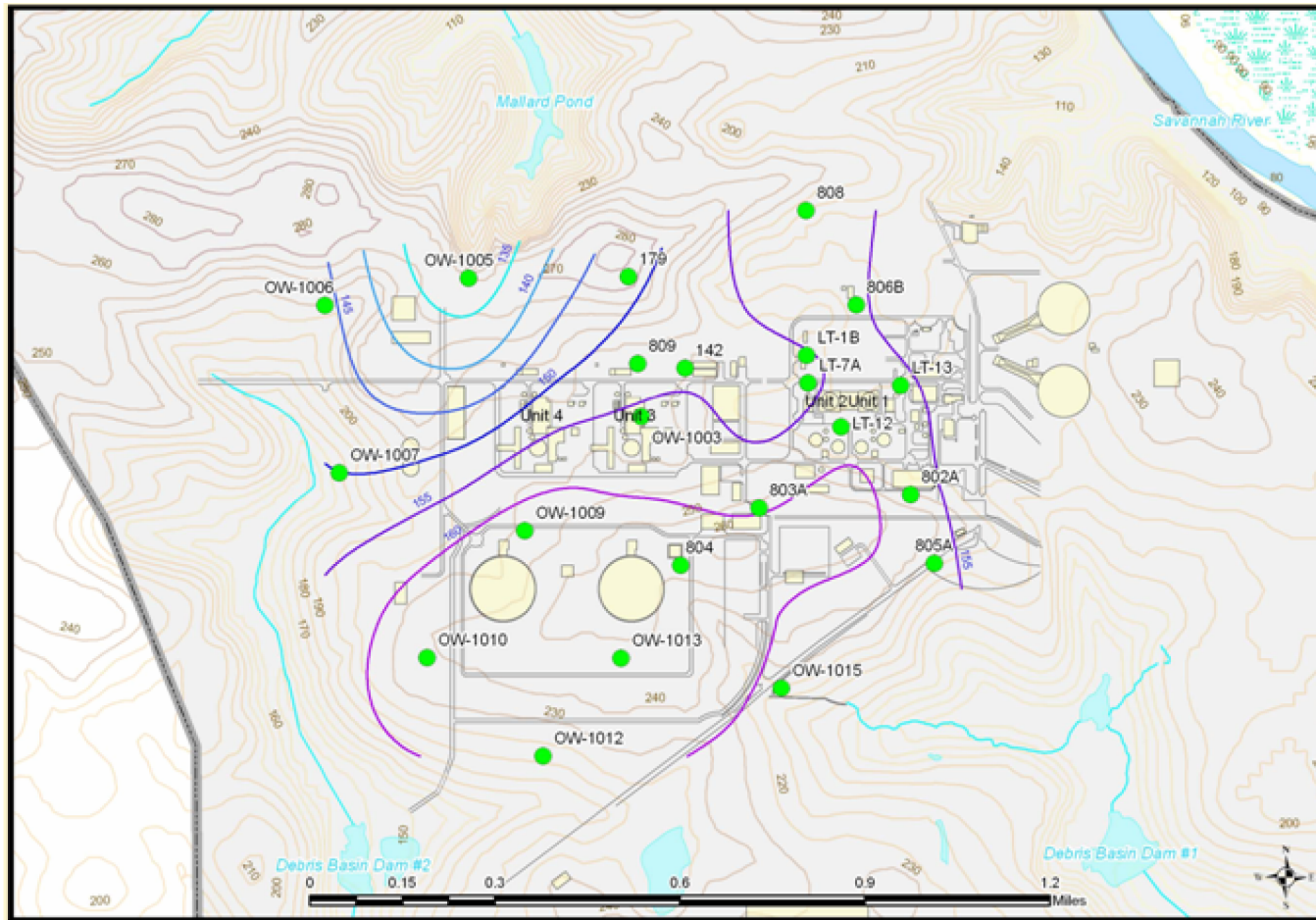
Figure 14: Water Table Aquifer: Piezometric Contour Map for November 2006 [Ref. 5, Figure 2.4.12-24]
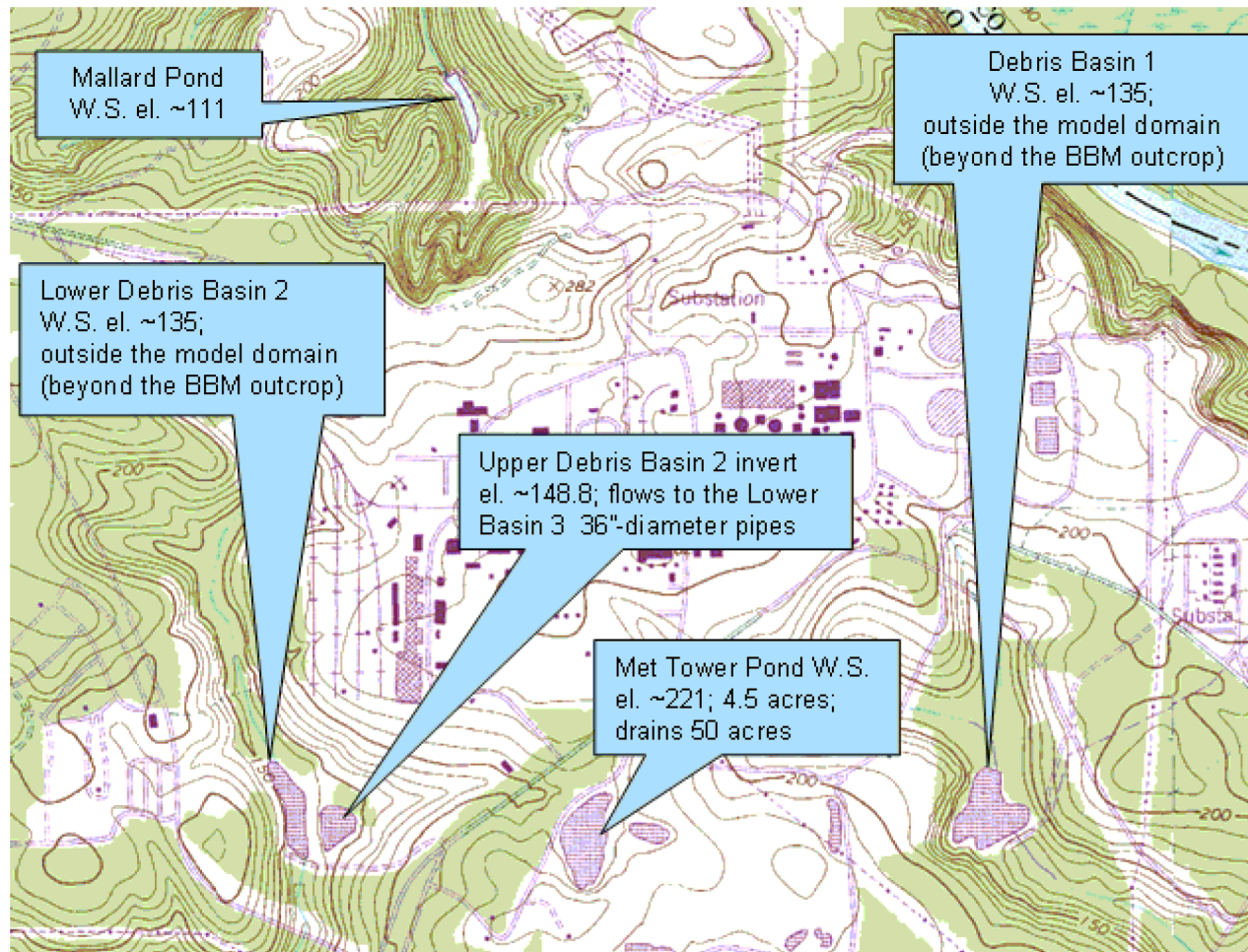
Figure 15:   Ponds Near the Site of Units 3 & 4
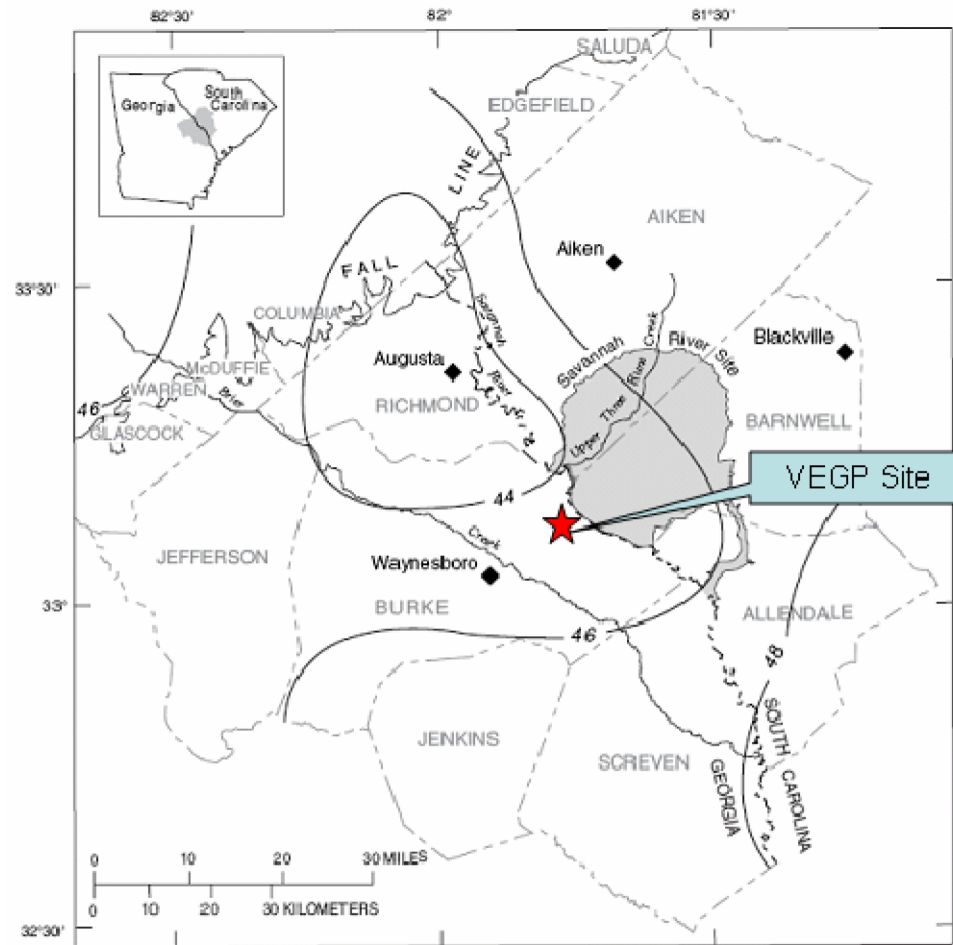
Figure 16:   Mean Annual Rainfall in the Savannah River Basin Based on Data from 1941 to 1970 [from Ref. 4]
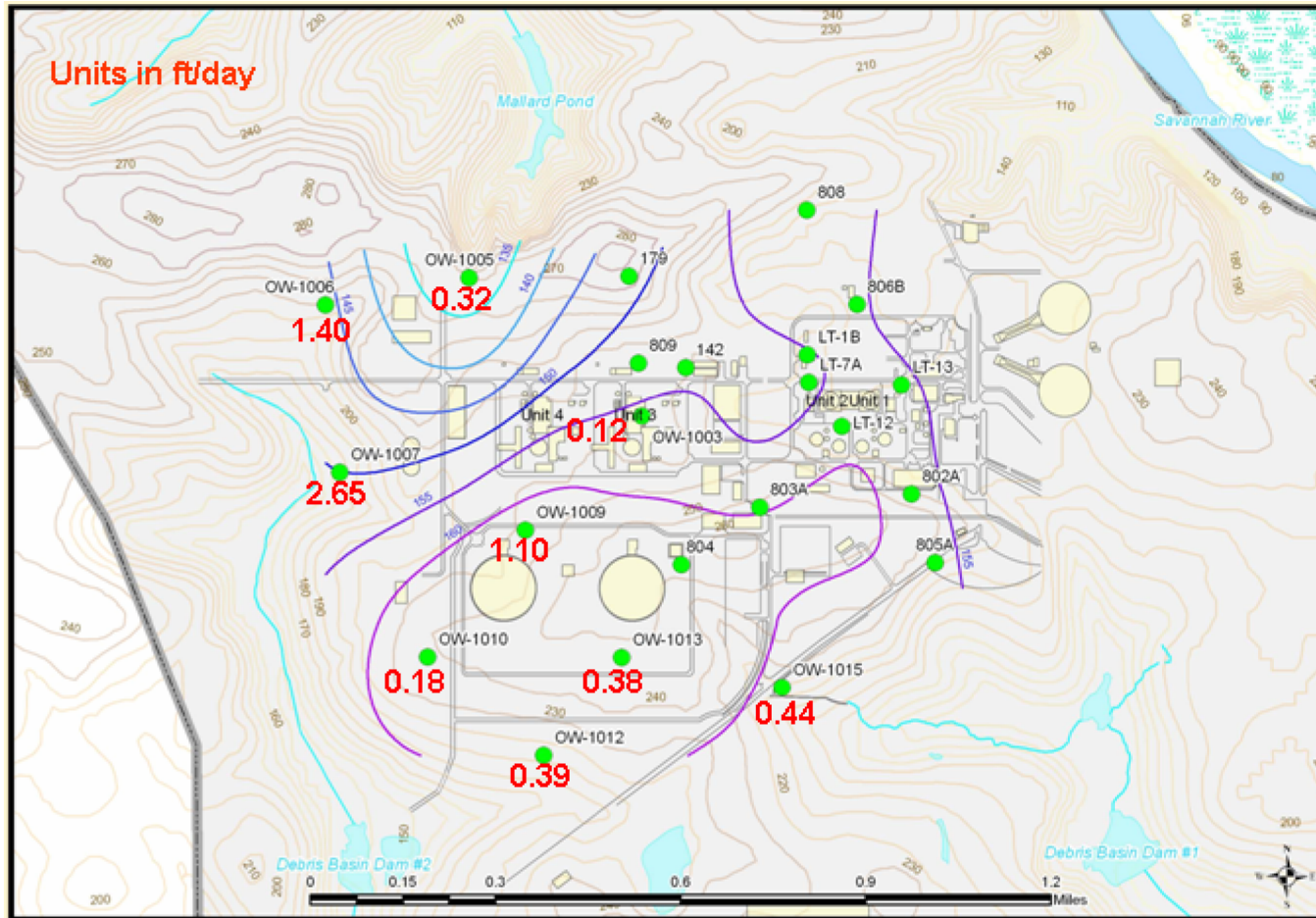
Figure 17: Distribution of Estimated Hydraulic Conductivity Values for Units 3 & 4

Outcrop of the blue bluff marl (bottom of the water table aquifer). The water table aquifer ends here. Groundwater discharges at the face of the bluff. The model cells along the bluff are treated as drains.

No flow boundary along the watershed boundary (surface water divide). It is assumed that the groundwater divide coincides with the surface water divide.

Outcrop of the blue bluff marl (bottom of the water table aquifer). The water table aquifer ends here. The model cells along the bluff are treated as drains.

Drains
No flow boundary

Figure 18:   Conceptual Groundwater Model for the VEGP Site
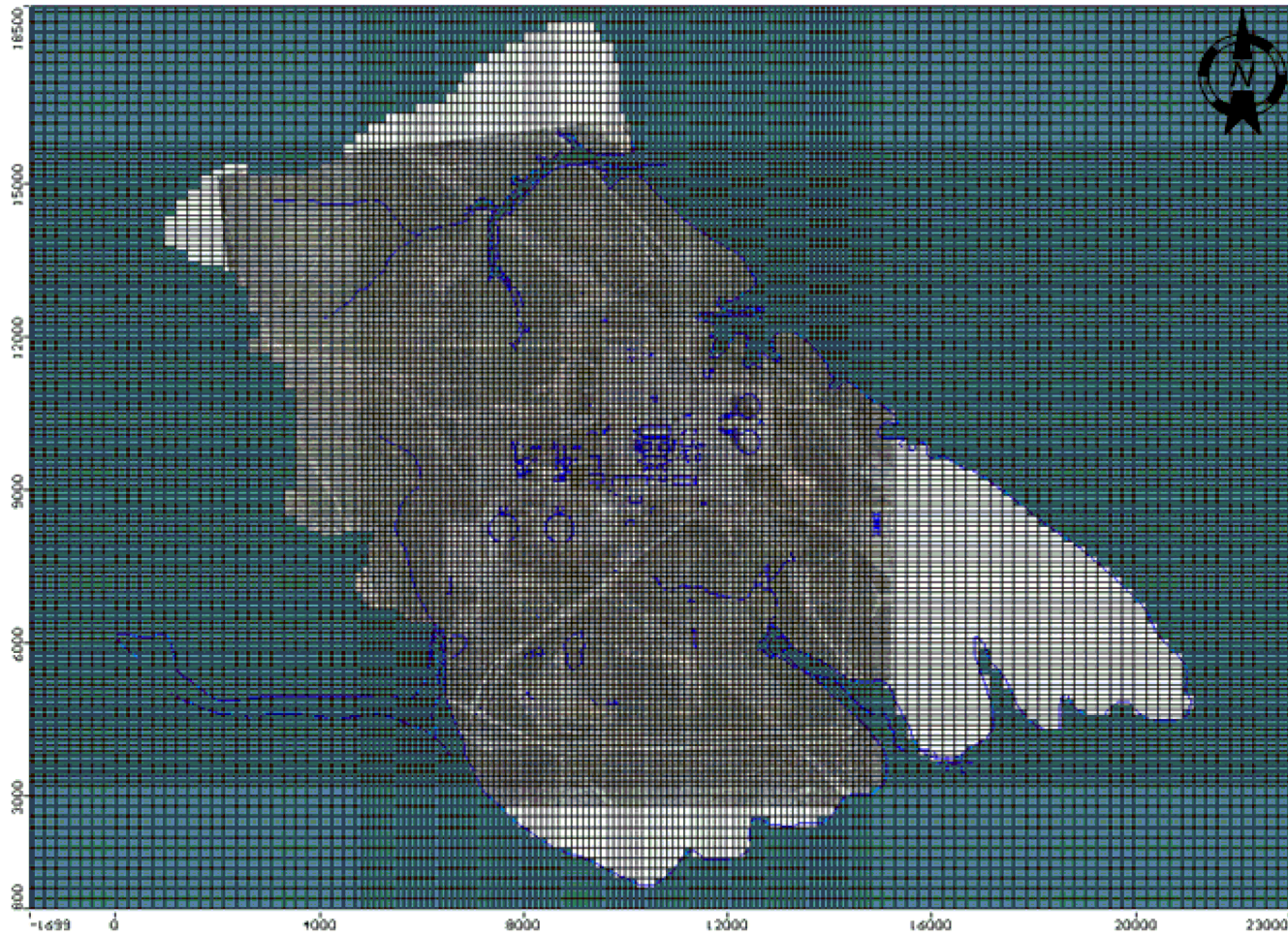
Figure 19:   Numerical Model Grid and Boundary Conditions
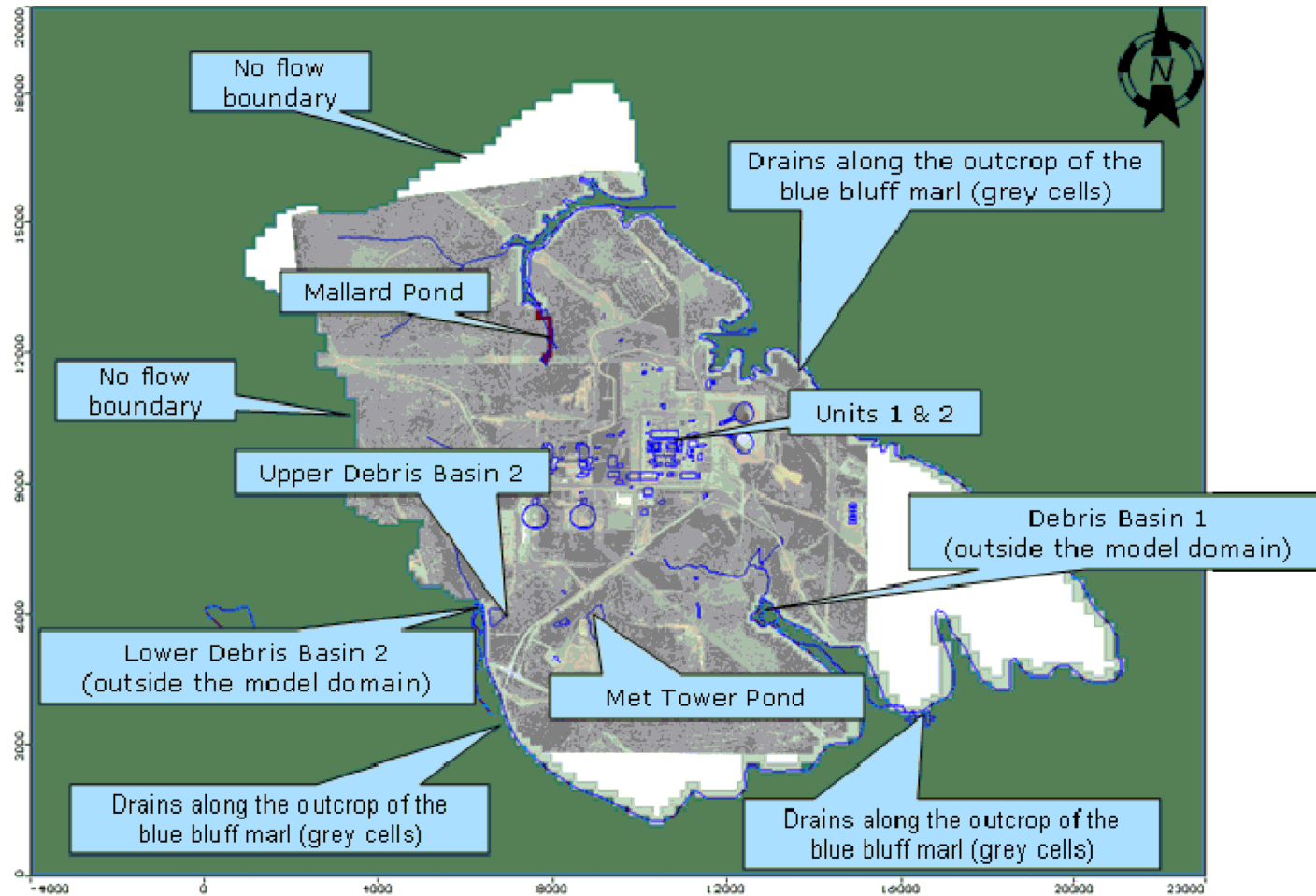
Figure 20:   Numerical Groundwater Model Domain with Key Physical Features and Respective Flow Boundaries
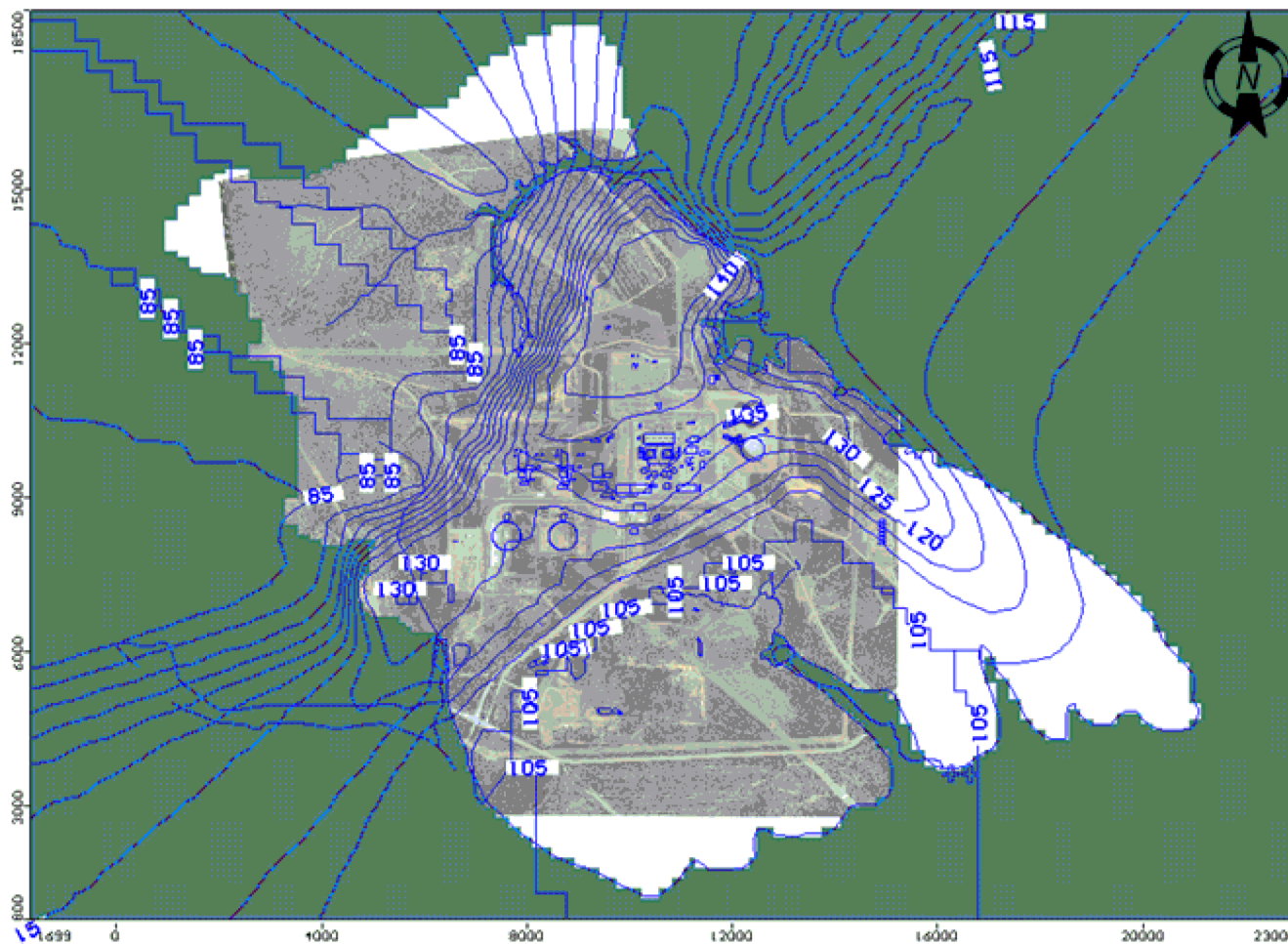
Figure 21:   Bottom of the Model Domain (Top of Blue Bluff Marl) as Described in the Model
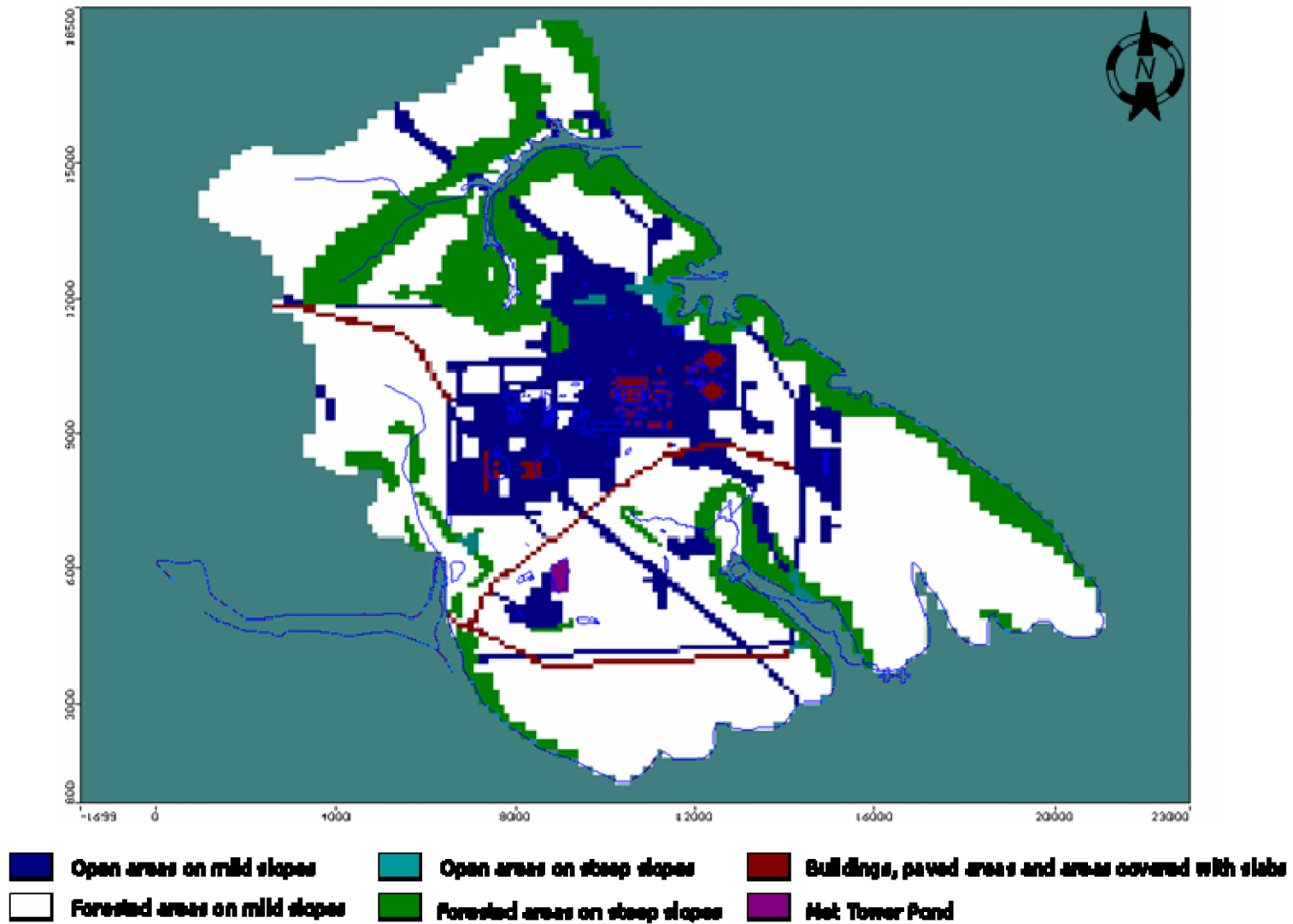
Figure 22:   Groundwater Recharge Zones Used in the Model
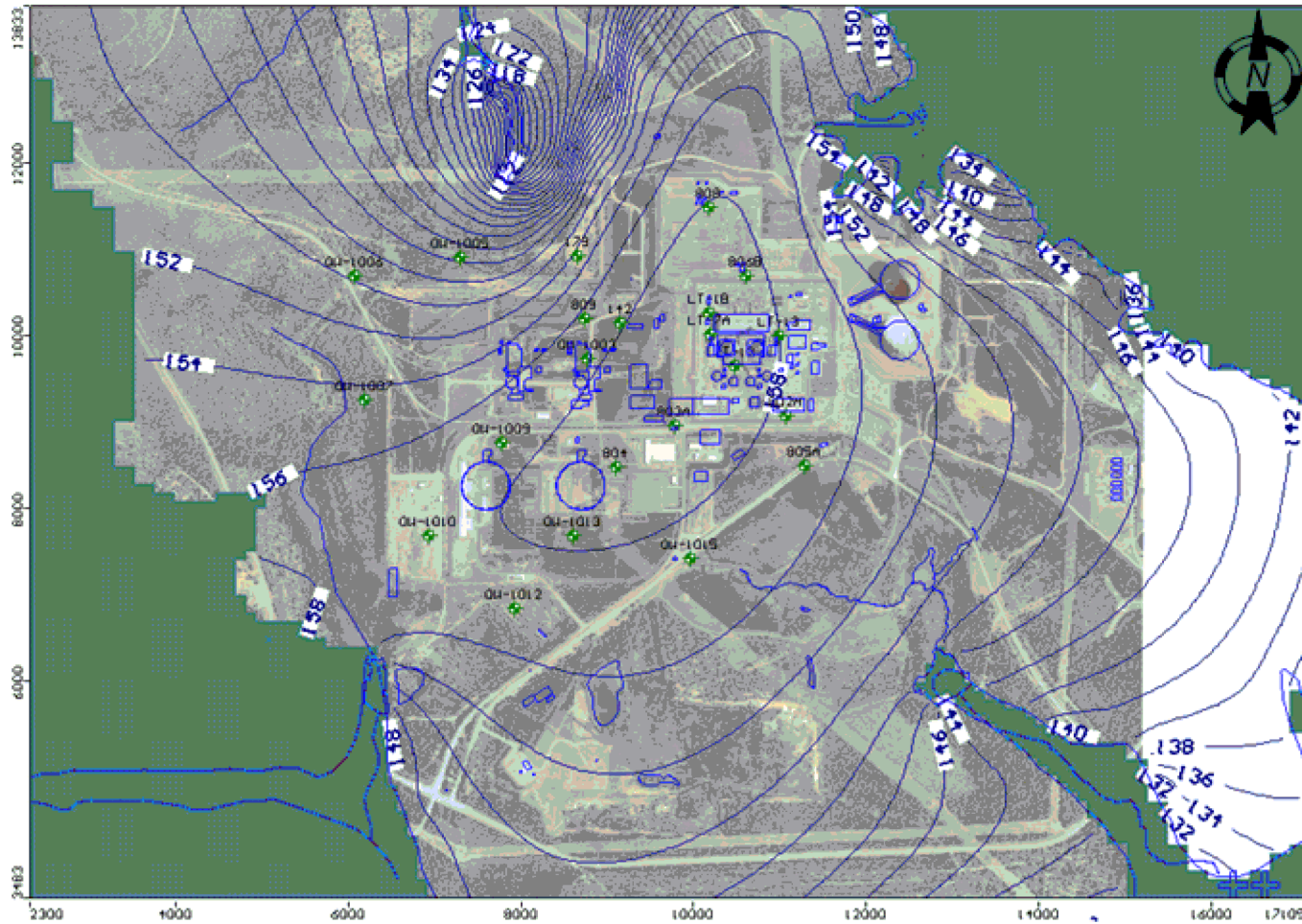
Figure 23: Model 1 - Simulated Water Levels for Run 109
(K_1 = 27 ft/day; R_1 =7 in/yr)
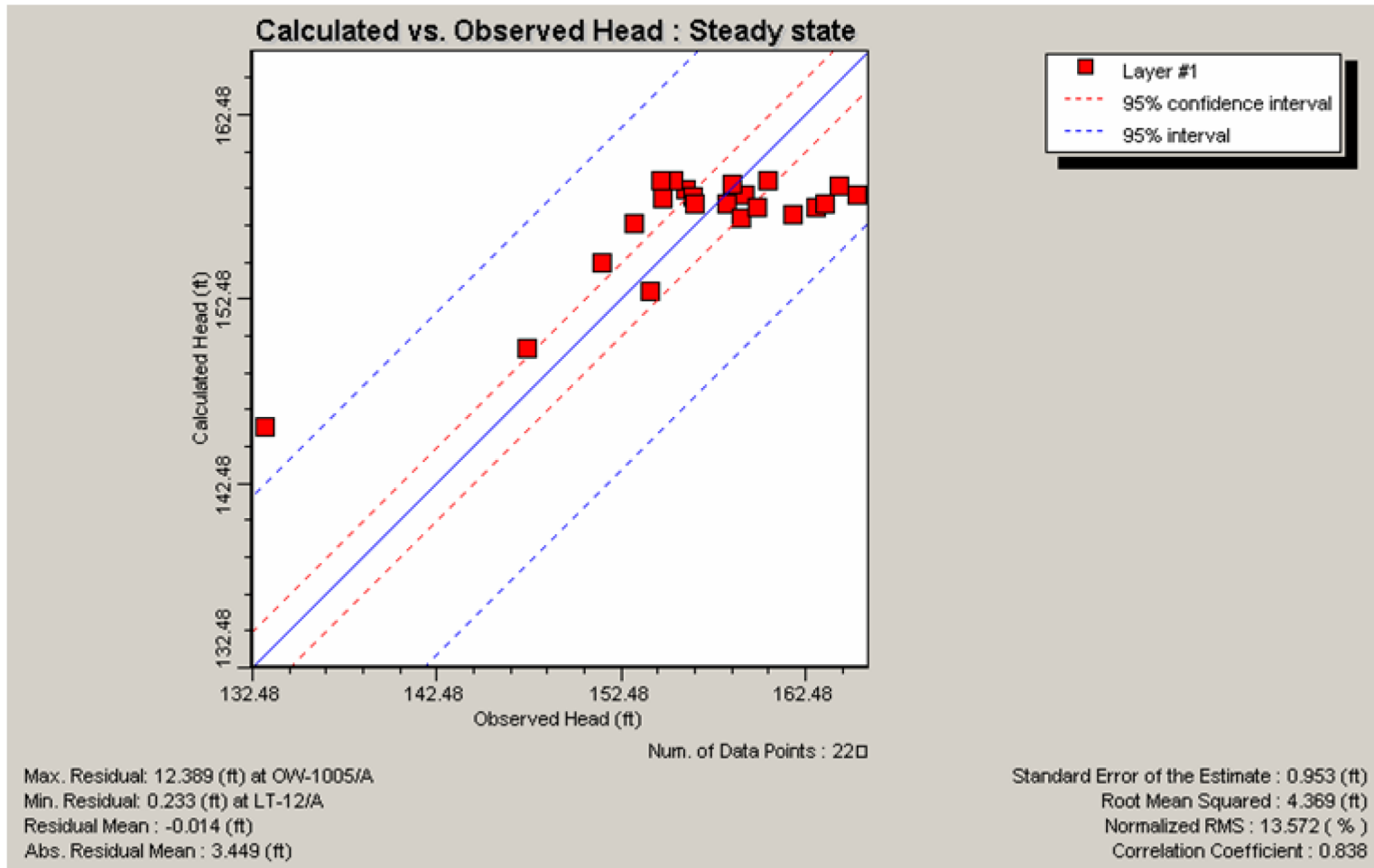
Figure 24:   Model 1 - Simulated Vs. Observed Water Levels for Run 109
(K_1 = 27 ft/day; R_1 =7 in/yr)
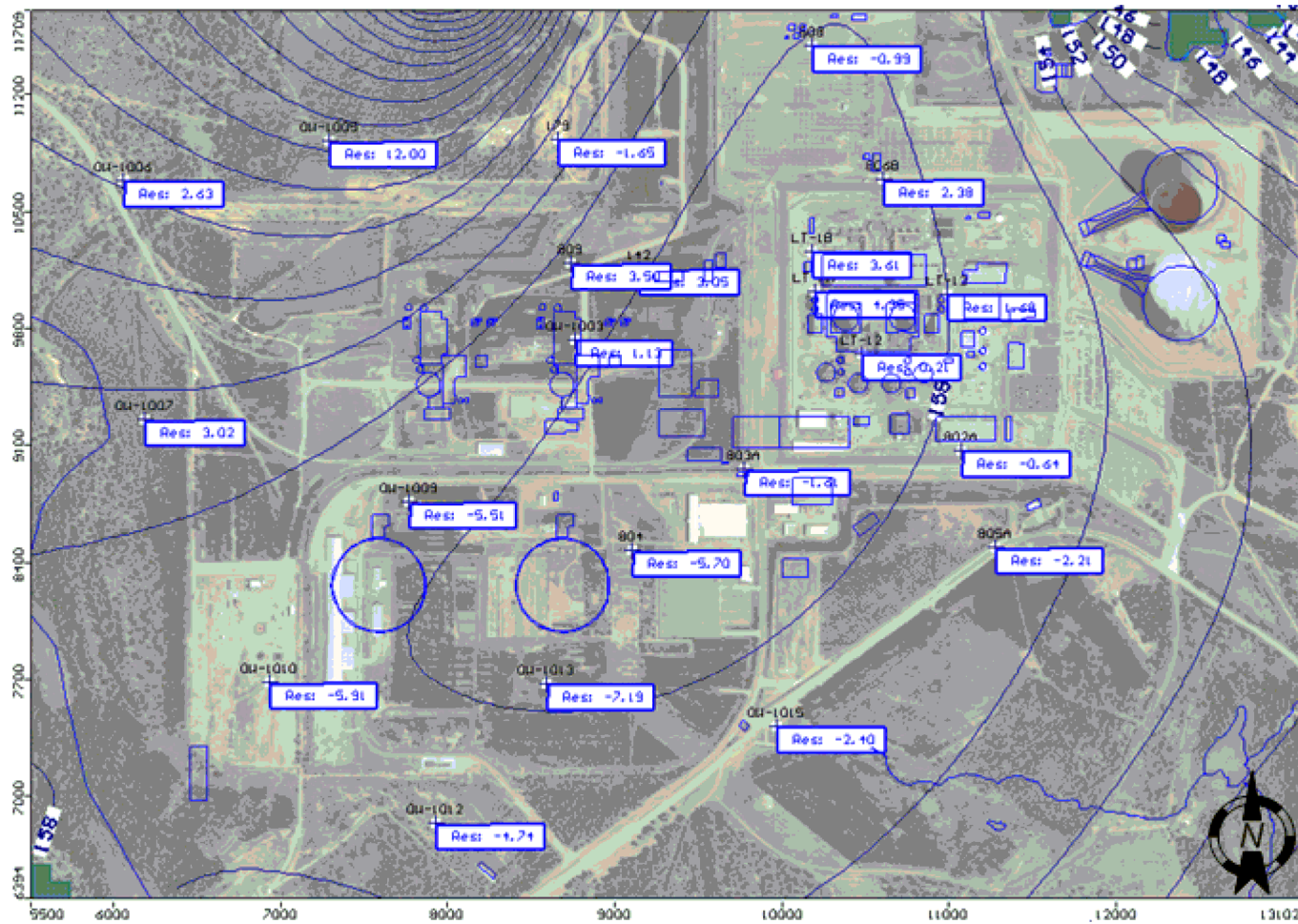
Figure 25:   Model 1 - Estimated Residuals for Run 109
(K_1 = 27 ft/day; R_1 =7 in/yr)

Figure 26:   Model 2 - Simulated Water Levels for Run 201
(K_1=27 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
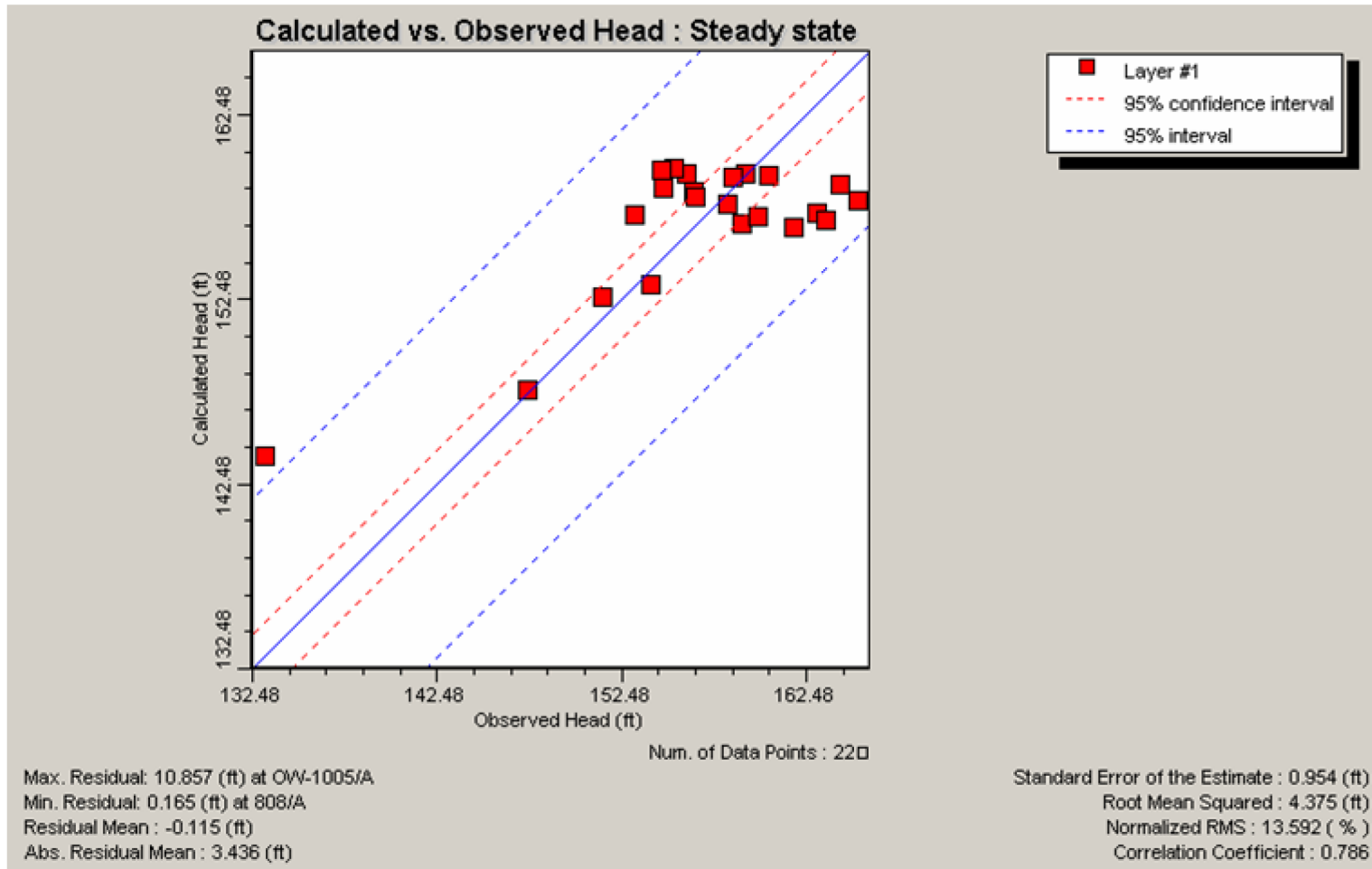
Figure 27: Model 2 - Simulated Vs. Observed Water Levels for Run 201
(K_1=27 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
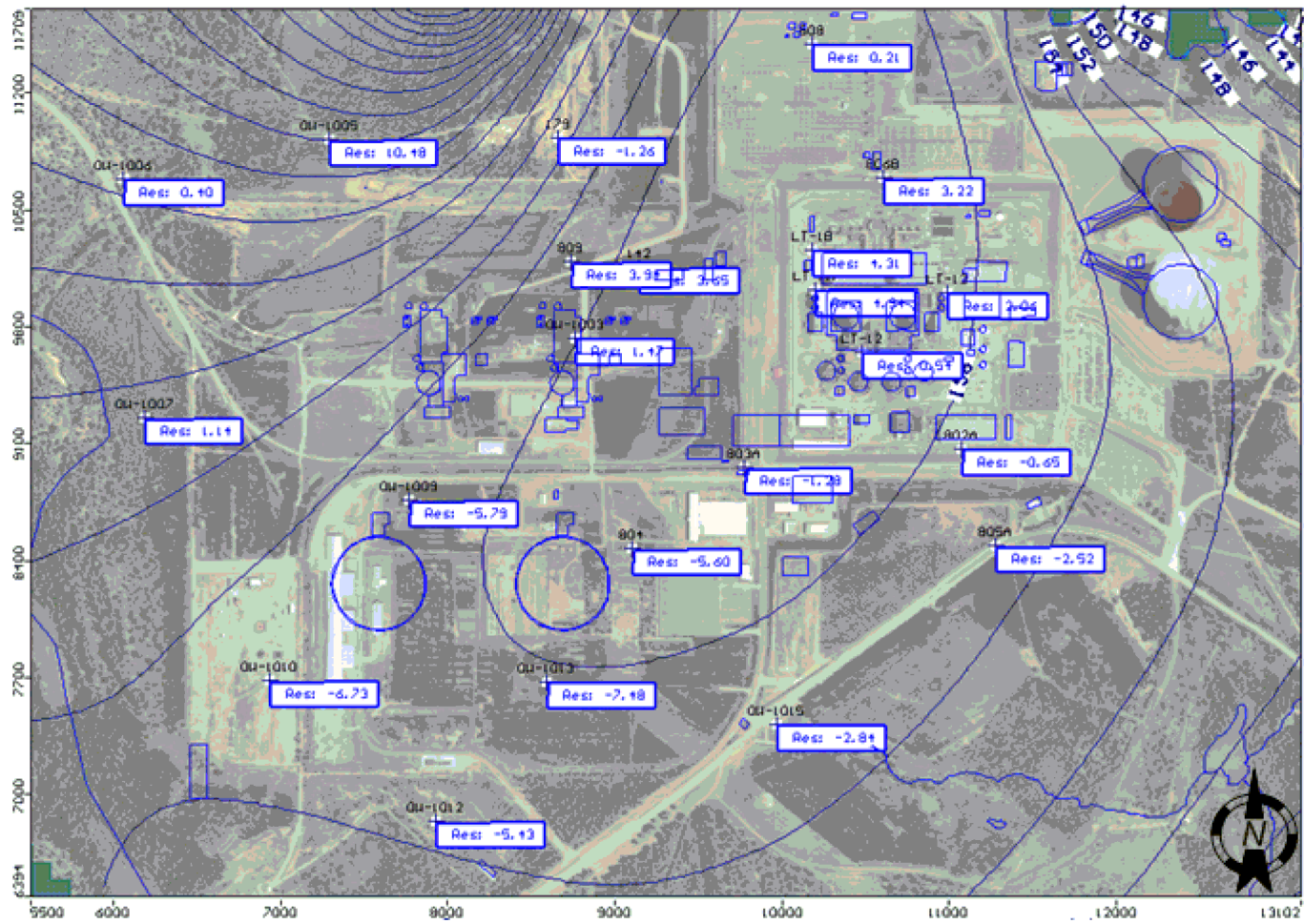
Figure 28: Model 2 - Estimated Residuals for Run 201
$(K_1=27 \text{ ft/day}; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 \text{ in/yr})$
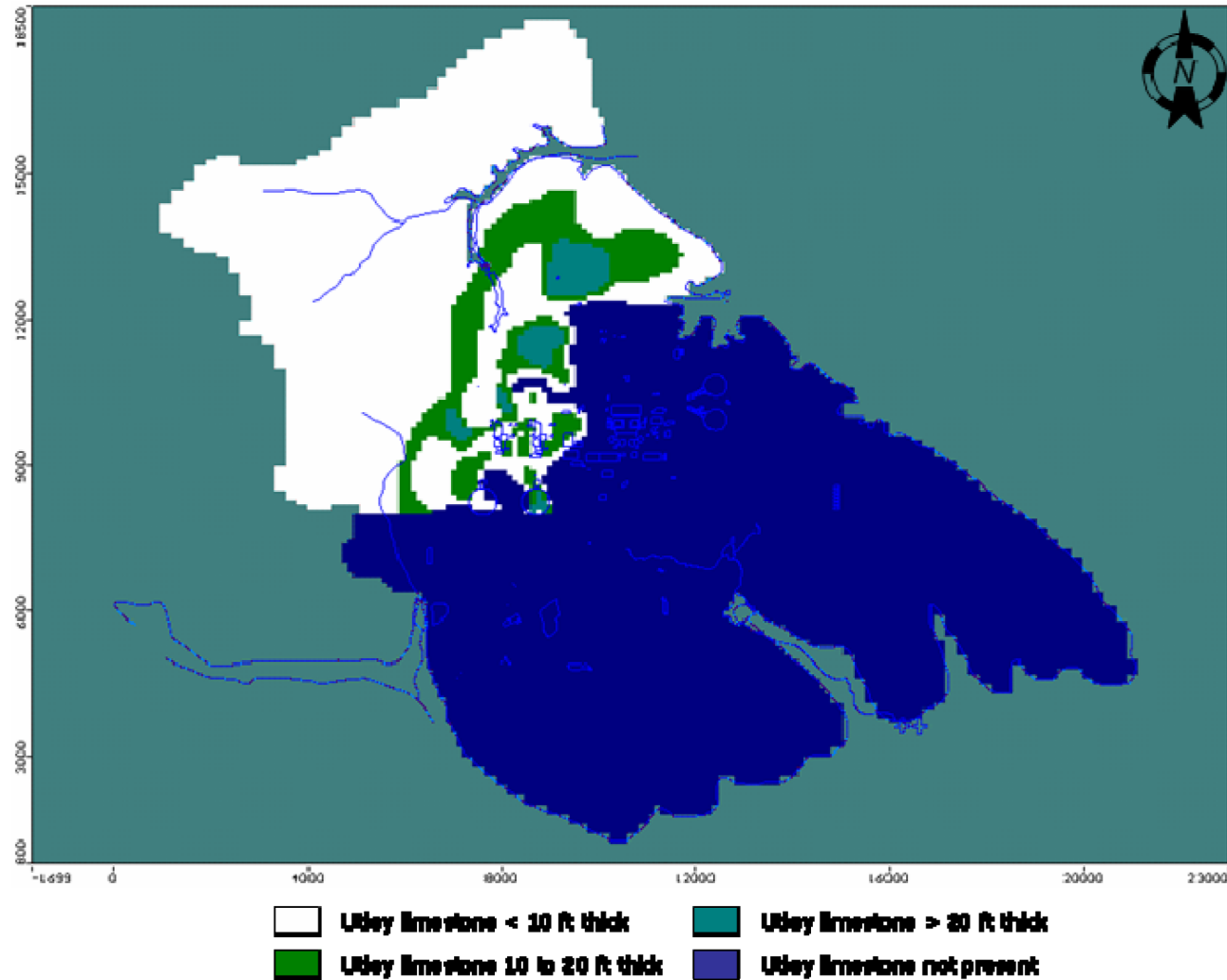
Figure 29: Model 3, Run 305 - Hydraulic Conductivity Zones Accounting for the Presence of the Utley Limestone (K_1=27; K_2=20; K_3=30; K_4=60 ft/day)
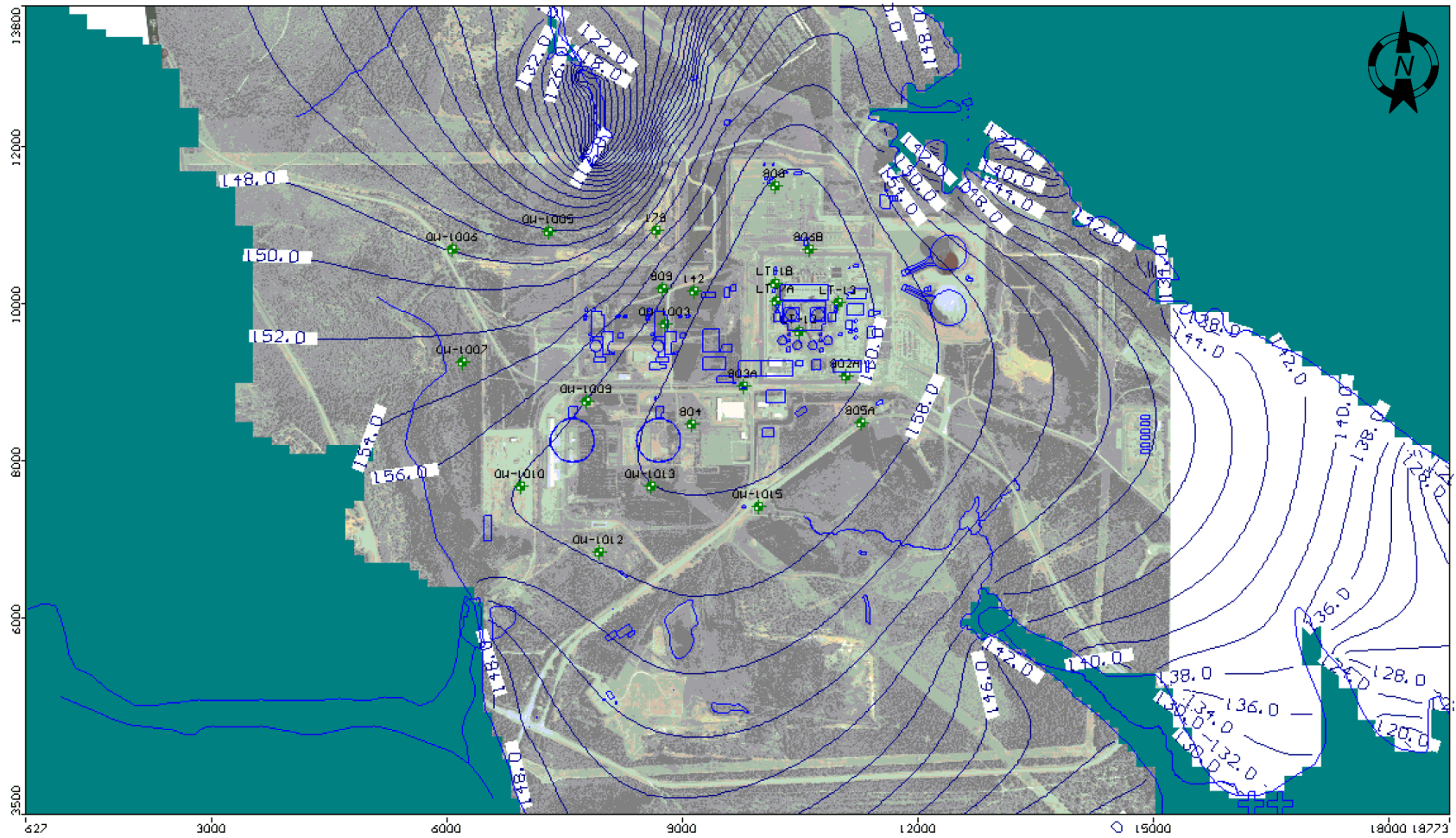
Figure 30:   Model 3 - Simulated Water Levels for Run 305
(K_1=27; K_2=20; K_3=30; K_4=60 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
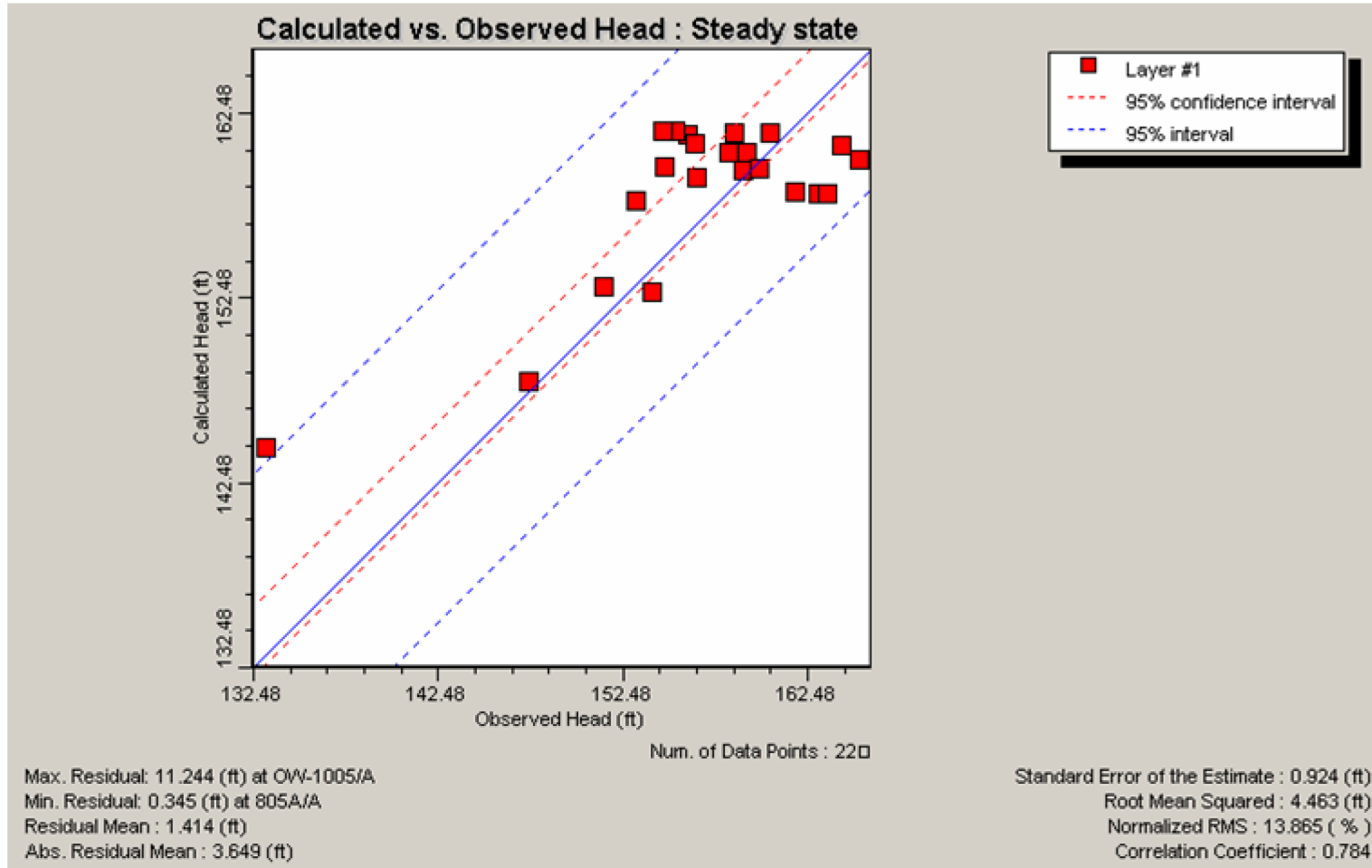
Figure 31: Model 3 - Simulated Vs. Observed Water Levels for Run 305
(K_1=27; K_2=20; K_3=30; K_4=60 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
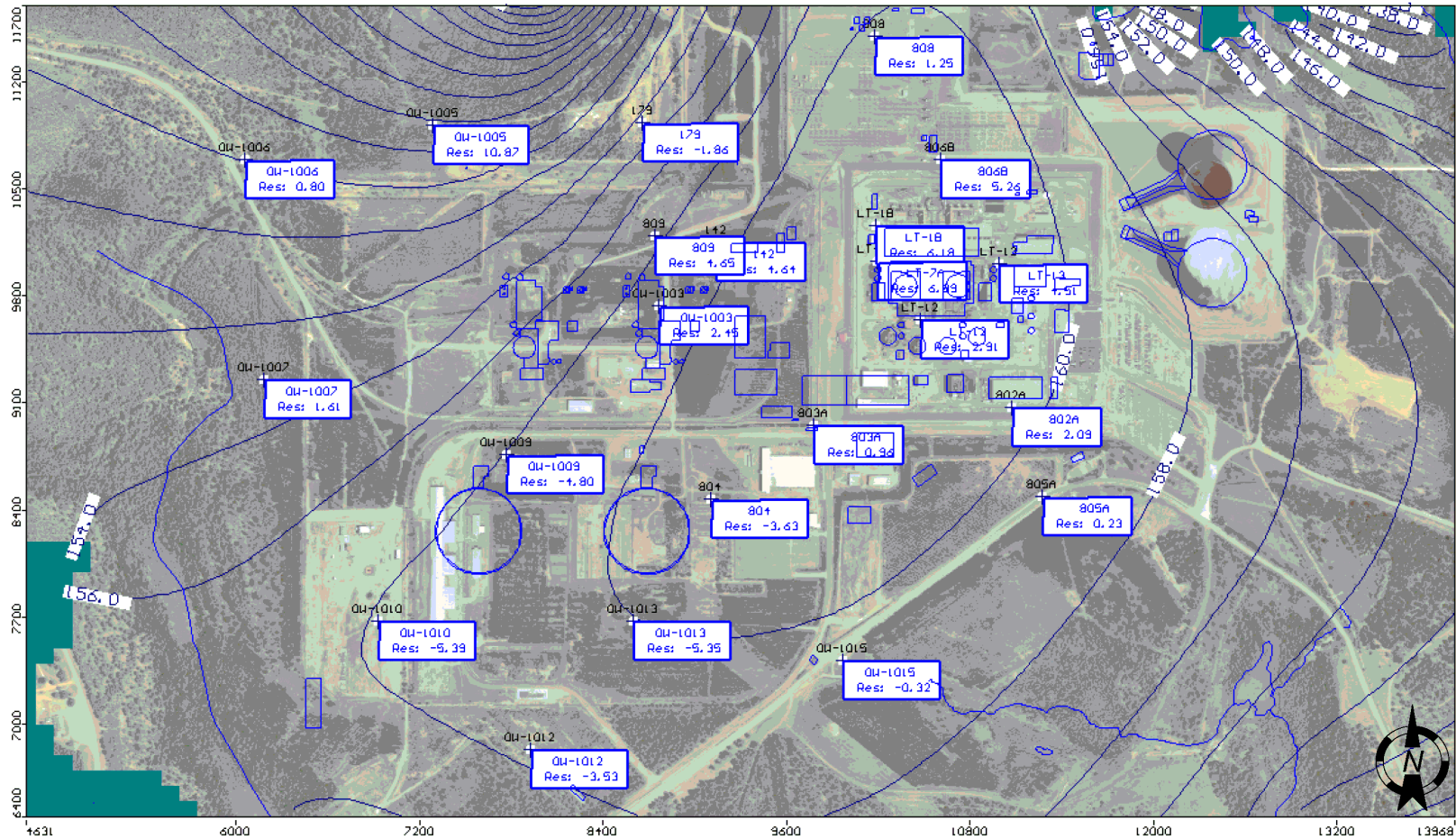
Figure 32:   Model 3 - Estimated Residuals for Run 305
(K_1=27; K_2=20; K_3=30; K_4=60 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
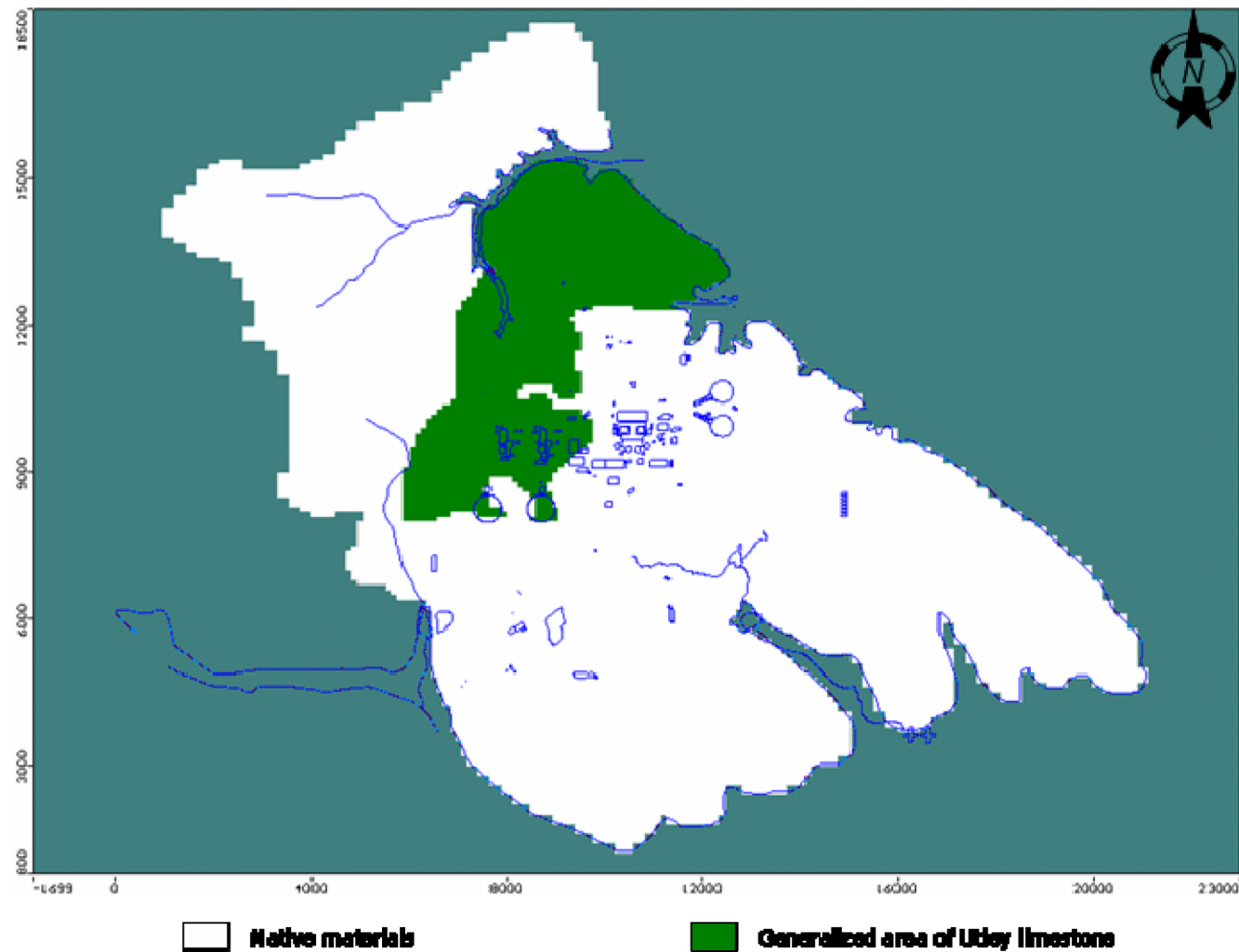
Figure 33: **Model 4, Run 403 – Simplified Hydraulic Conductivity Zones Accounting for the Presence of the Utley Limestone (K_1=20; K_2=35 ft/day)**
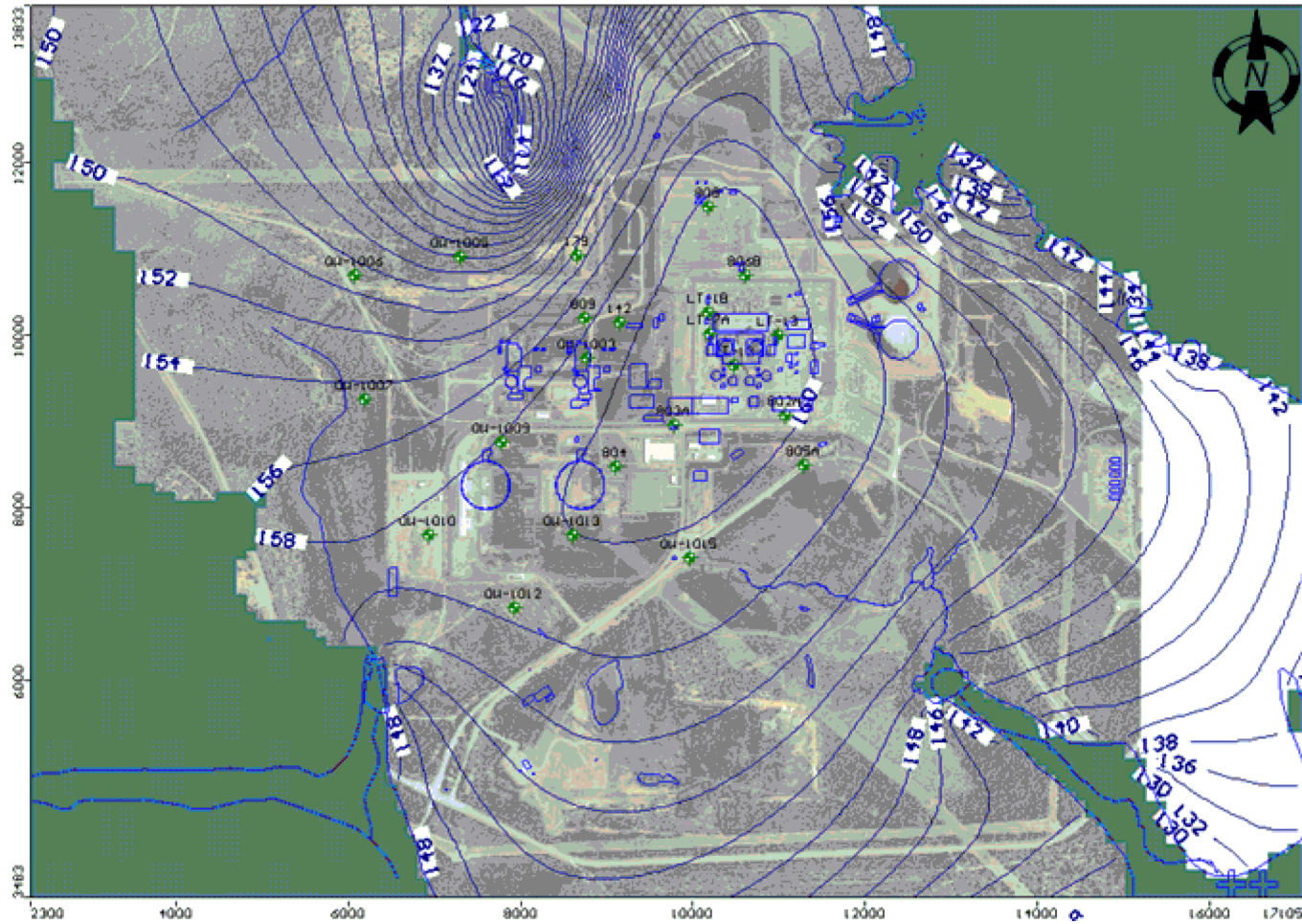
Figure 34:   Model 4- Simulated Water Levels for Run 403
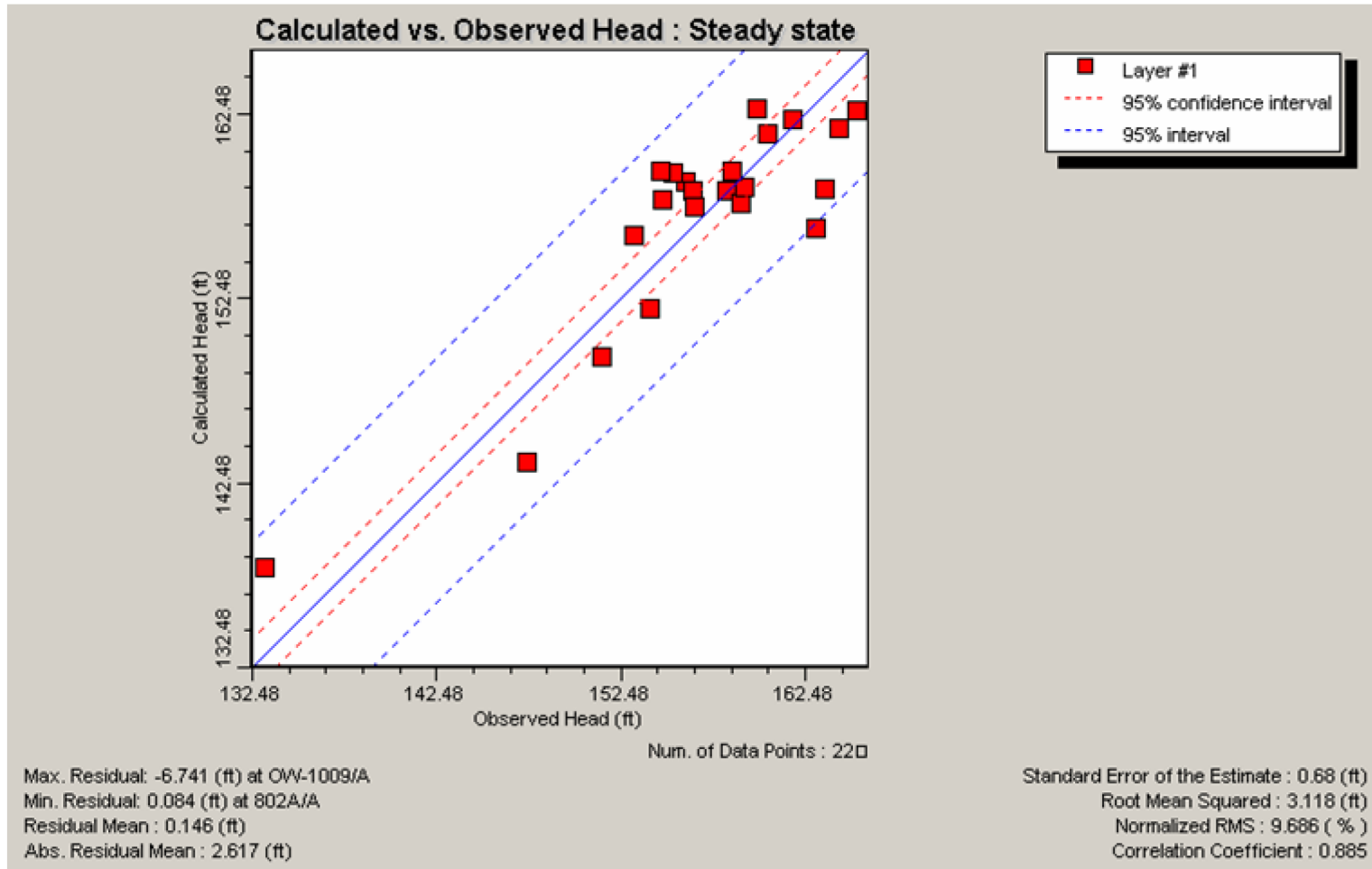**(K_1=20; K_2=35 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)**
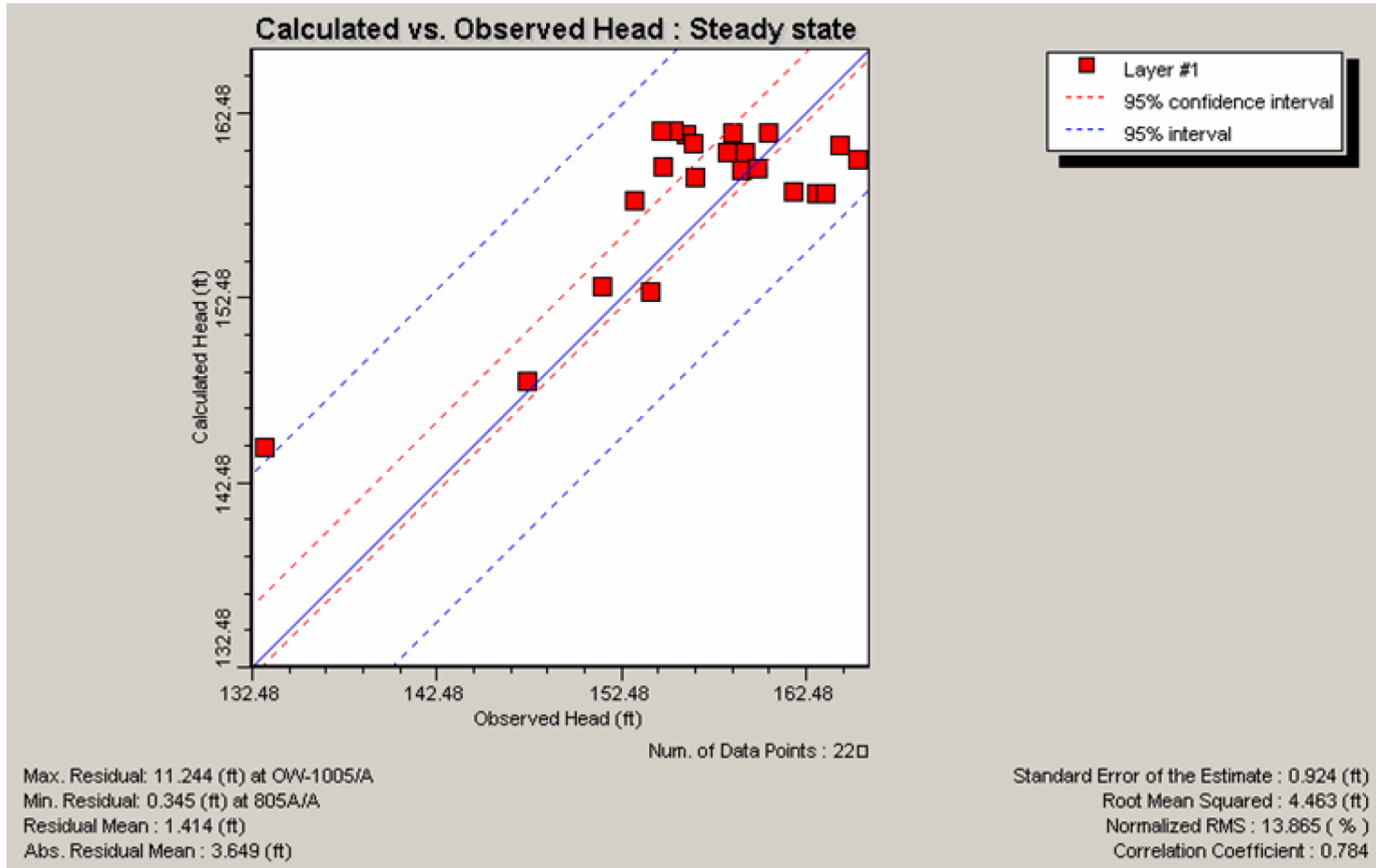
Figure 35: Model 4- Simulated Vs. Observed Water Levels for Run 403
**(K_1=20; K_2=35 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)**
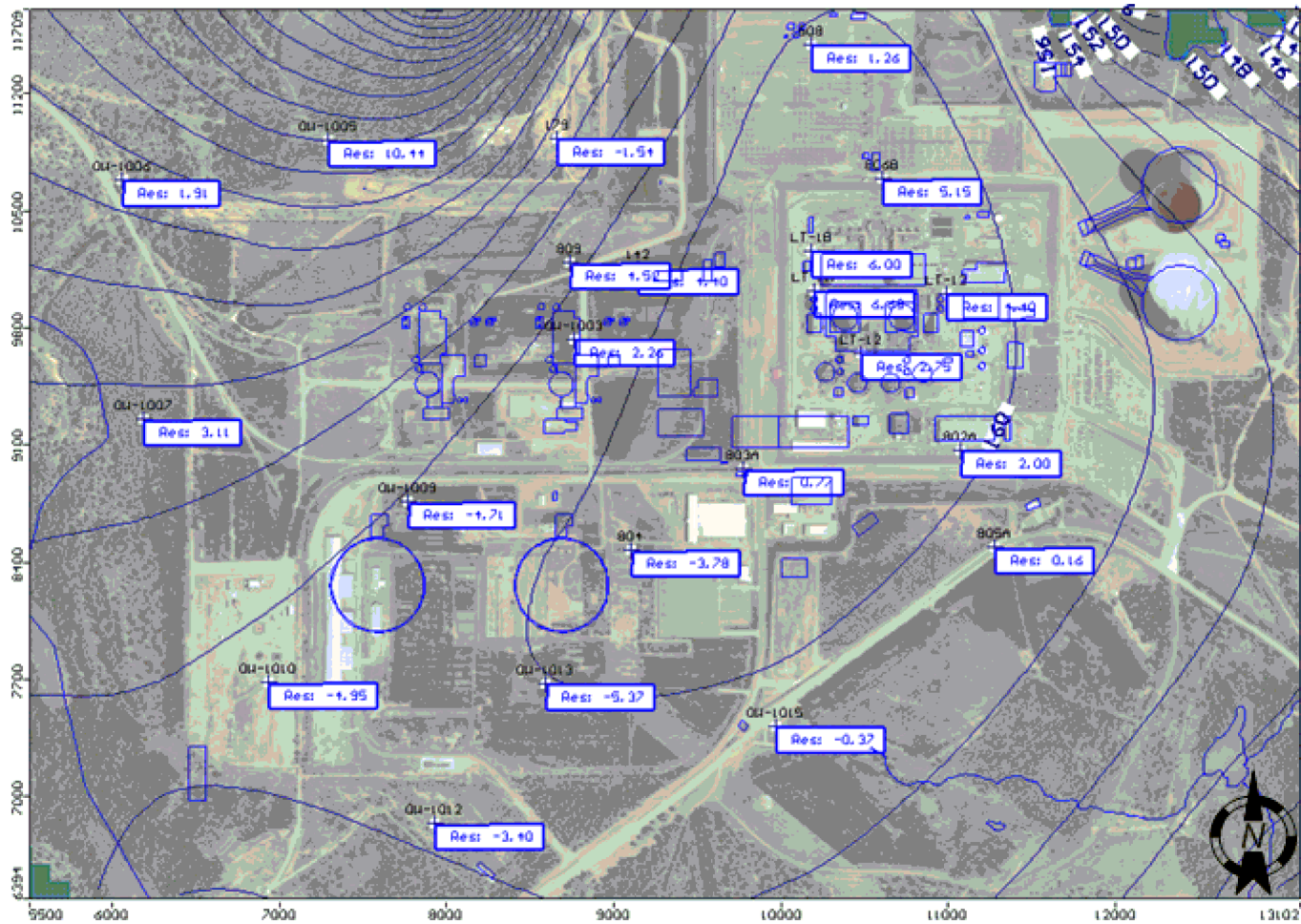
Figure 36: Model 4- Estimated Residuals for Run 403
(K_1=20; K_2=35 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
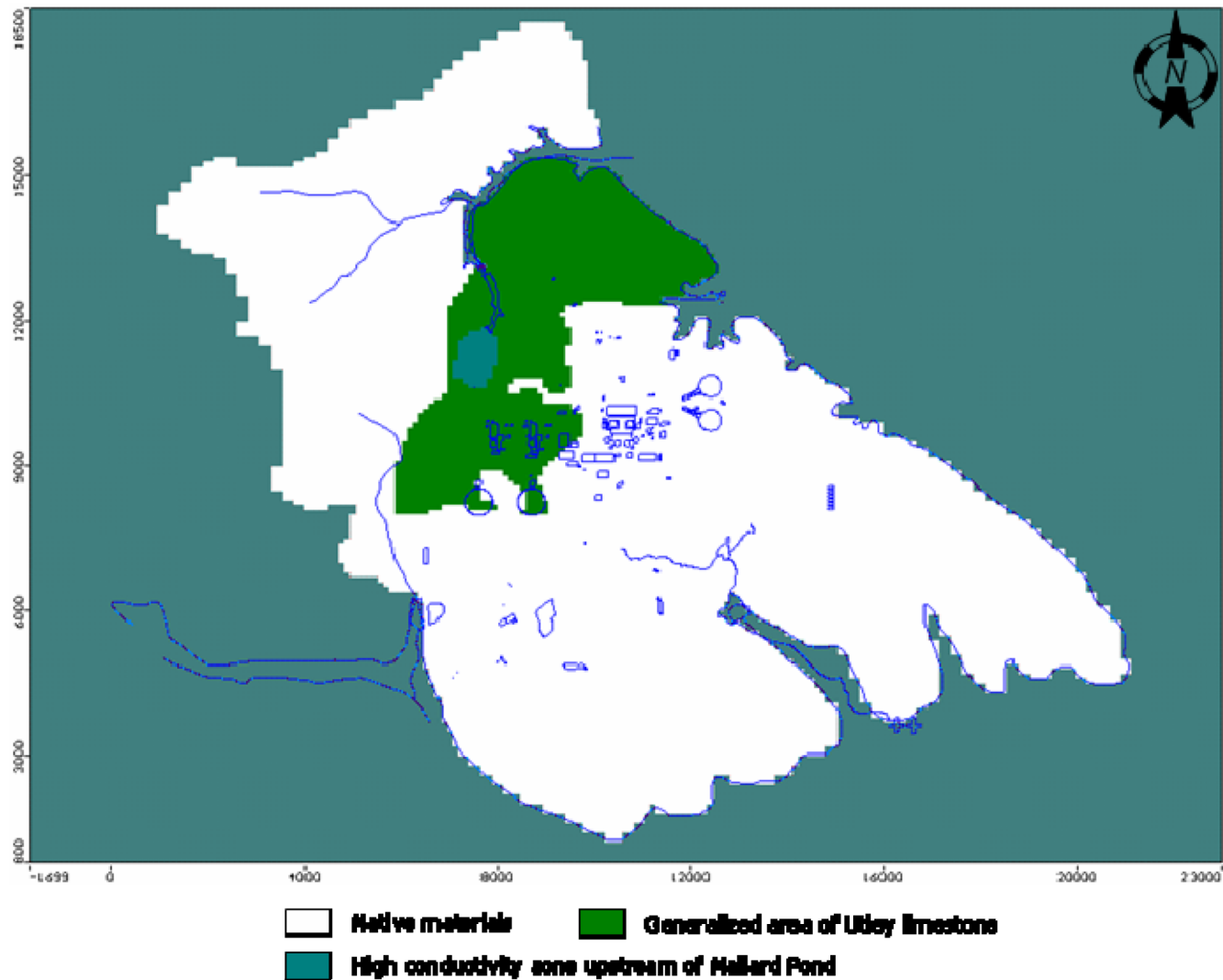
Figure 37:   Model 5, Run 504 – Hydraulic Conductivity Zones as for Model 4 and a High Conductivity Zone Upstream of Mallard Pond (K_1=20; K_2=35; K_3=100 ft/day)
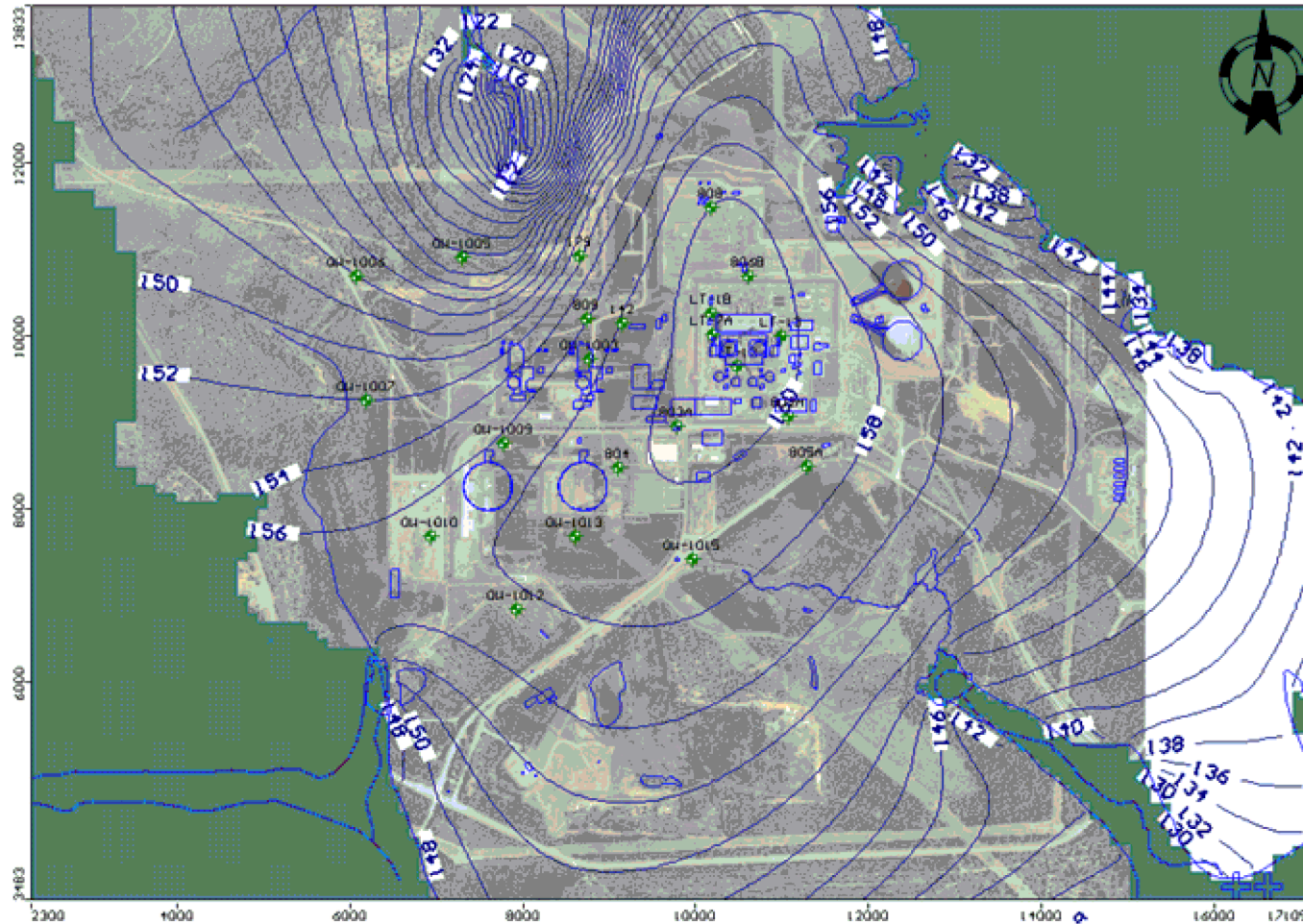
Figure 38:   Model 5- Simulated Water Levels for Run 504
(K_1=20; K_2=35; K_3=100 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
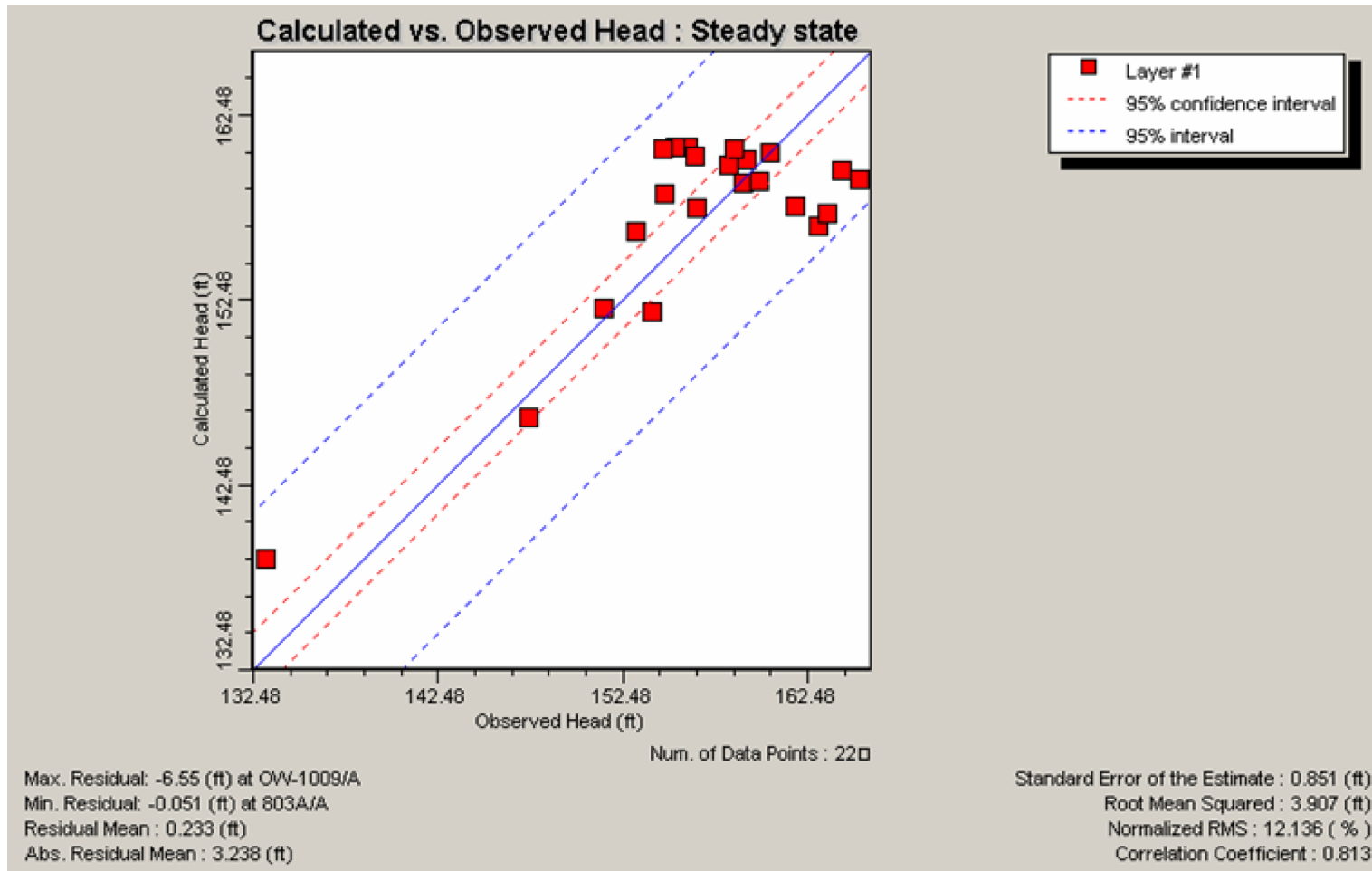
Figure 39: Model 5- Simulated Vs. Observed Water Levels for Run 504 (K$_1$=20; K$_2$=35; K$_3$=100 ft/day; R$_1$=10; R$_2$=6; R$_3$=6; R$_4$=4; R$_5$=0 in/yr)
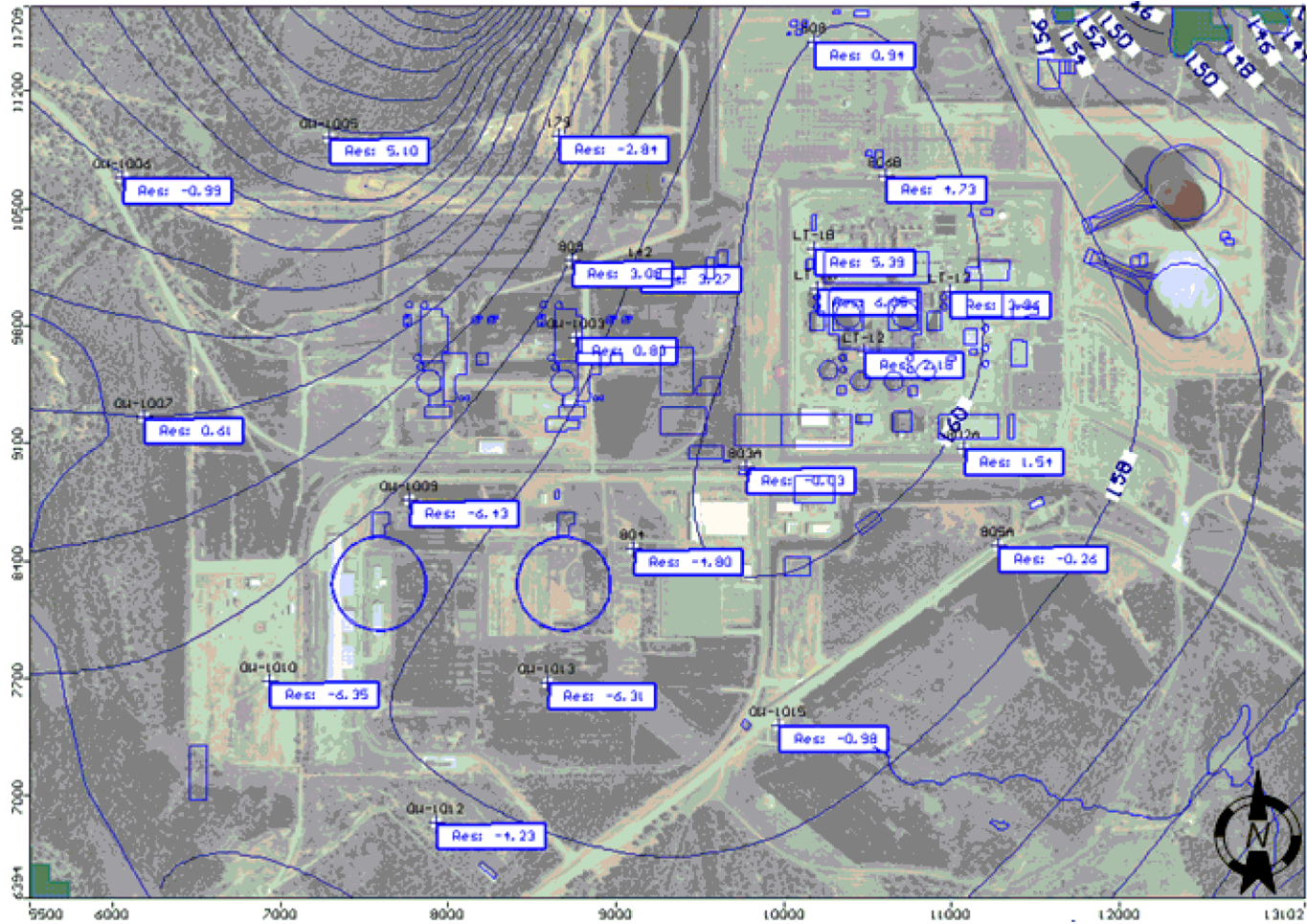
Figure 40: Model 5- Estimated Residuals for Run 504
(K_1=20; K_2=35; K_3=100 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
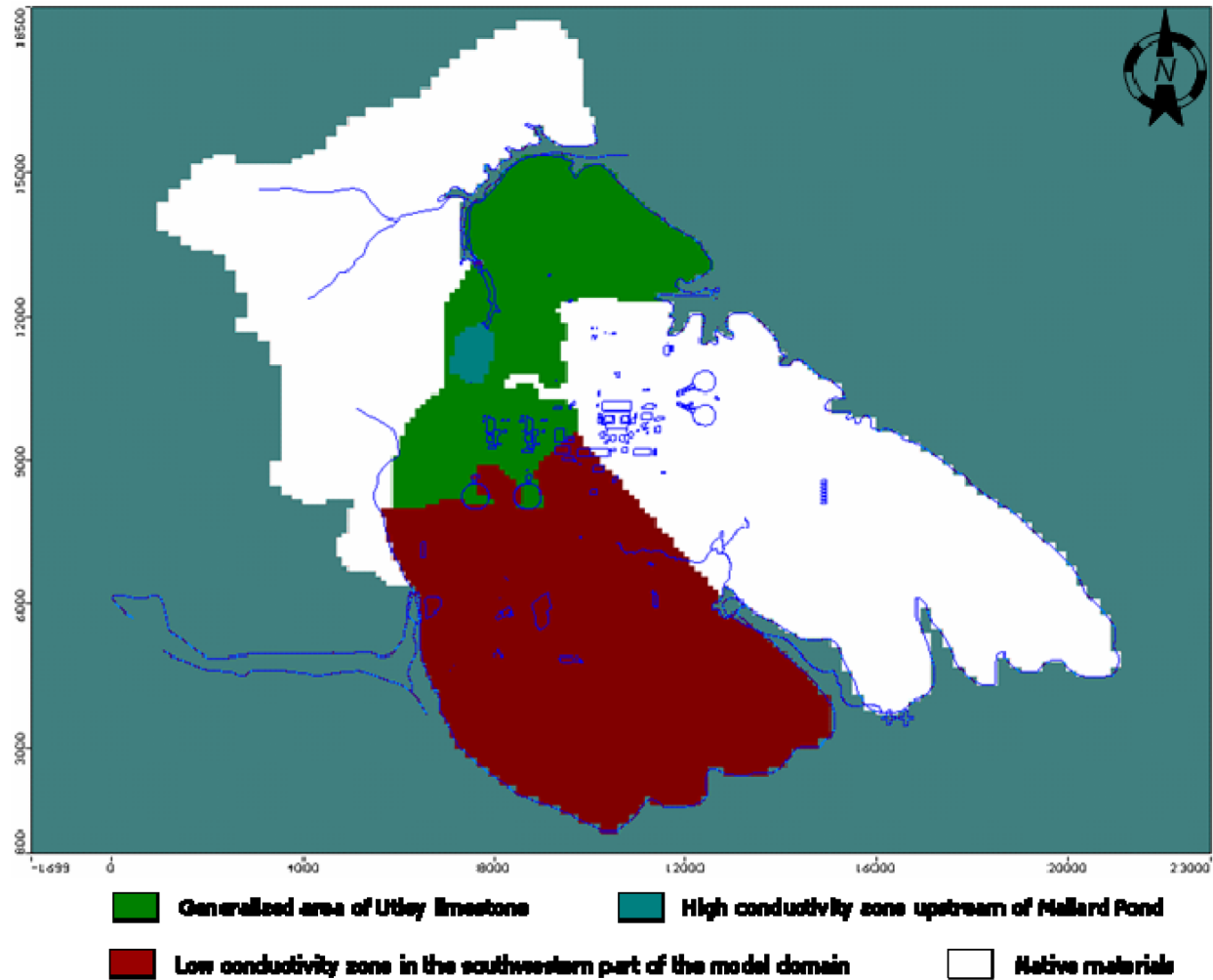
Figure 41: Model 6, Run 612 – Hydraulic Conductivity Zones as for Model 5 and a Low Conductivity Zone in the Southwestern Quarter of the Model (K_1=28; K_2=33; K_3=200; K_4=8 ft/day)
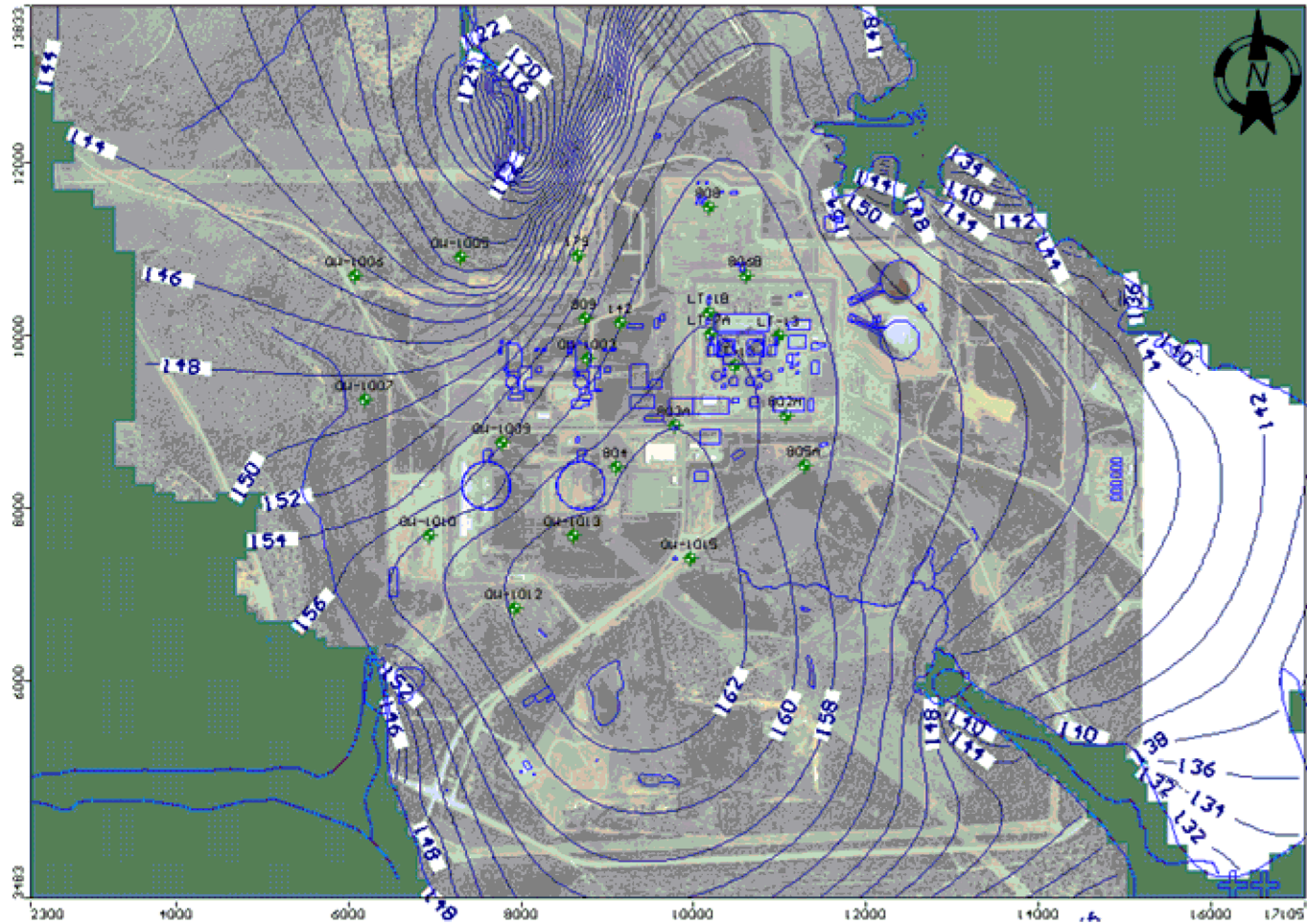
Figure 42: Model 6 - Simulated Water Levels for Run 612
$(K_1=28; K_2=33; K_3=200; K_4=8$ ft/day; $R_1=10; R_2=6; R_3=6; R_4=4; R_5=0$ in/yr)
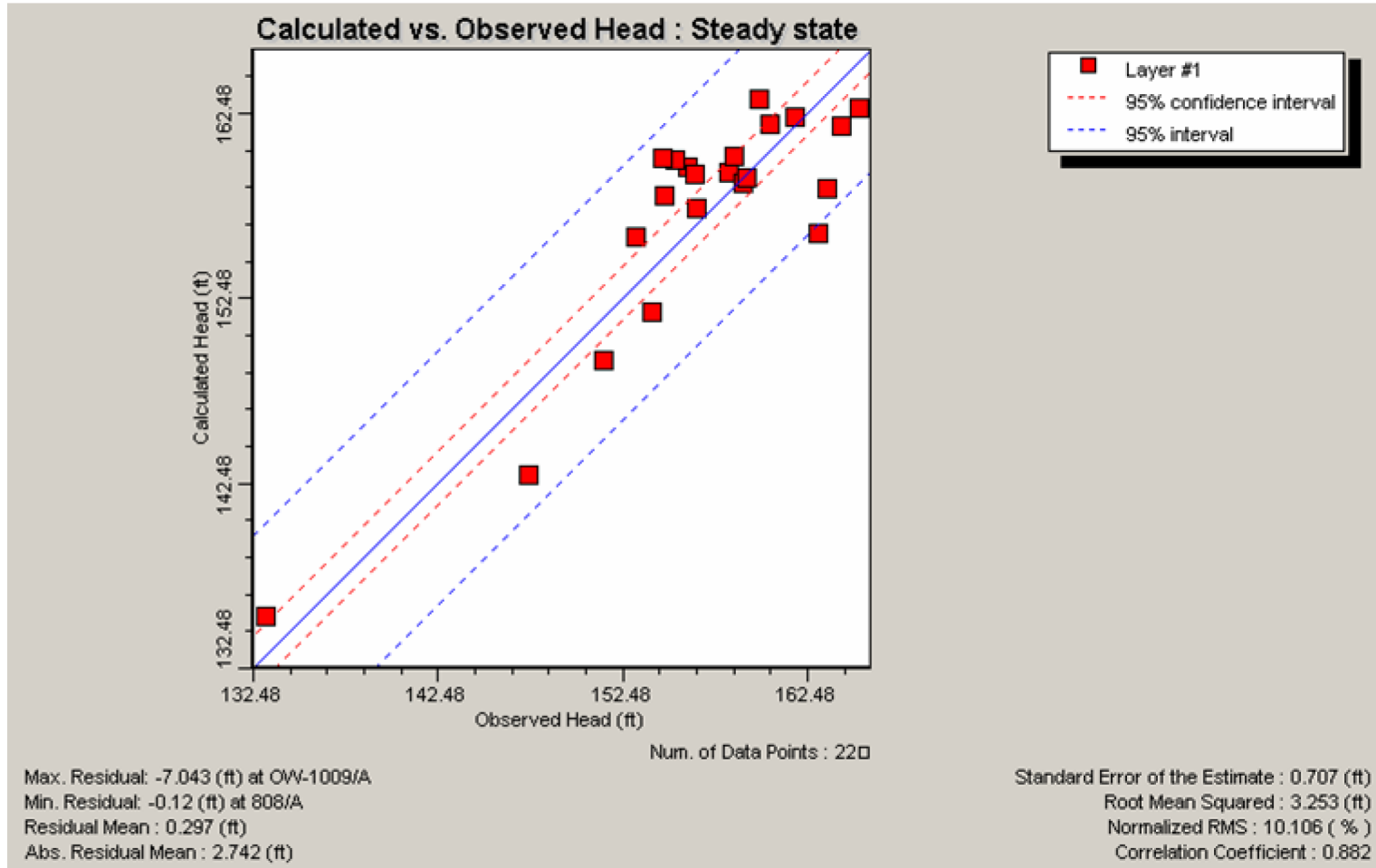
Figure 43:   Model 6 - Simulated Vs. Observed Water Levels for Run 612
(K_1=28; K_2=33; K_3=200; K_4=8 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
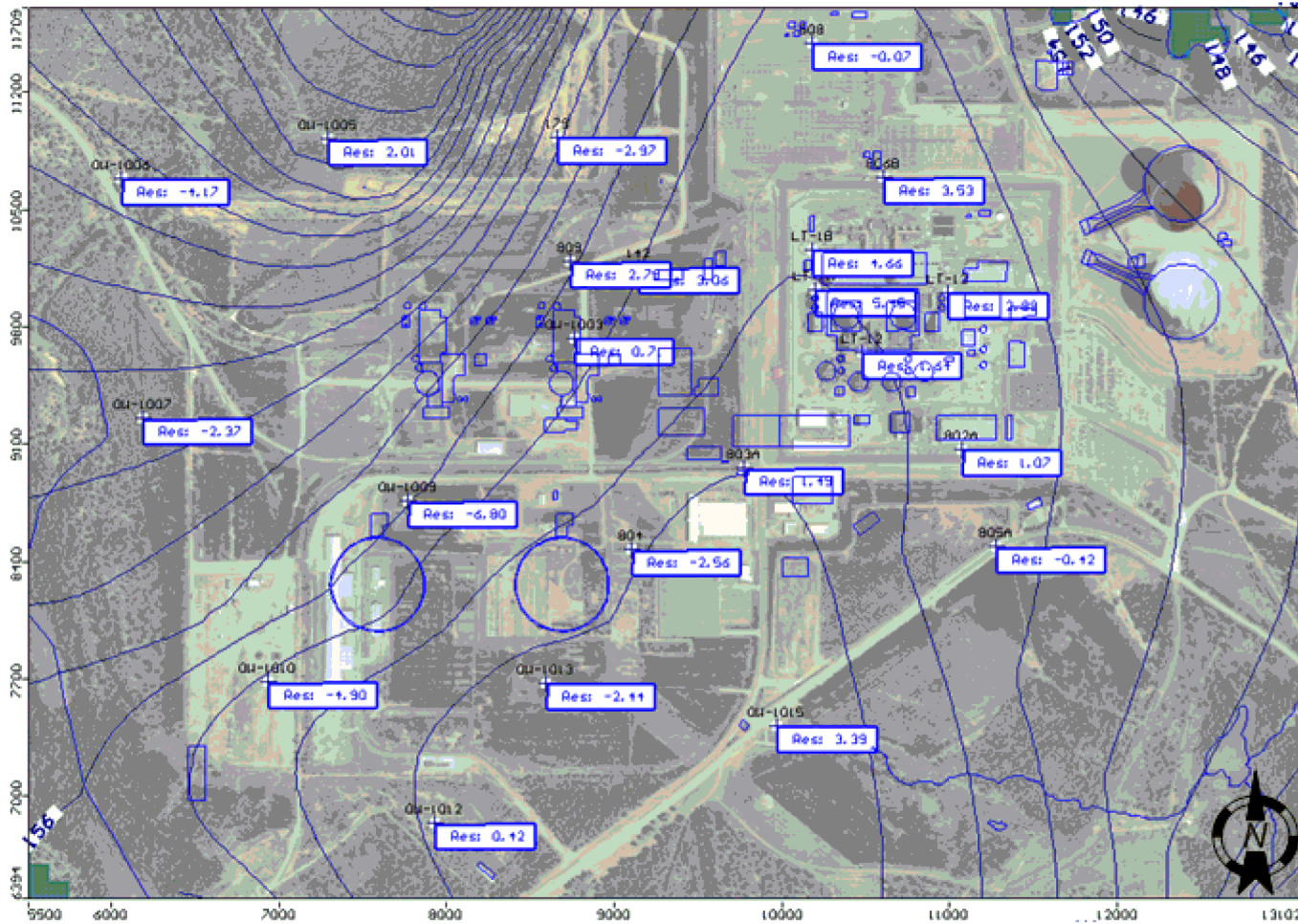
Figure 44:   Model 6- Estimated Residuals for Run 612
(K_1=28; K_2=33; K_3=200; K_4=8 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
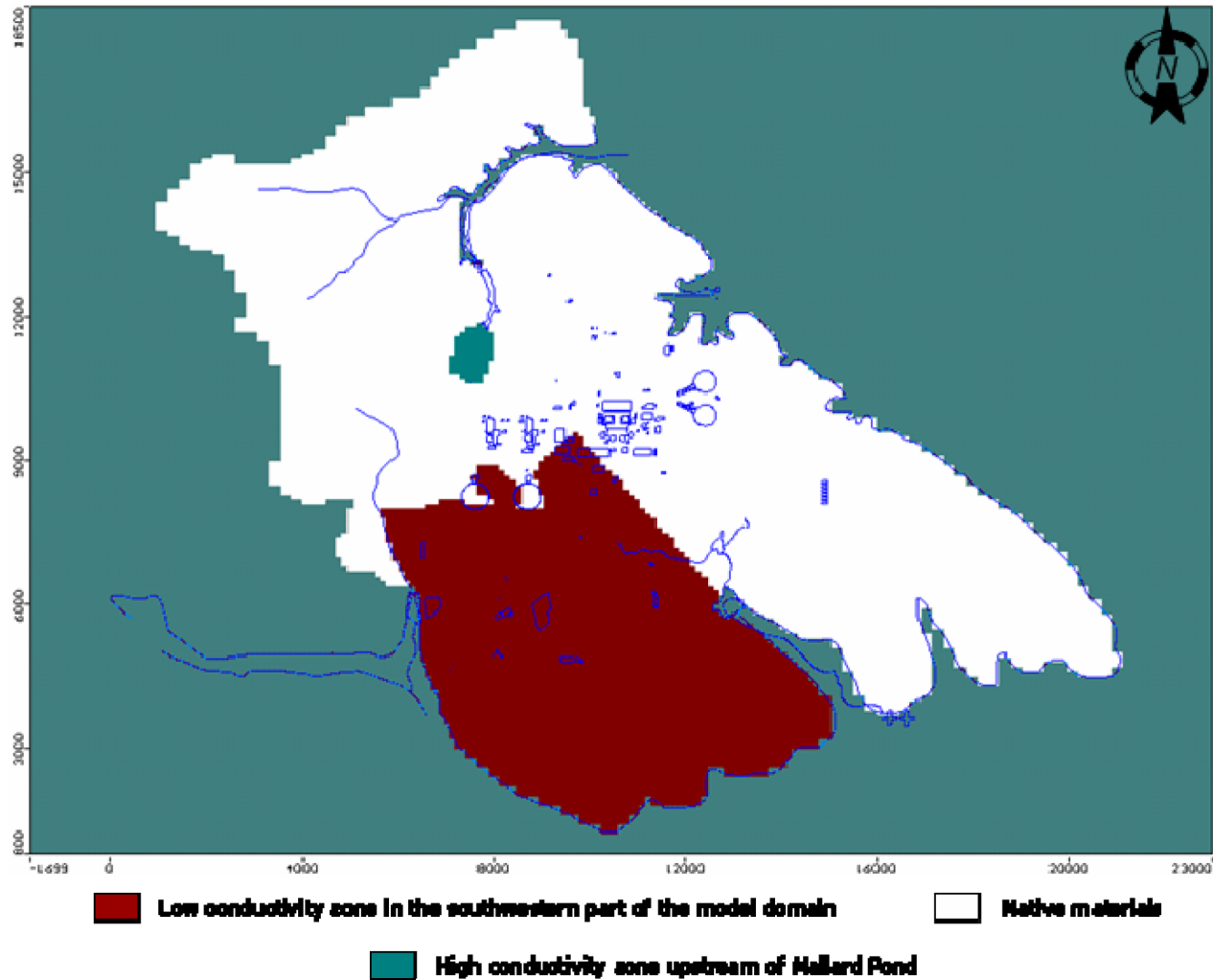
**Figure 45:   Model 7, Run 708 – Simplified Version of Model 6
(K_1=32; K_2=100; K_3=8 ft/day)**
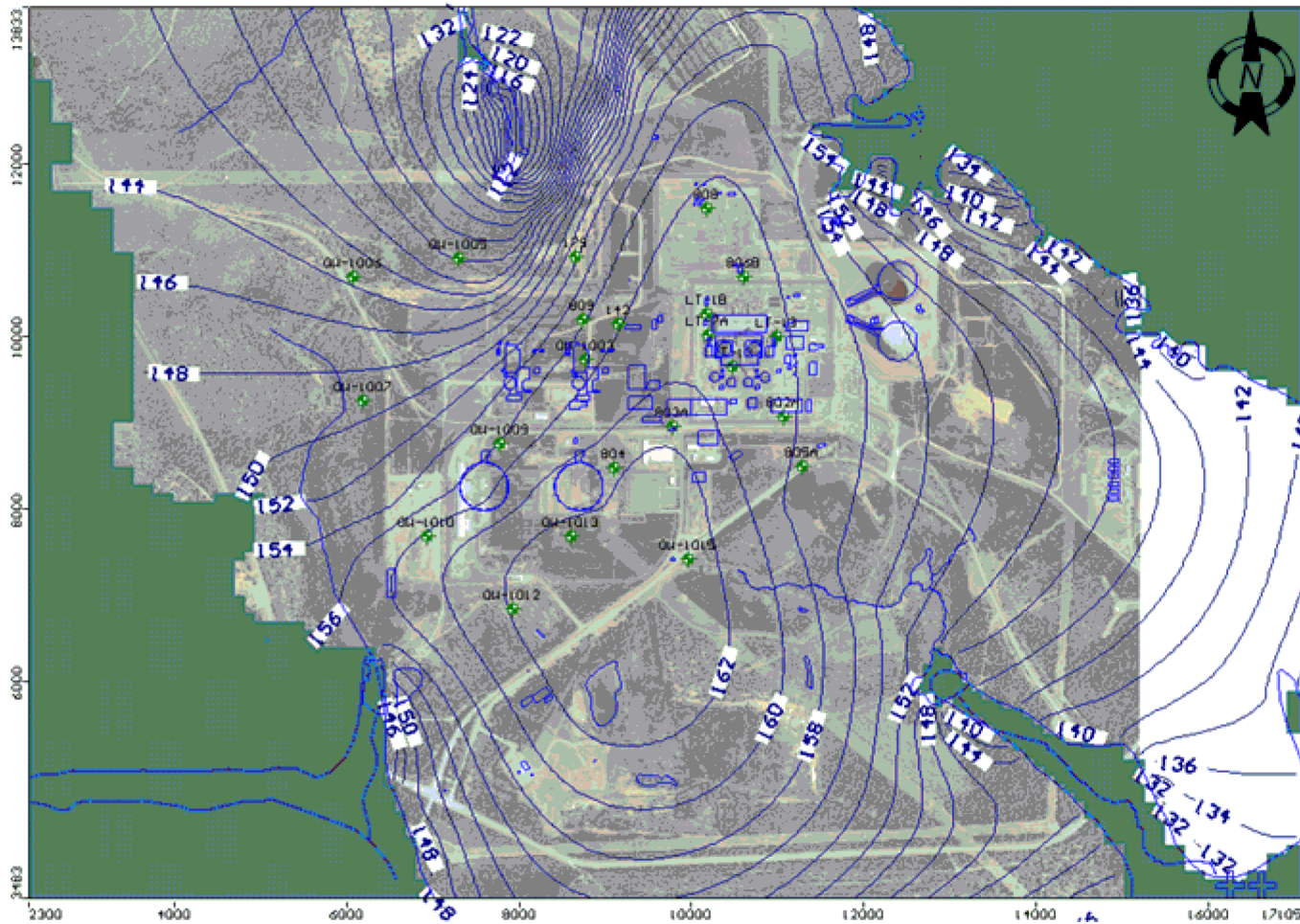
Figure 46:   Model 7 - Simulated Water Levels for Run 708
(K_1=32; K_2=100; K_3=8 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)

Figure 47: Model 7 - Simulated Vs. Observed Water Levels for Run 708
(K_1=32; K_2=100; K_3=8; ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)

Figure 48:   Model 7- Estimated Residuals for Run 708
(K$_1$=32; K$_2$=100; K$_3$=8; ft/day; R$_1$=10; R$_2$=6; R$_3$=6; R$_4$=4; R$_5$=0 in/yr)
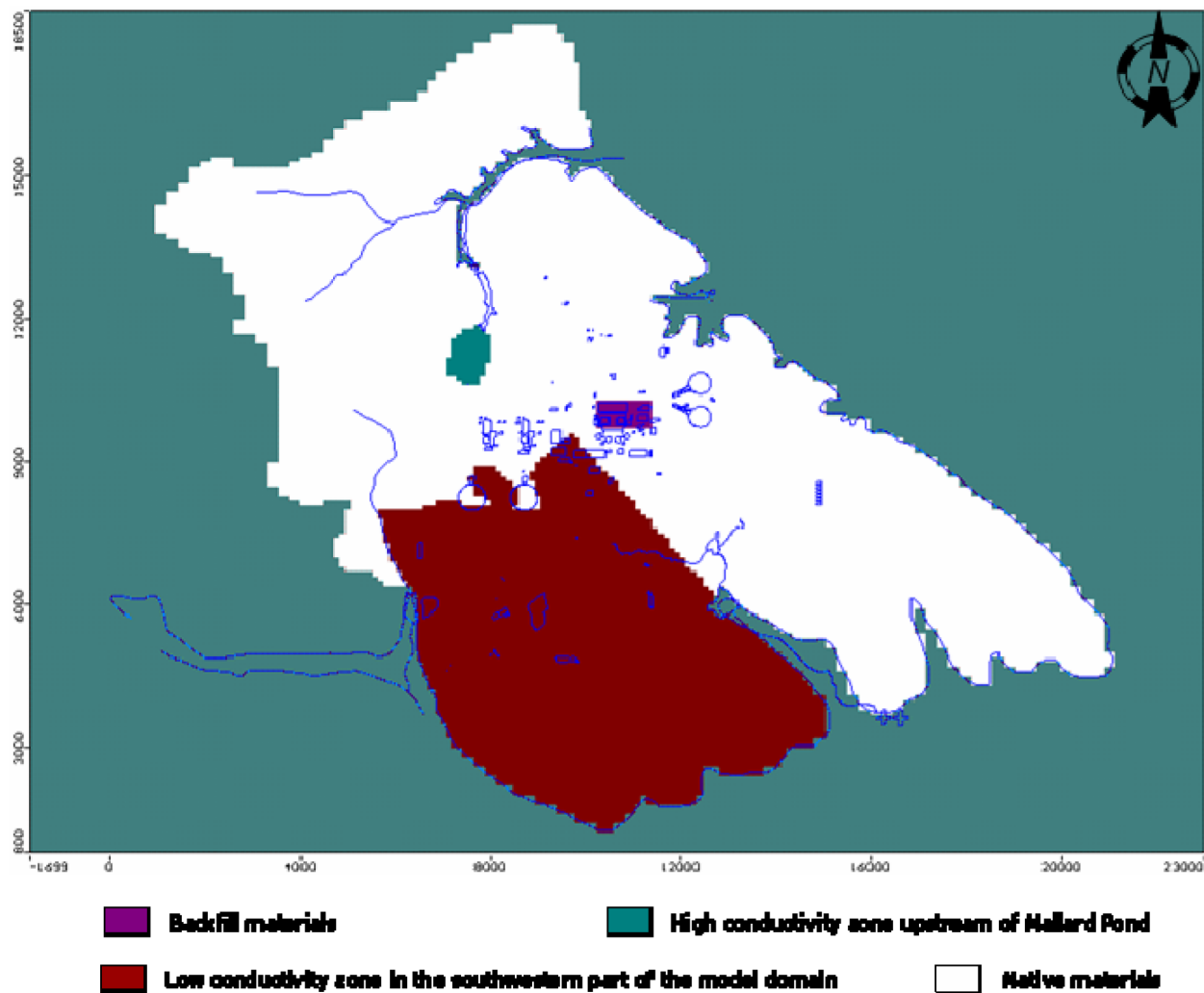
Figure 49: Hydraulic Conductivity Zones Used in Model 7 to Evaluate the Sensitivity of the Model to the Hydraulic Conductivity for the Backfill Material Around Units 1 & 2
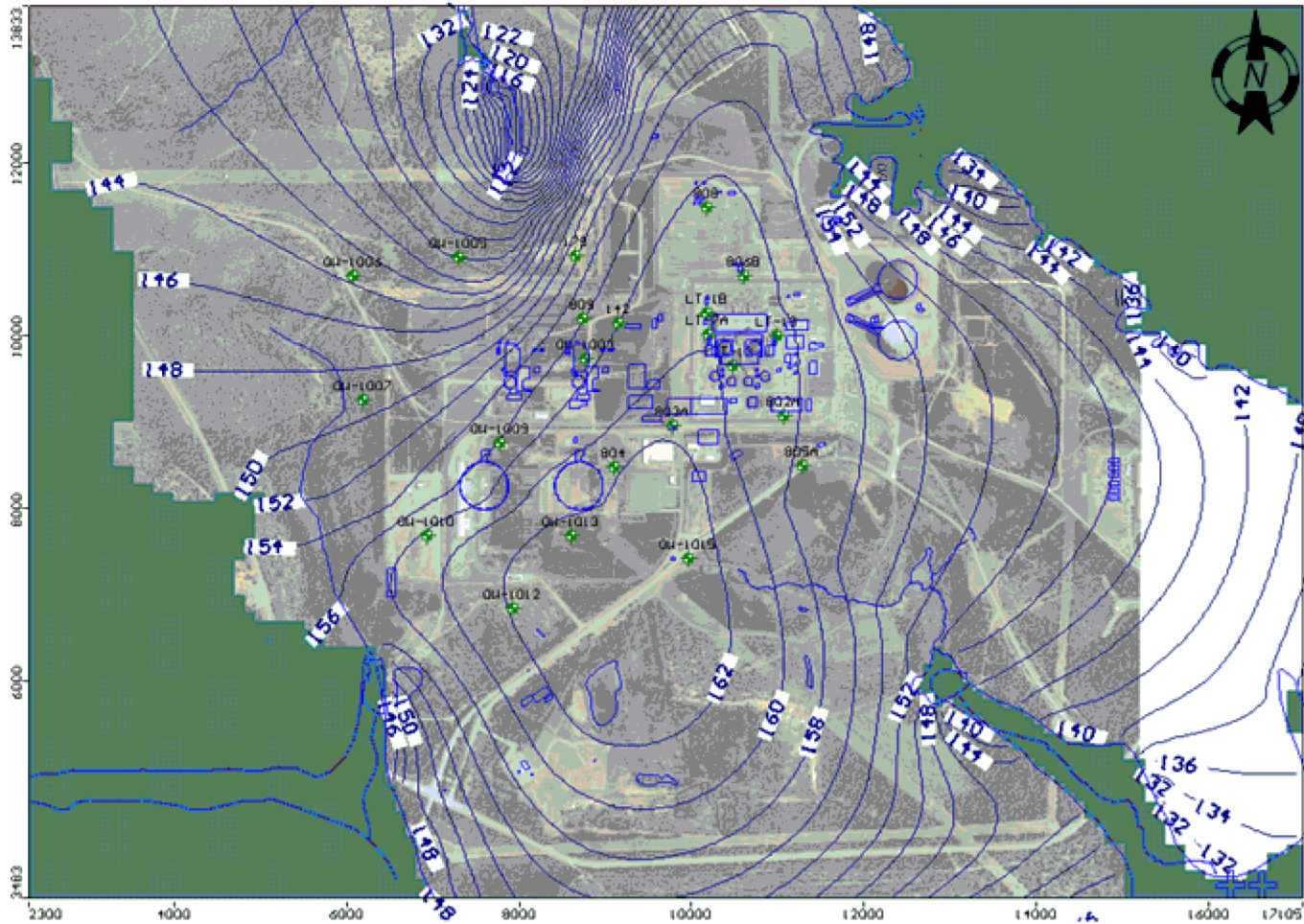
Figure 50:   Simulated Water Levels with Model 7 Accounting for the Backfill Material for Units 1 & 2 as a Different Material with Hydraulic Conductivity Equal to 3.3 ft/day
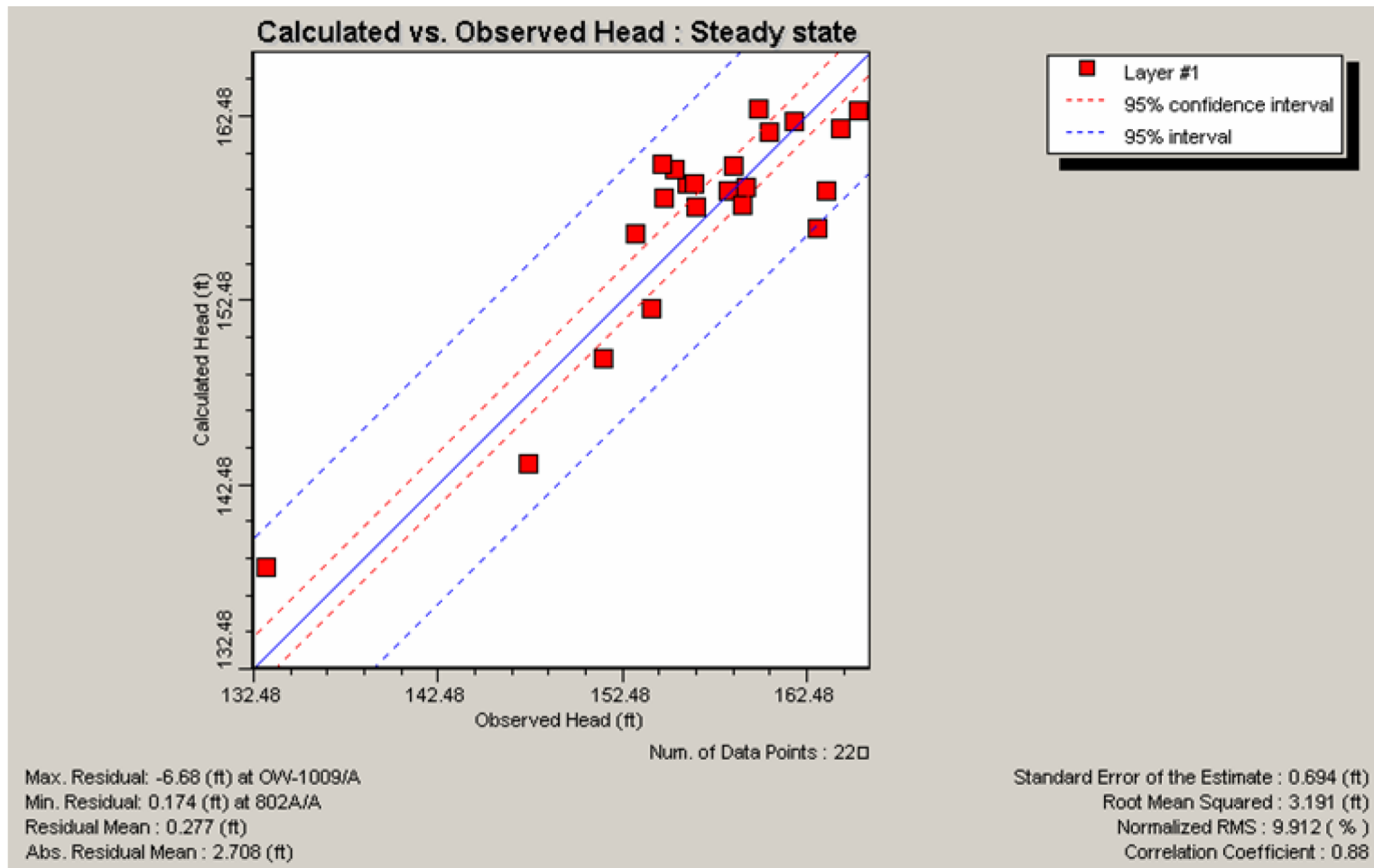
Figure 51:   Simulated Versus Observed Water Levels with Model 7 Accounting for the Backfill Material for Units 1 & 2 as a Different Material with Hydraulic Conductivity Equal to 3.3 ft/day
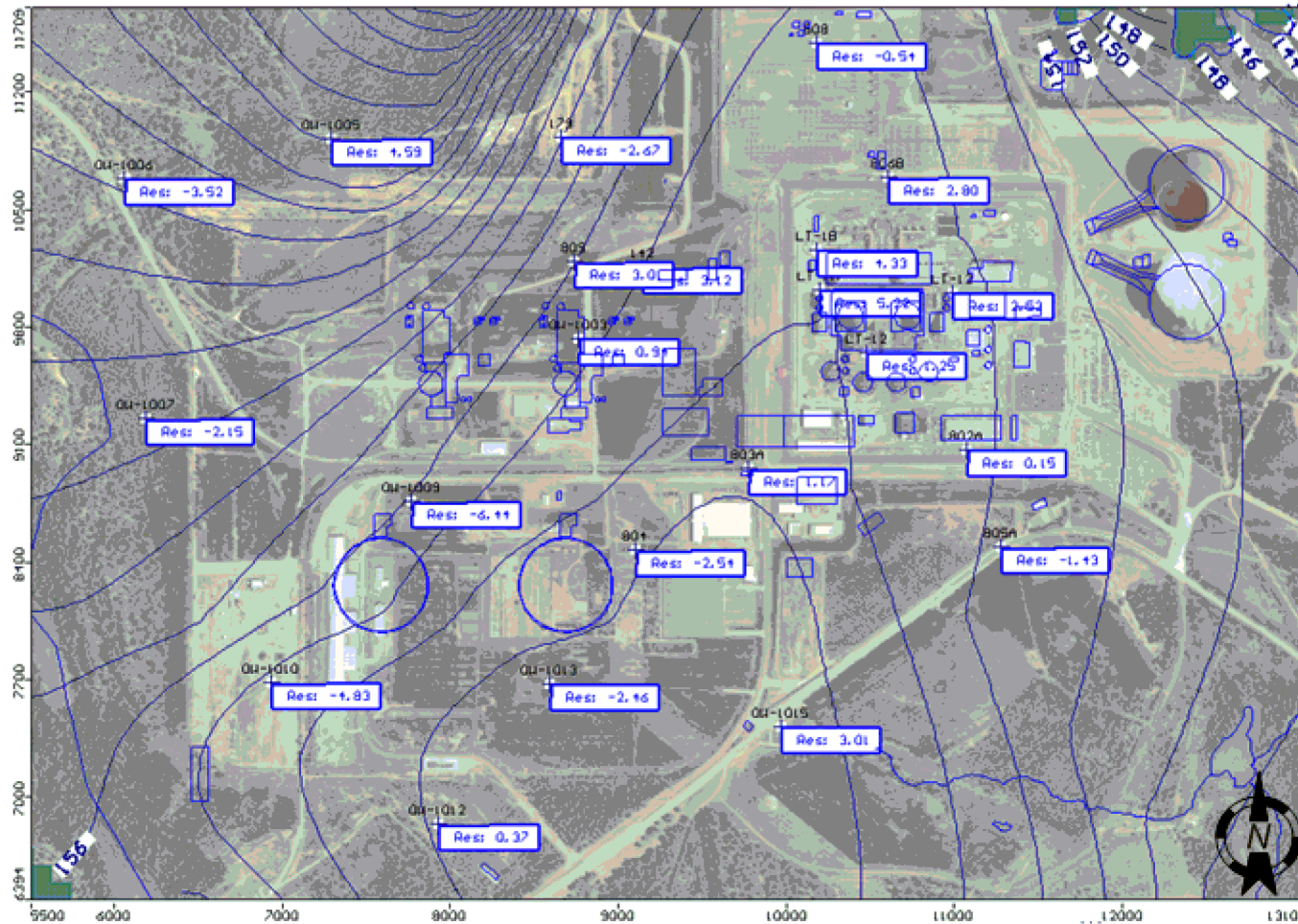
Figure 52:   Estimated Residuals with Model 7 Accounting for the Backfill Material for Units 1 & 2 as a Different Material with Hydraulic Conductivity Equal to 3.3 ft/day
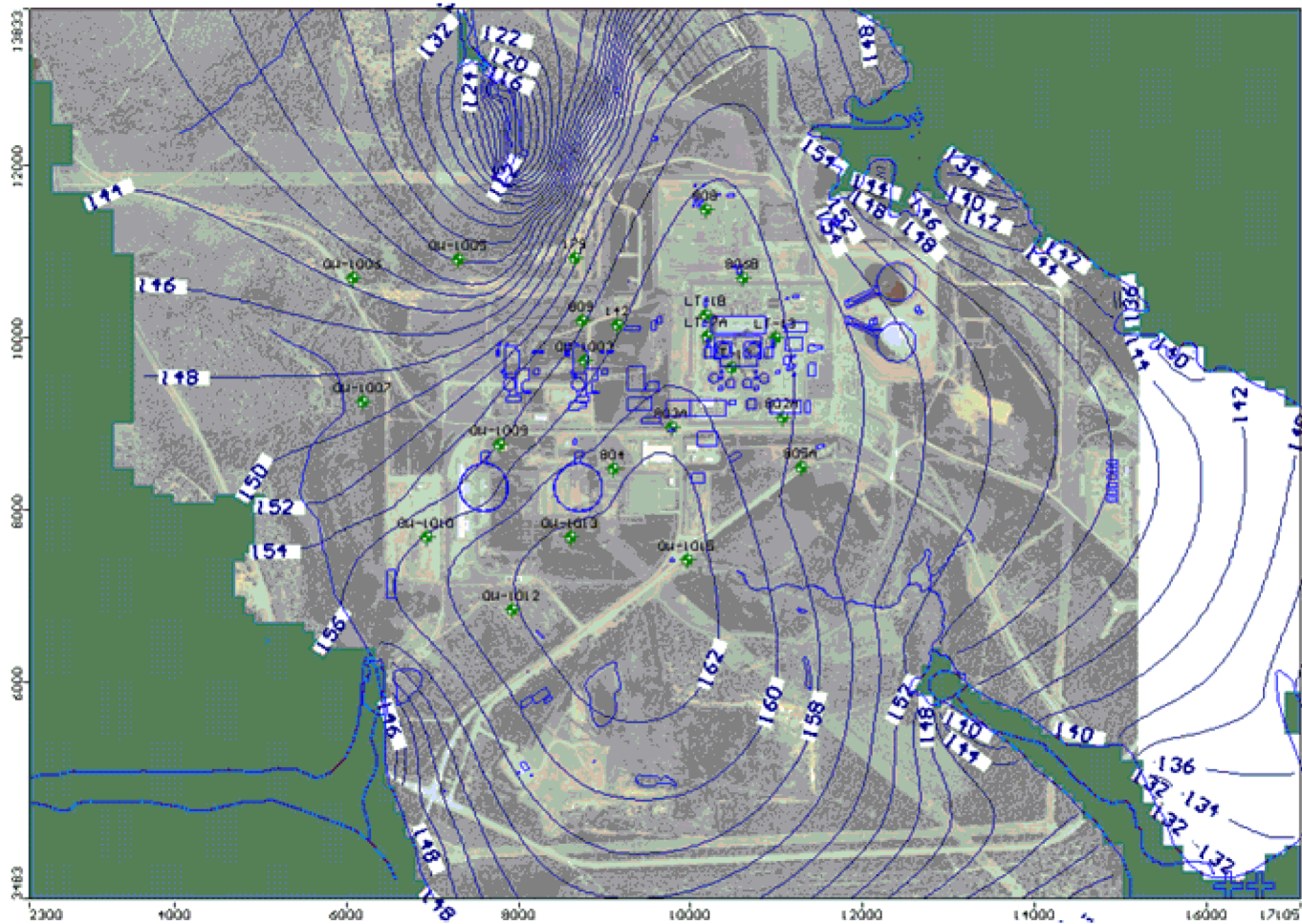
Figure 53: Simulated Water Levels with Model 7 Assuming that the Rate of Groundwater Recharge at the Met Tower Pond is 6 in/yr
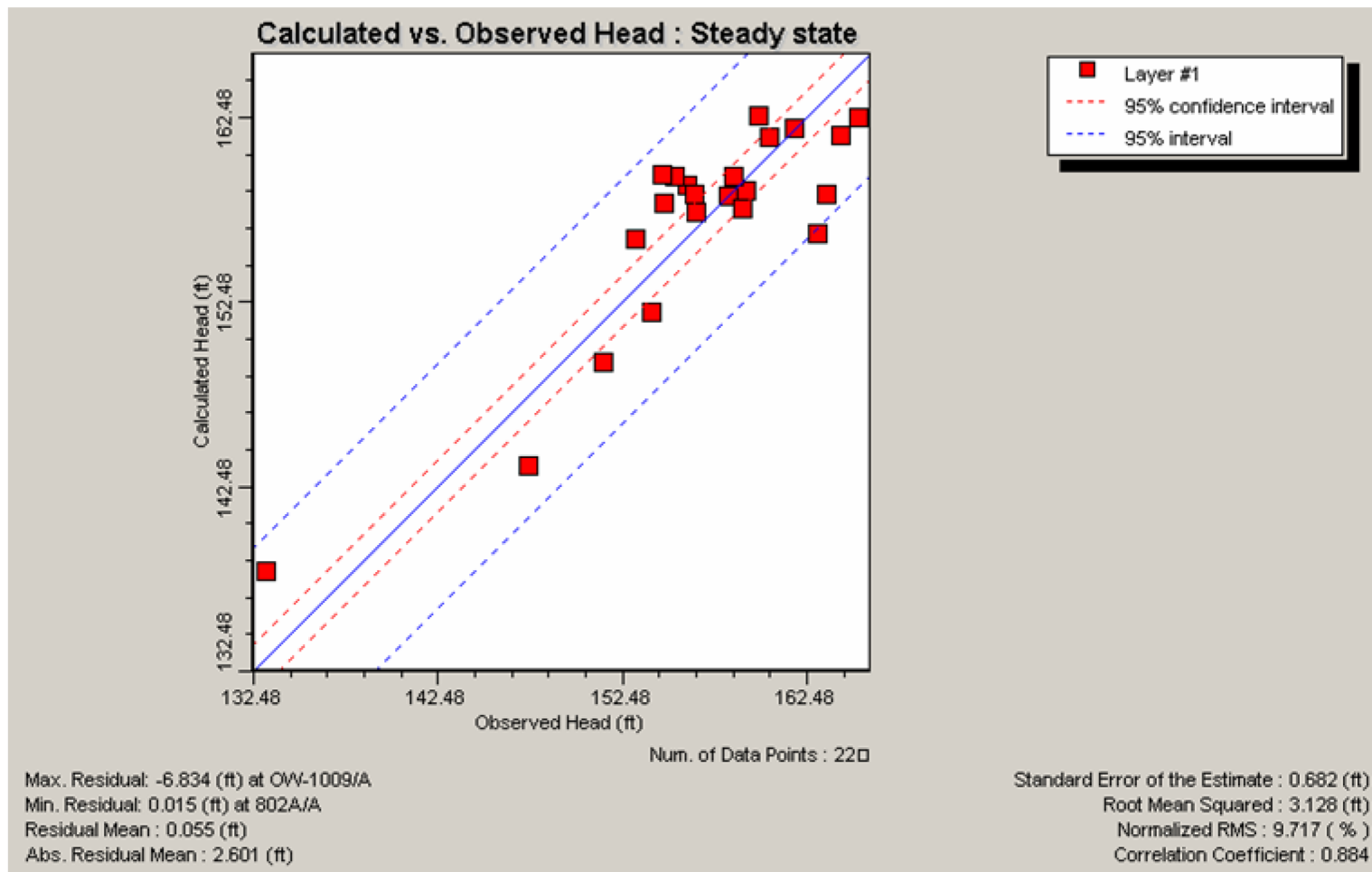
Figure 54: **Simulated Versus Observed Water Levels with Model 7 Assuming that the Rate of Groundwater Recharge at the Met Tower Pond is 6 in/yr**
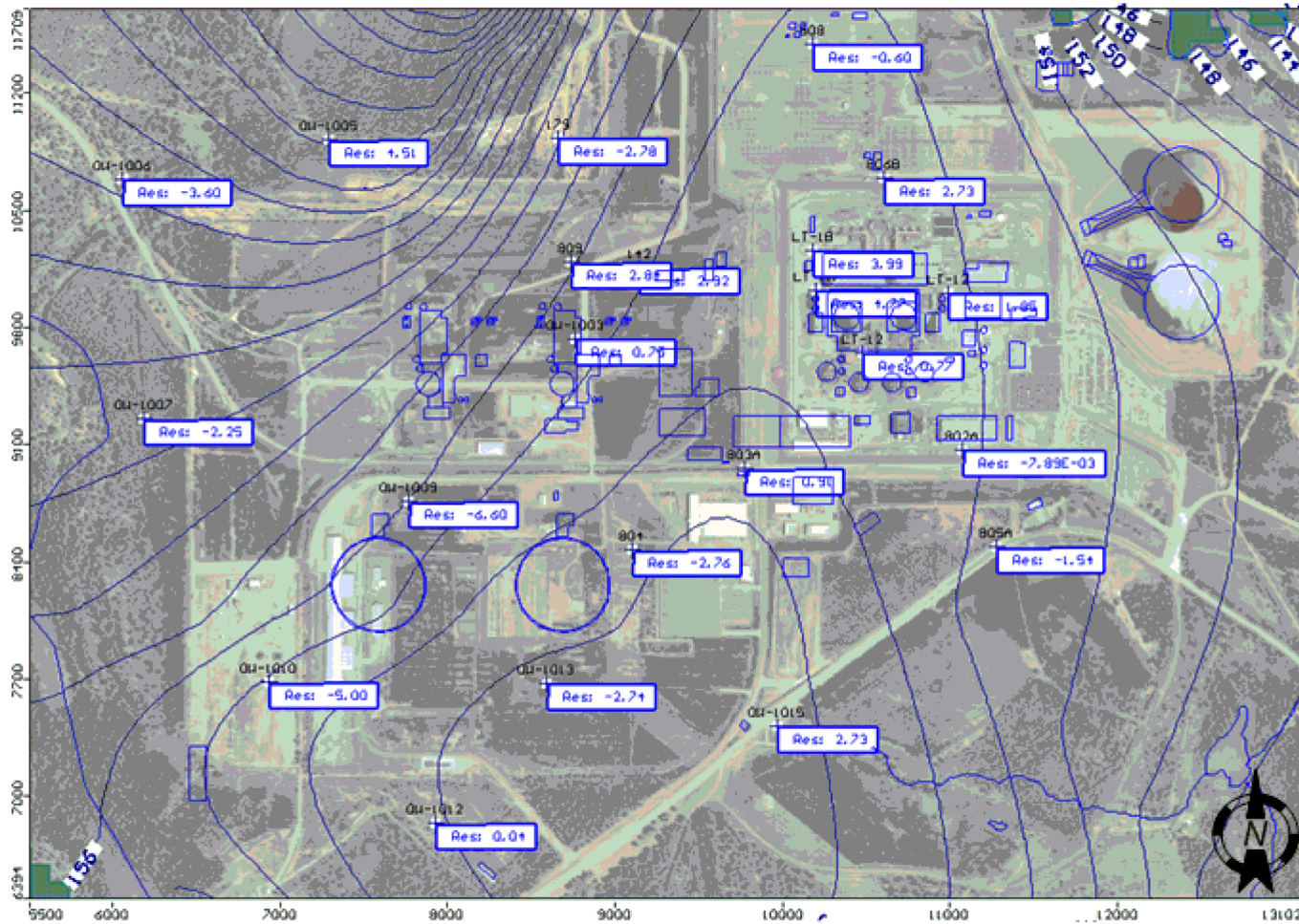
Figure 55: **Estimated Residuals with Model 7 Assuming that the Rate of Groundwater Recharge at the Met Tower Pond is 6 in/yr**
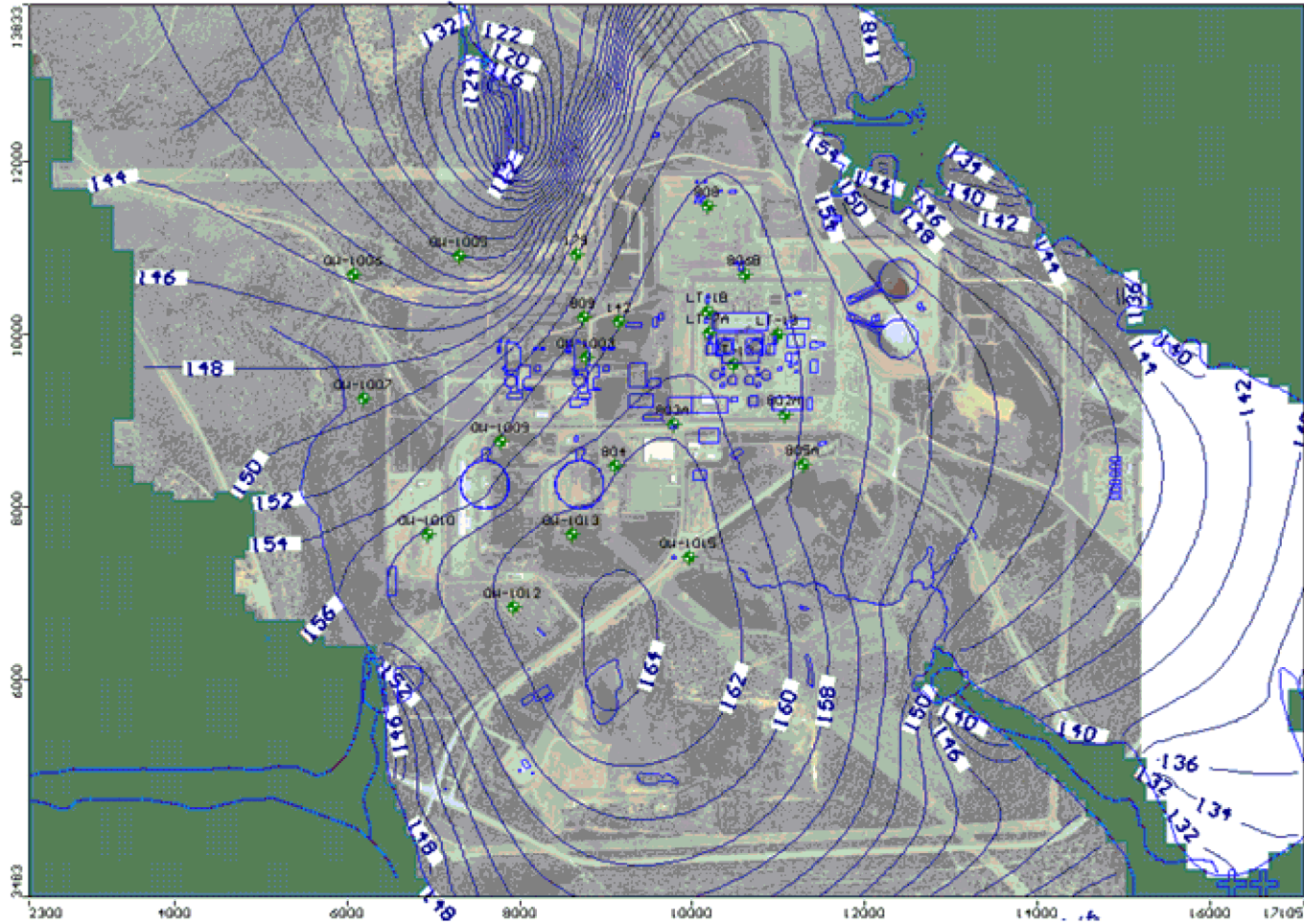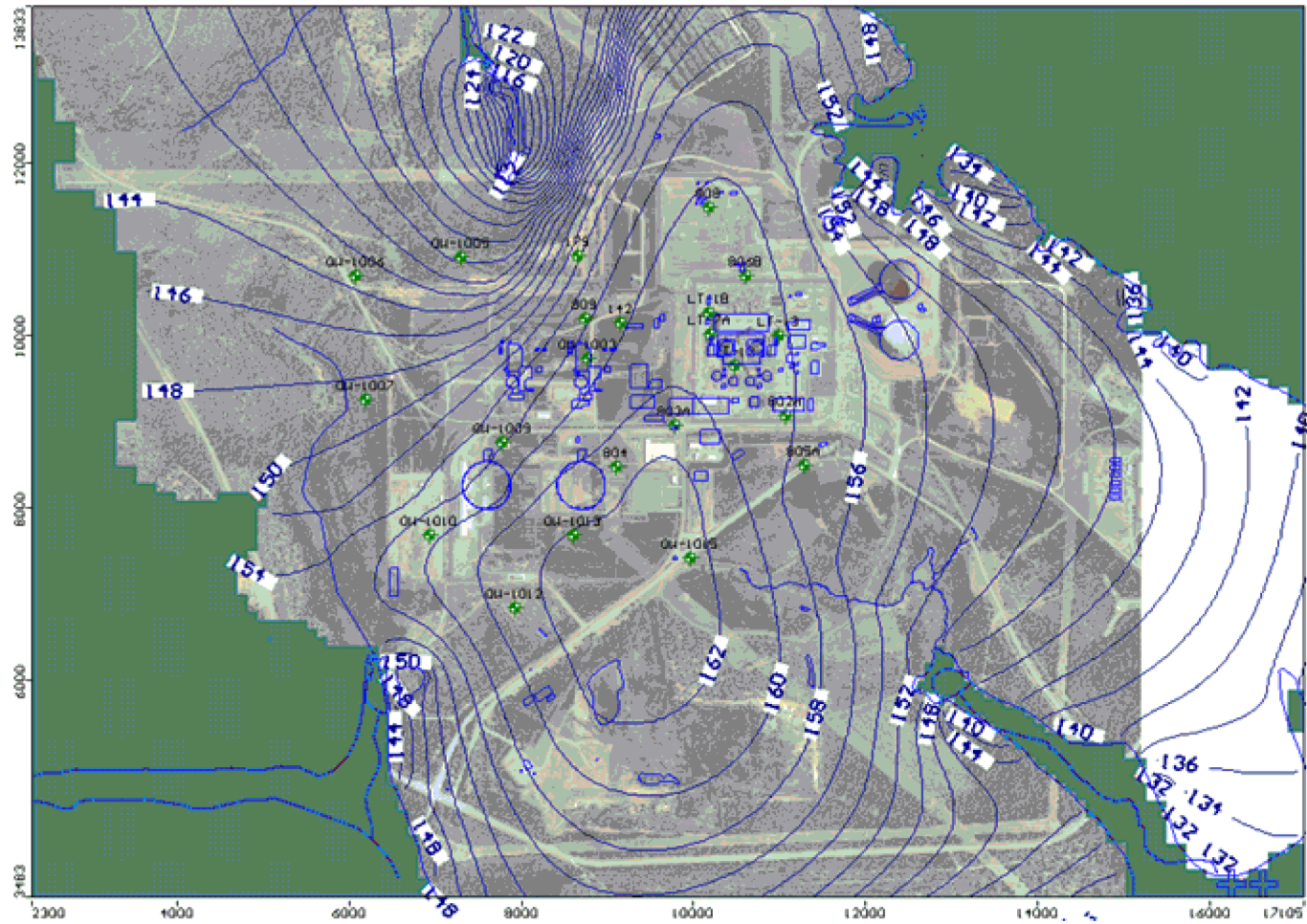
Figure 56:   Simulated Water Levels with Model 7 Assuming that the Rate of Groundwater Recharge at the Met Tower Pond is 40 in/yr
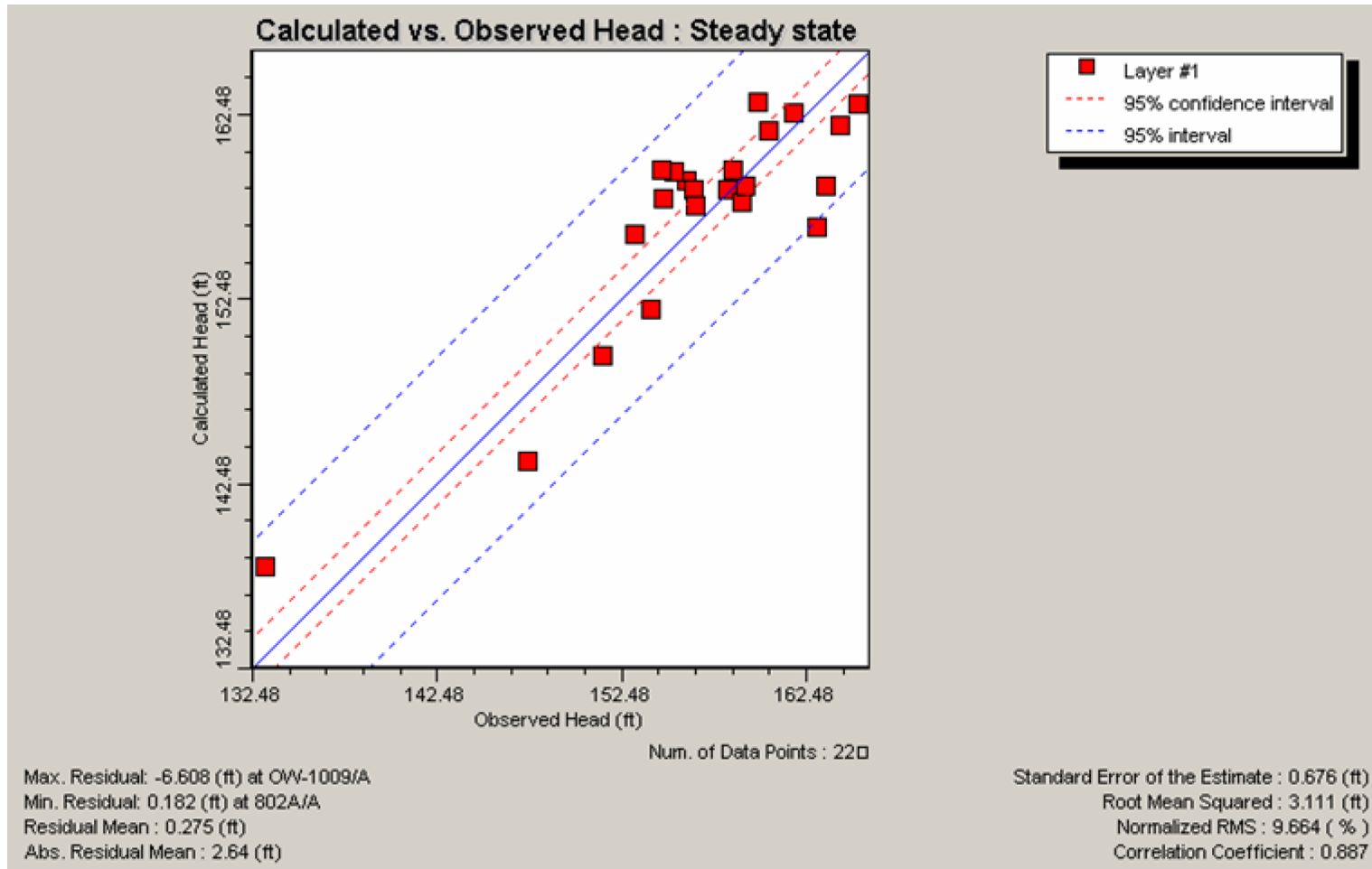
Figure 57: **Simulated Versus Observed Water Levels with Model 7 Assuming that the Rate of Groundwater Recharge at the Met Tower Pond is 40 in/yr**
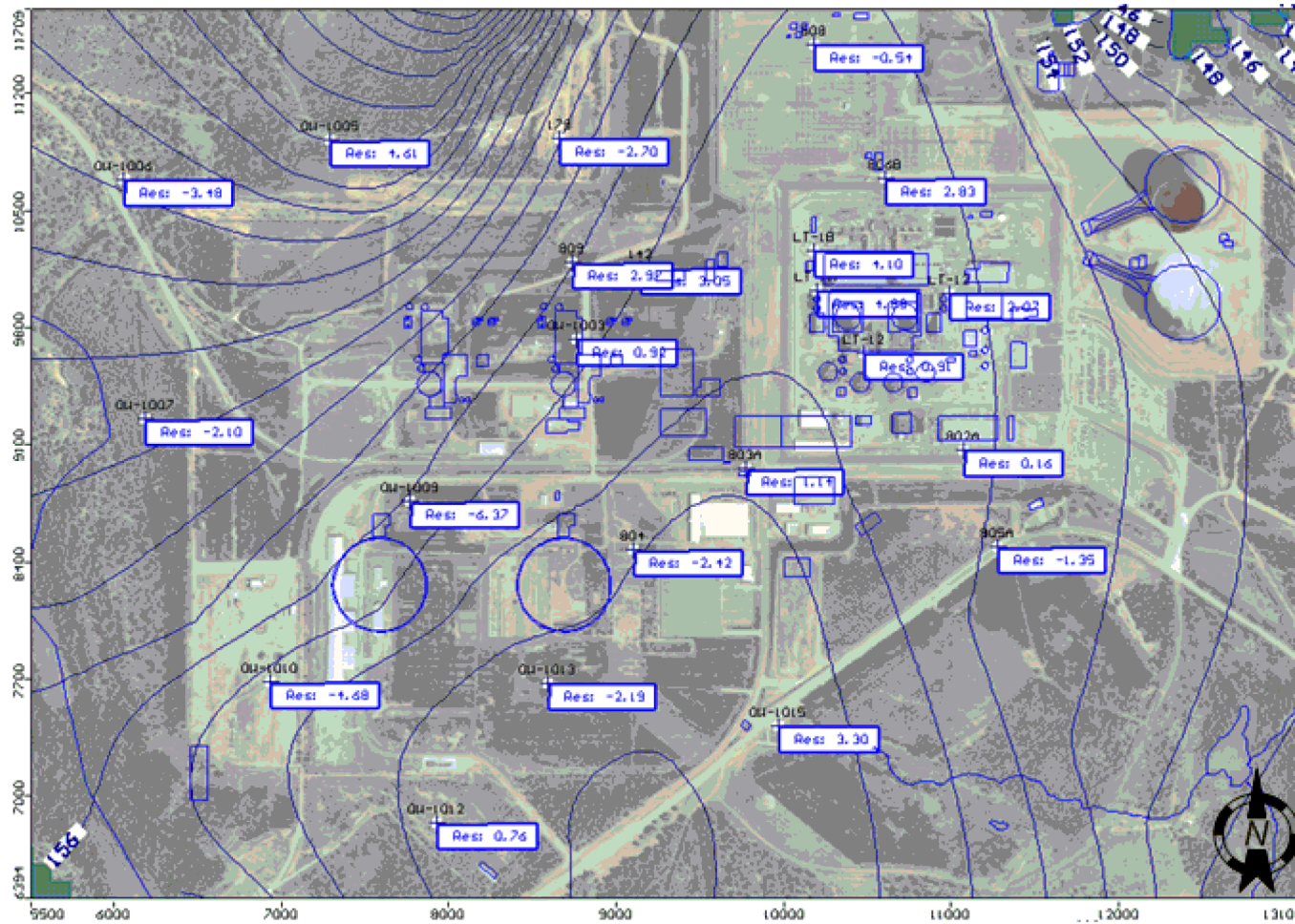
Figure 58:   Estimated Residuals with Model 7 Assuming that the Rate of Groundwater Recharge at the Met Tower Pond is 40 in/yr

Figure 59: **Simulated Water Levels with Model 7 Using a Constant Head Boundary Condition at the Upper Debris Basin 2 (The constant head used was 148.5 ft msl)**
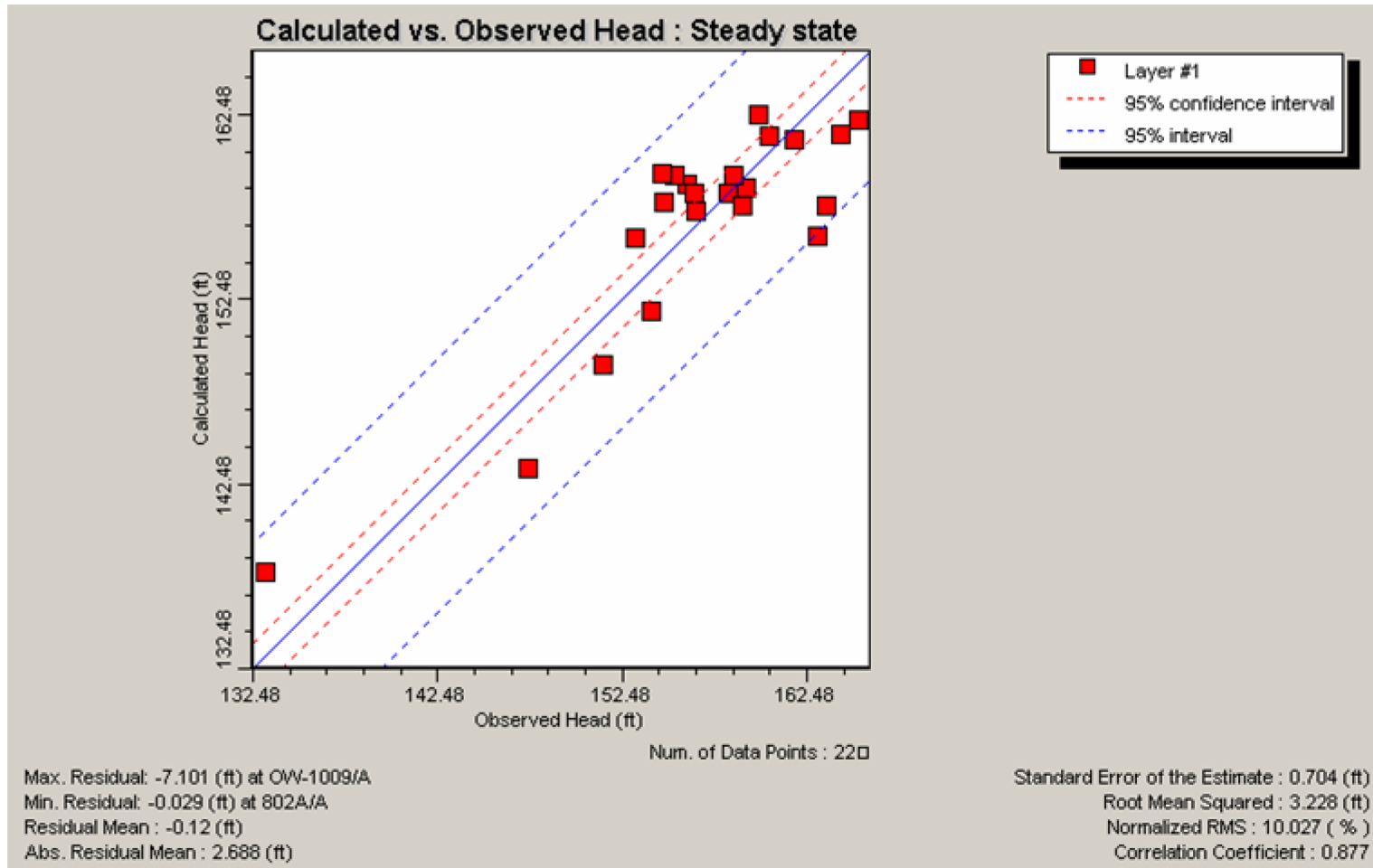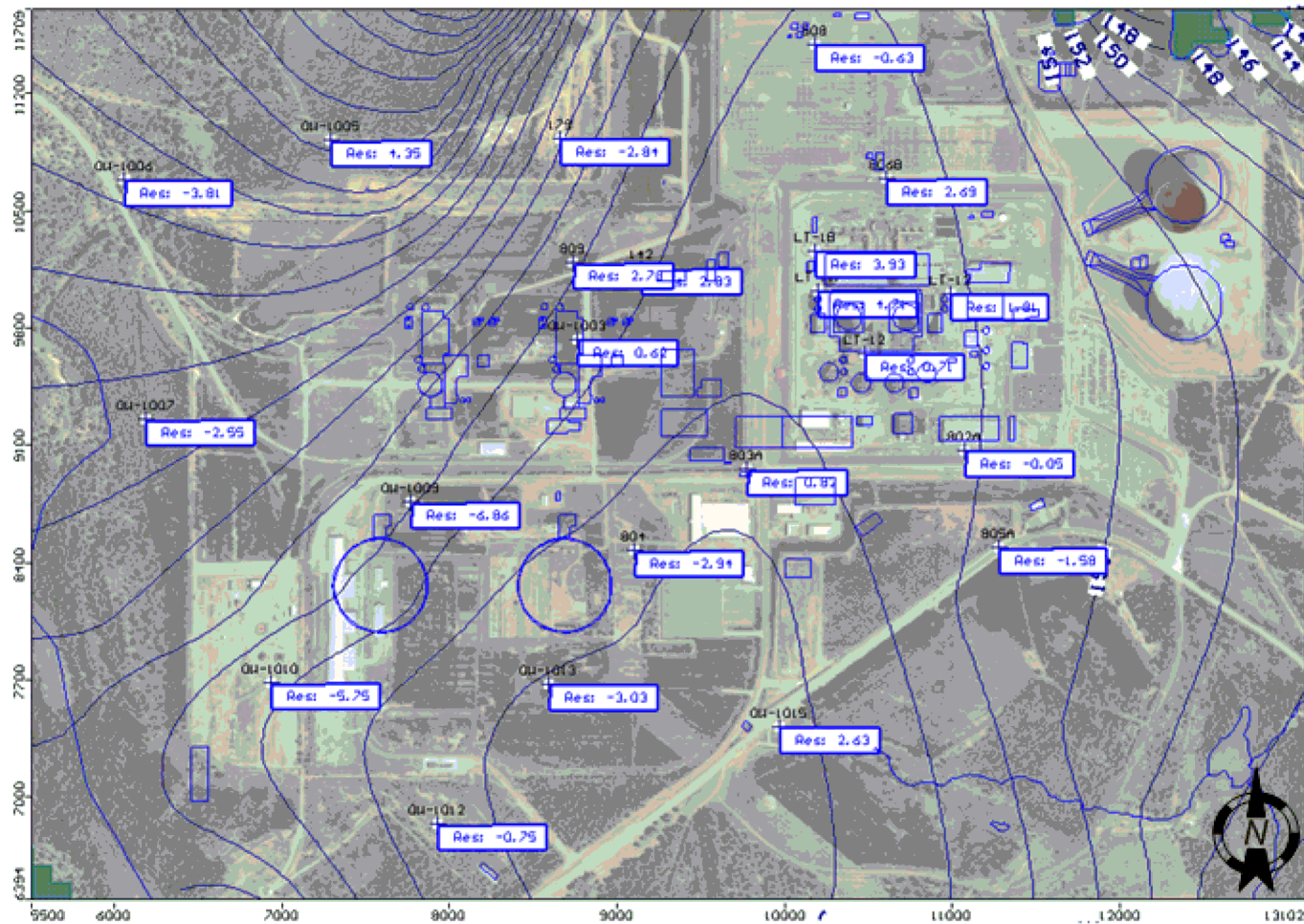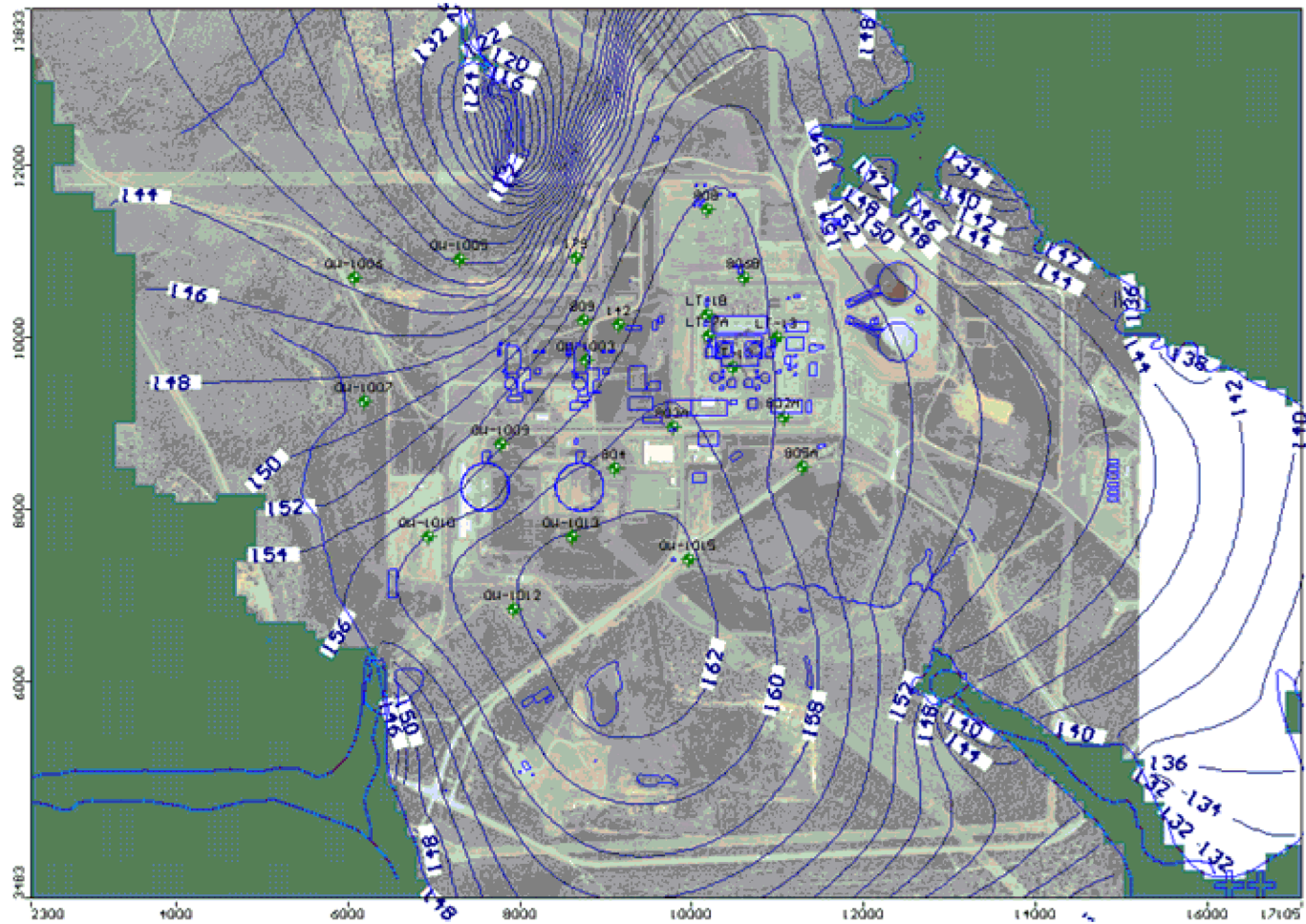
Figure 60: Simulated Versus Observed Water Levels with Model 7 Using a Constant Head Boundary Condition at the Upper Debris Basin 2 (The constant head used was 148.5 ft msl)

Figure 61: Estimated Residuals with Model 7 Using a Constant Head Boundary Condition at the Upper Debris Basin 2 (The constant head used was 148.5 ft msl)

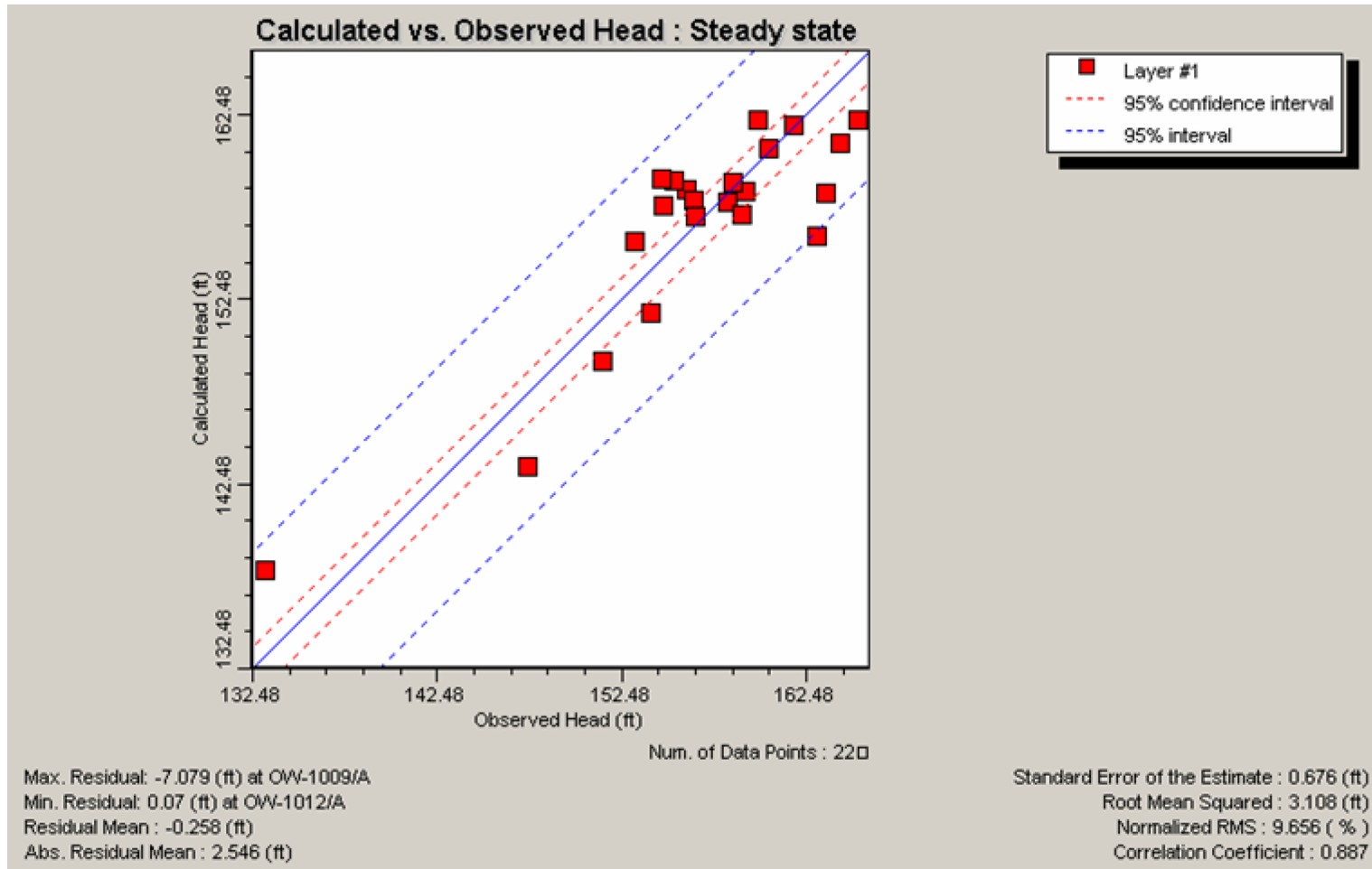**Figure 62:   Simulated Water Levels with Model 7 Using Additional Paved Areas Around Units 1 & 2 and 3 & 4**

Figure 63: Simulated Versus Observed Water Levels with Model 7 Using Additional Paved Areas Around Units 1 & 2 and 3 & 4
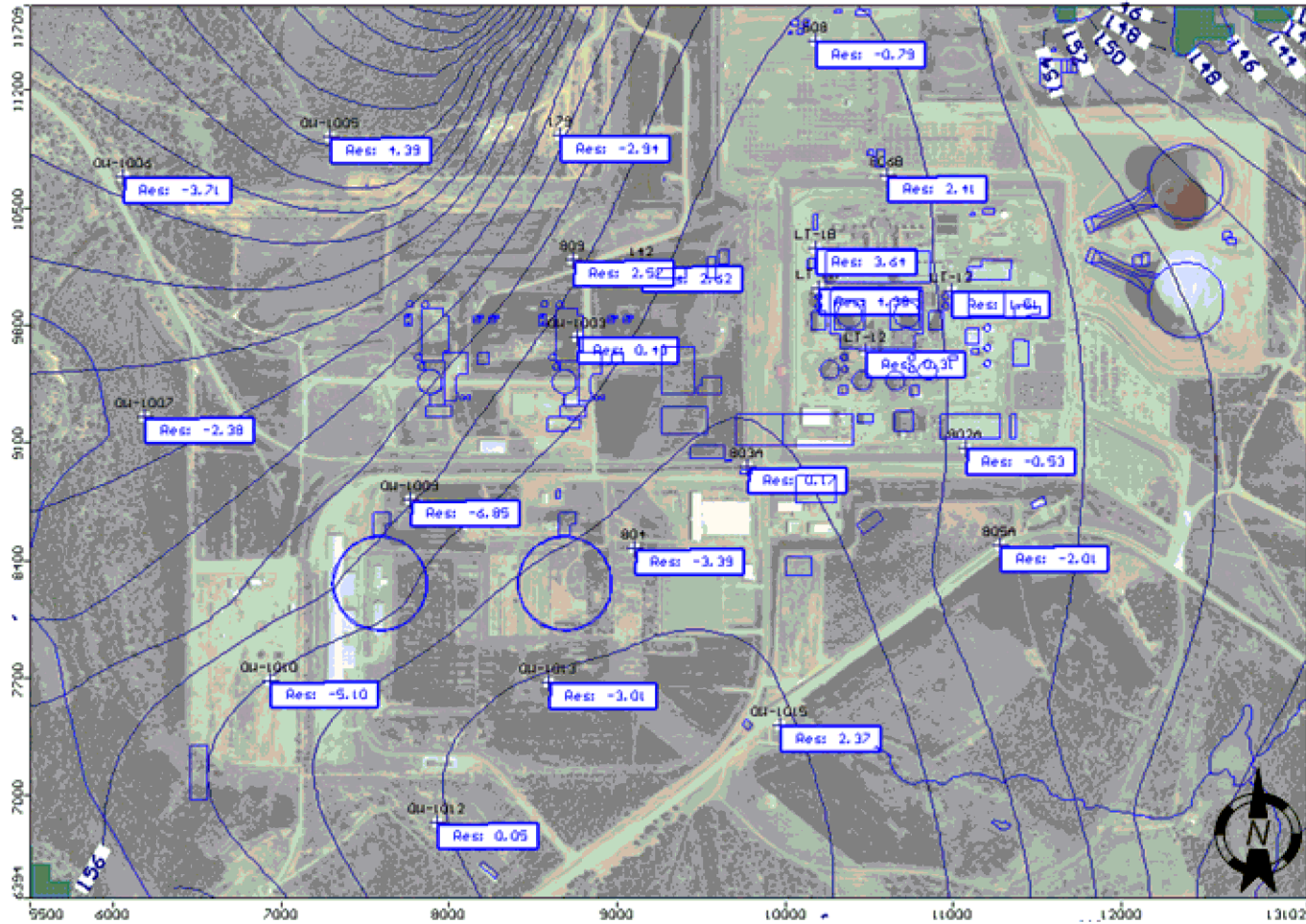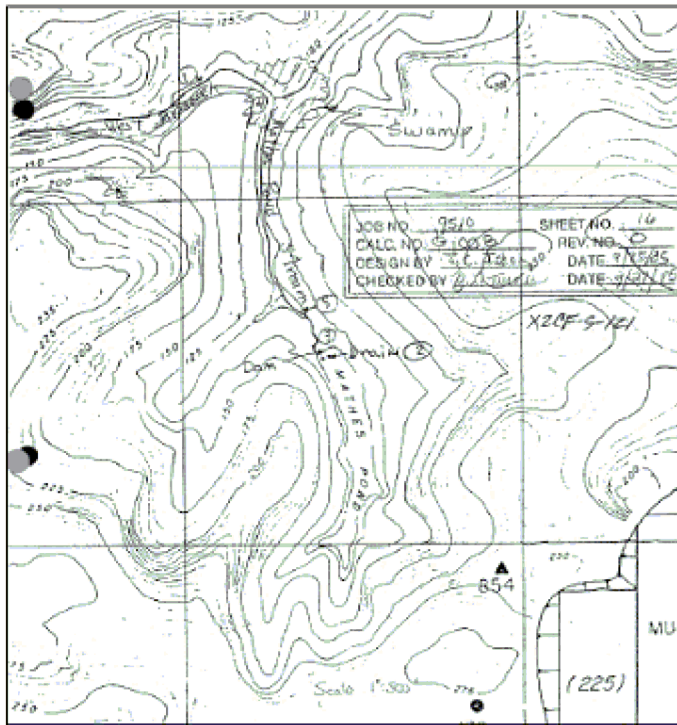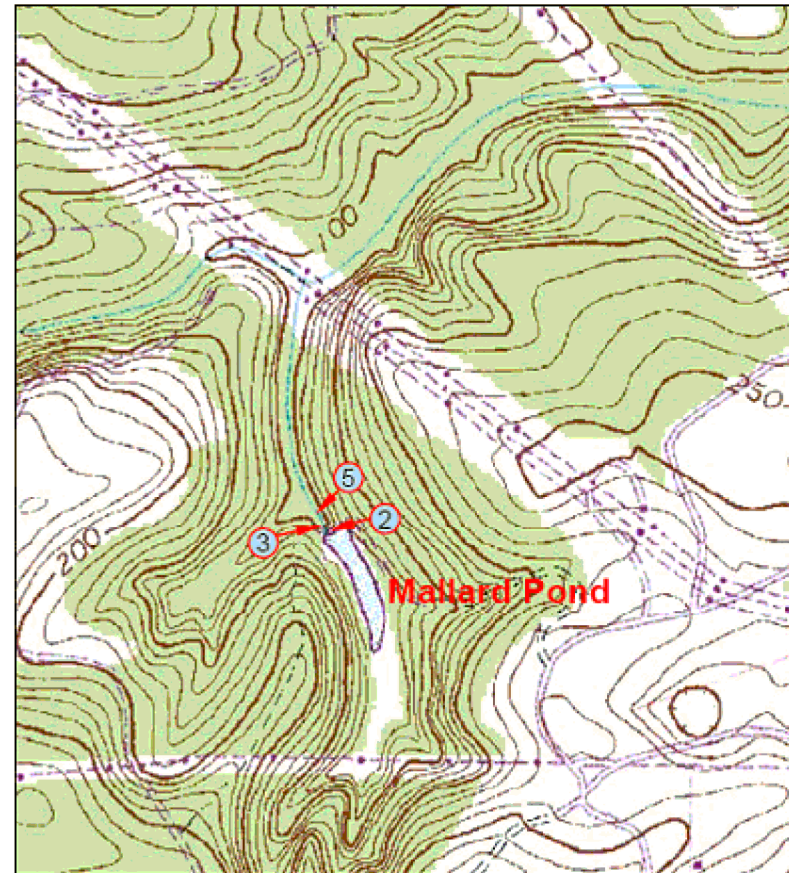
Figure 64:   Estimated Residuals with Model 7 Using Additional Paved Areas Around Units 1 & 2 and 3 & 4

(a) Obtained from Ref. 11

(b) Stations from Ref. 11 Superimposed on USGS Map

Figure 65: Location of Stream Flow Measurement Stations at Mallard Pond
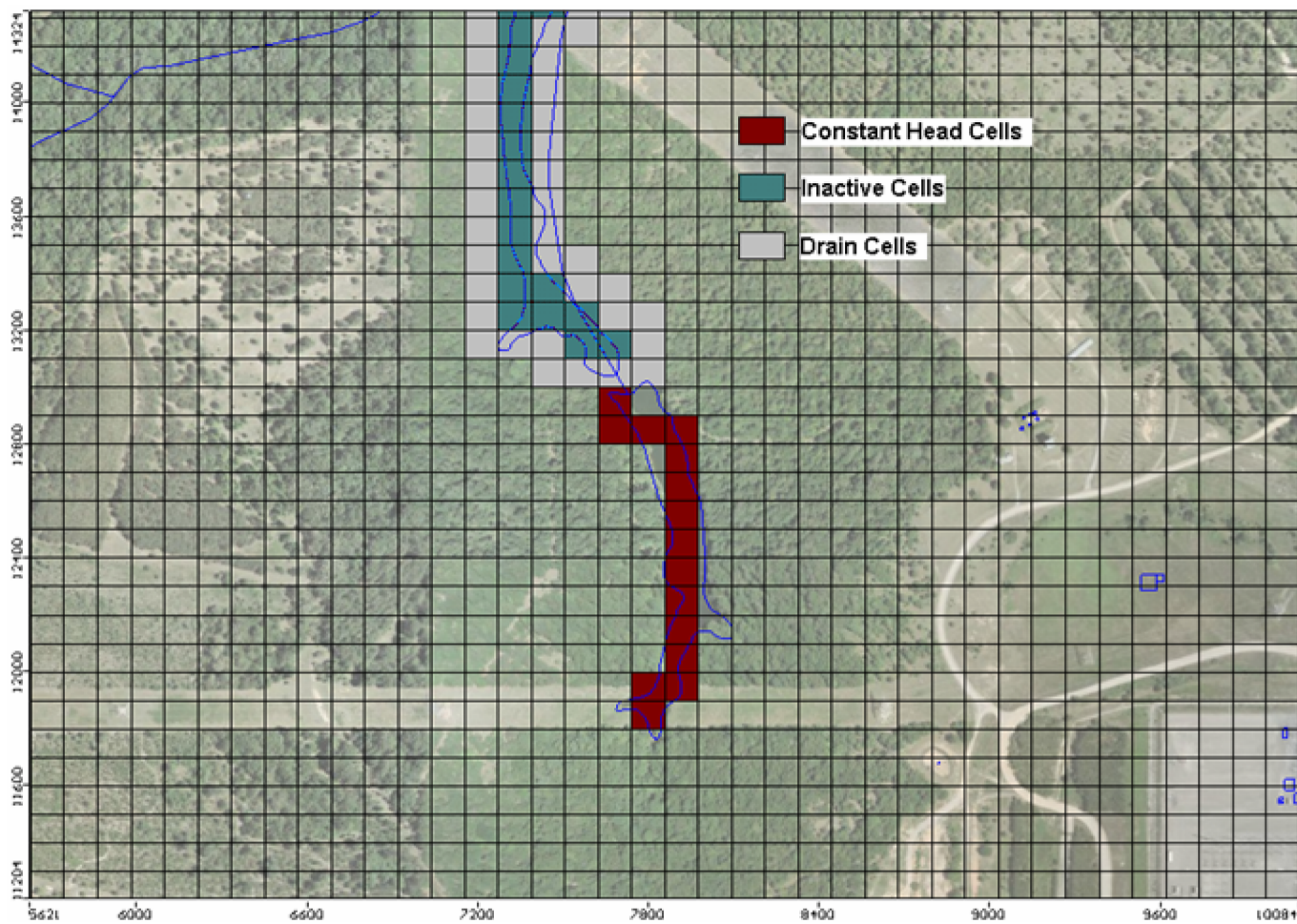
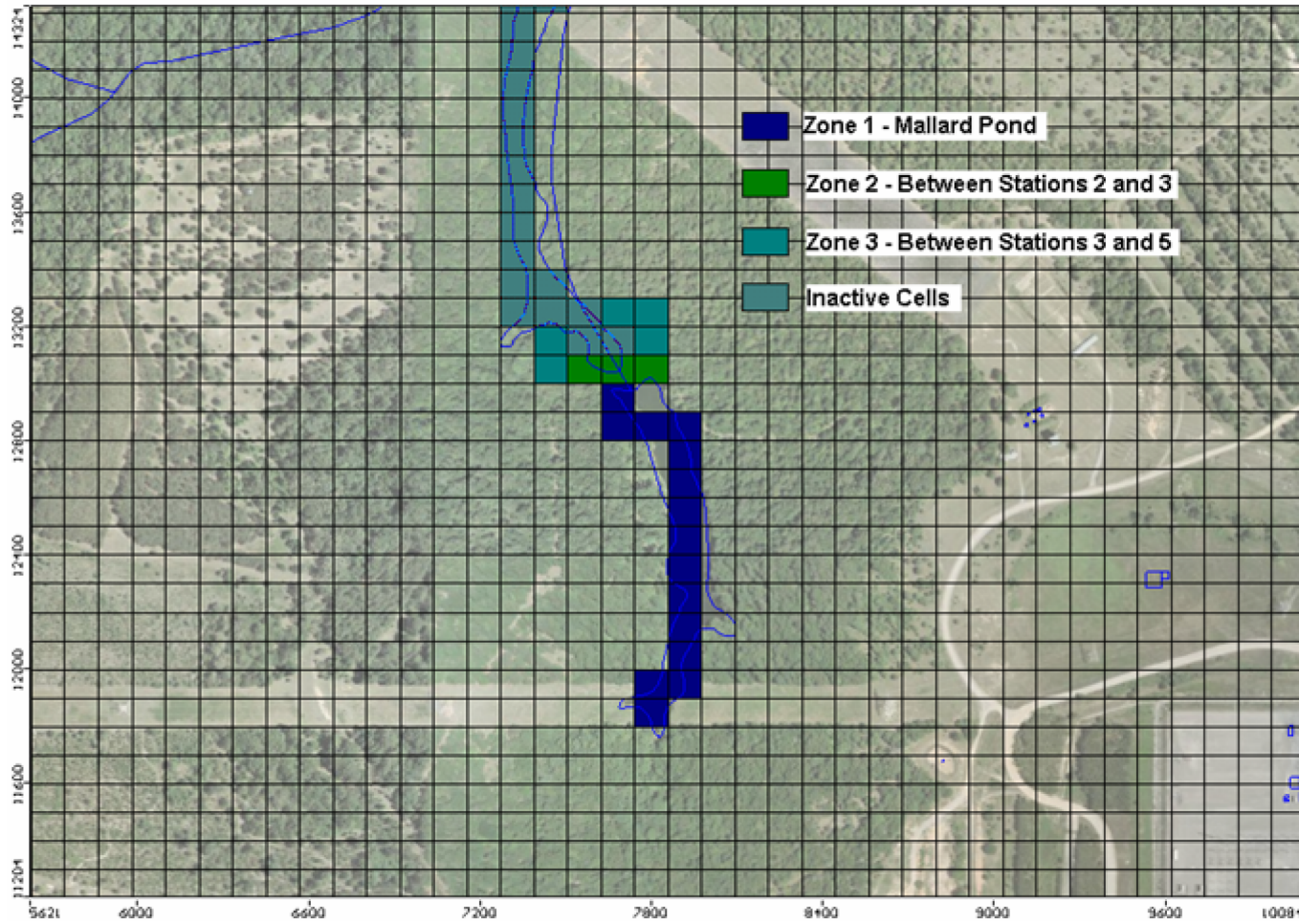**Figure 66: Representation of Mallard Pond and Area in Groundwater Model**

Figure 67:   Zone Budget Representation of Mallard Pond and Area in Groundwater Model
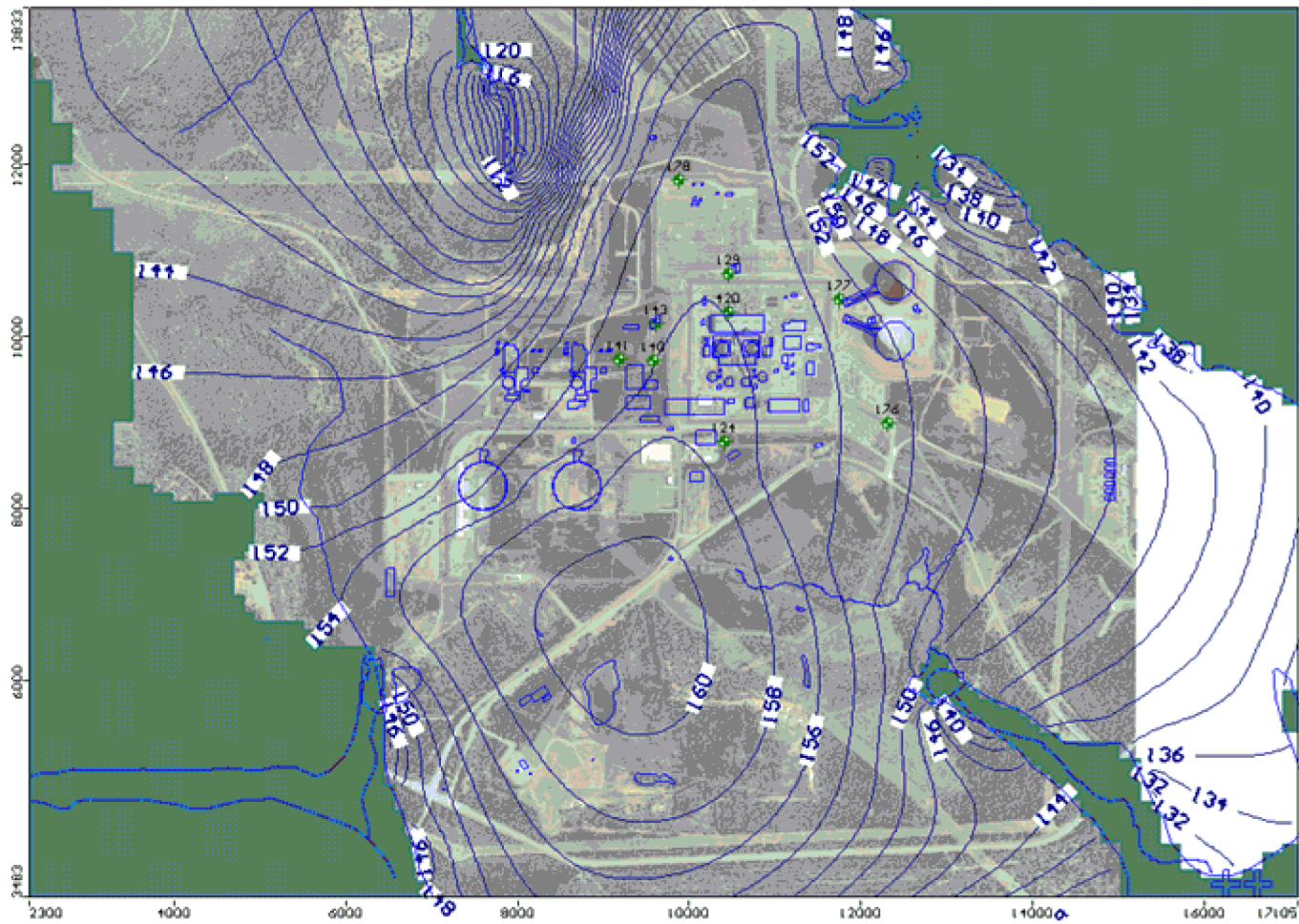
Figure 68:   Simulated Water Levels for 1971 Groundwater Model

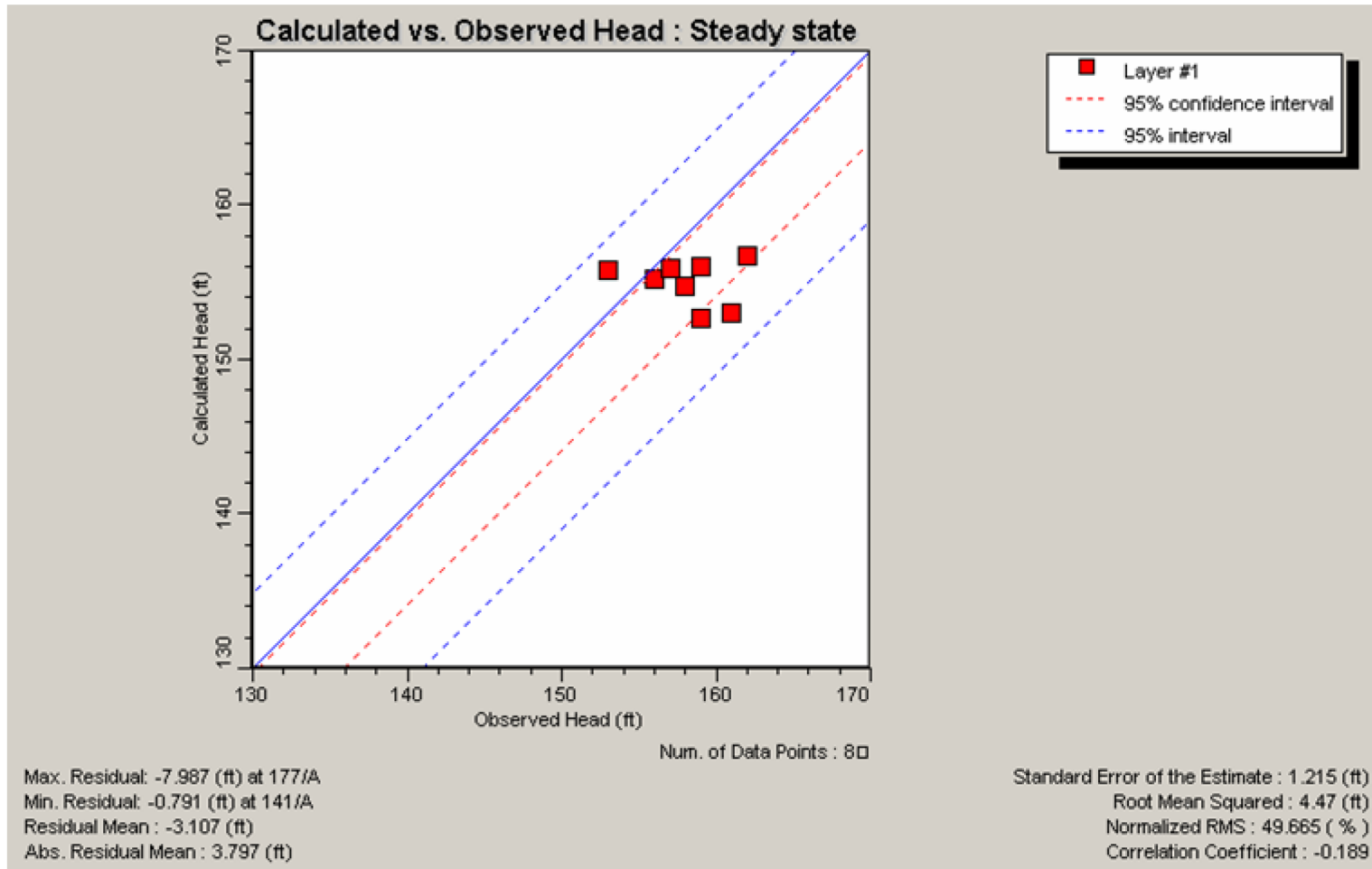**Figure 69: Simulated Vs. Observed Water Levels for 1971 Groundwater Model**
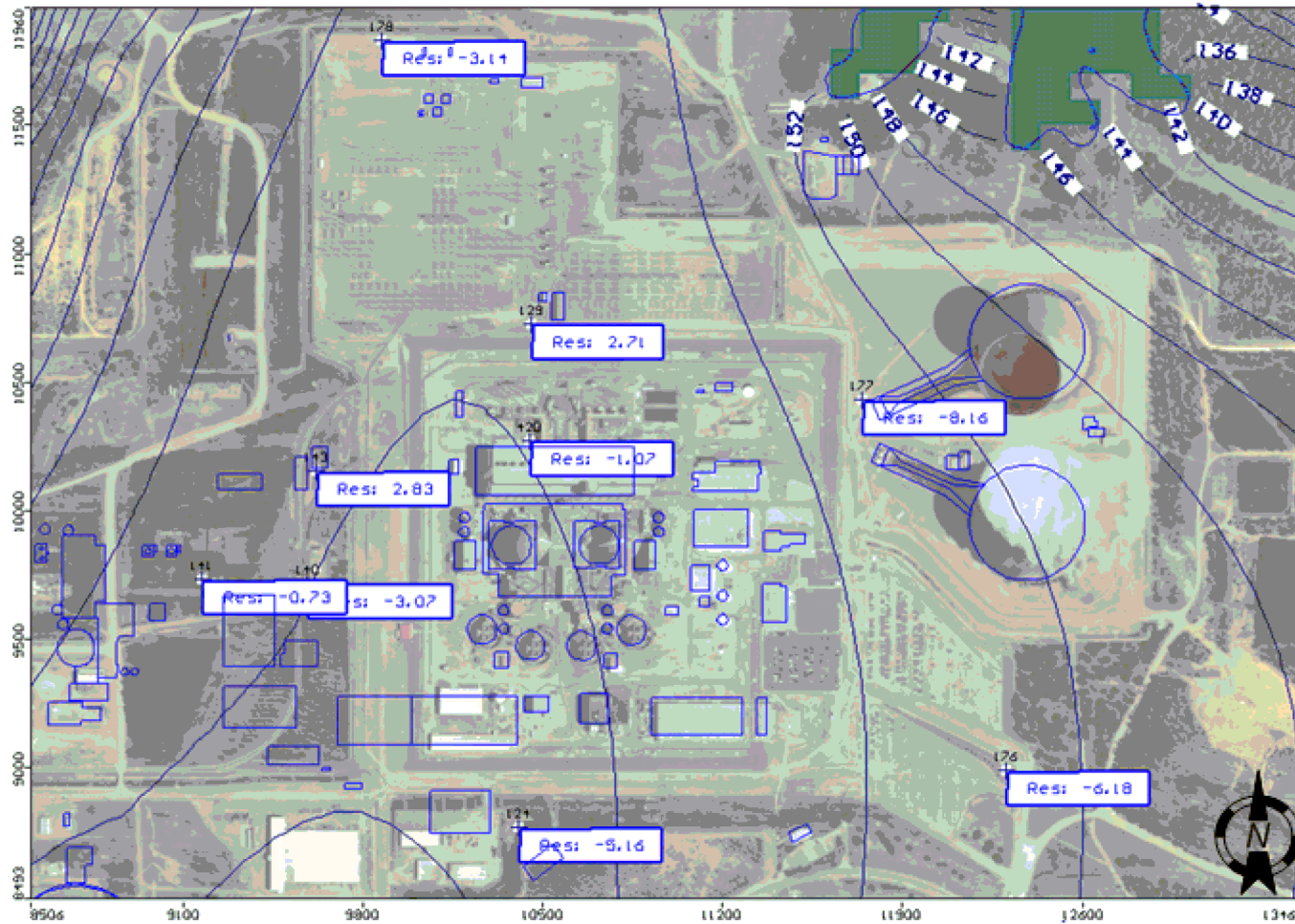
Figure 70:   Estimated Residuals for 1971 Groundwater Model
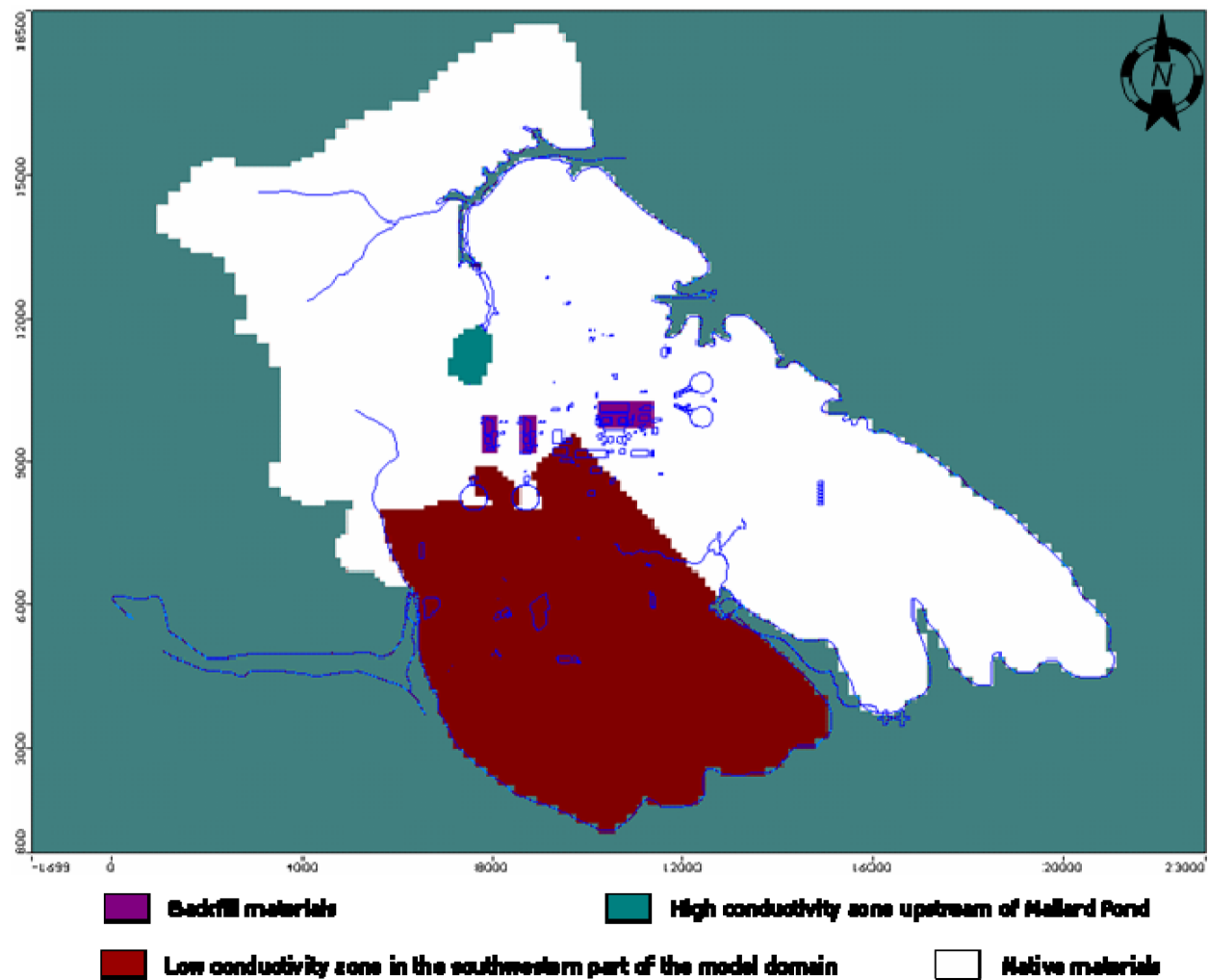
Figure 71: **Hydraulic Conductivity Zones Used in Model 7 to Evaluate Post-Construction Conditions**
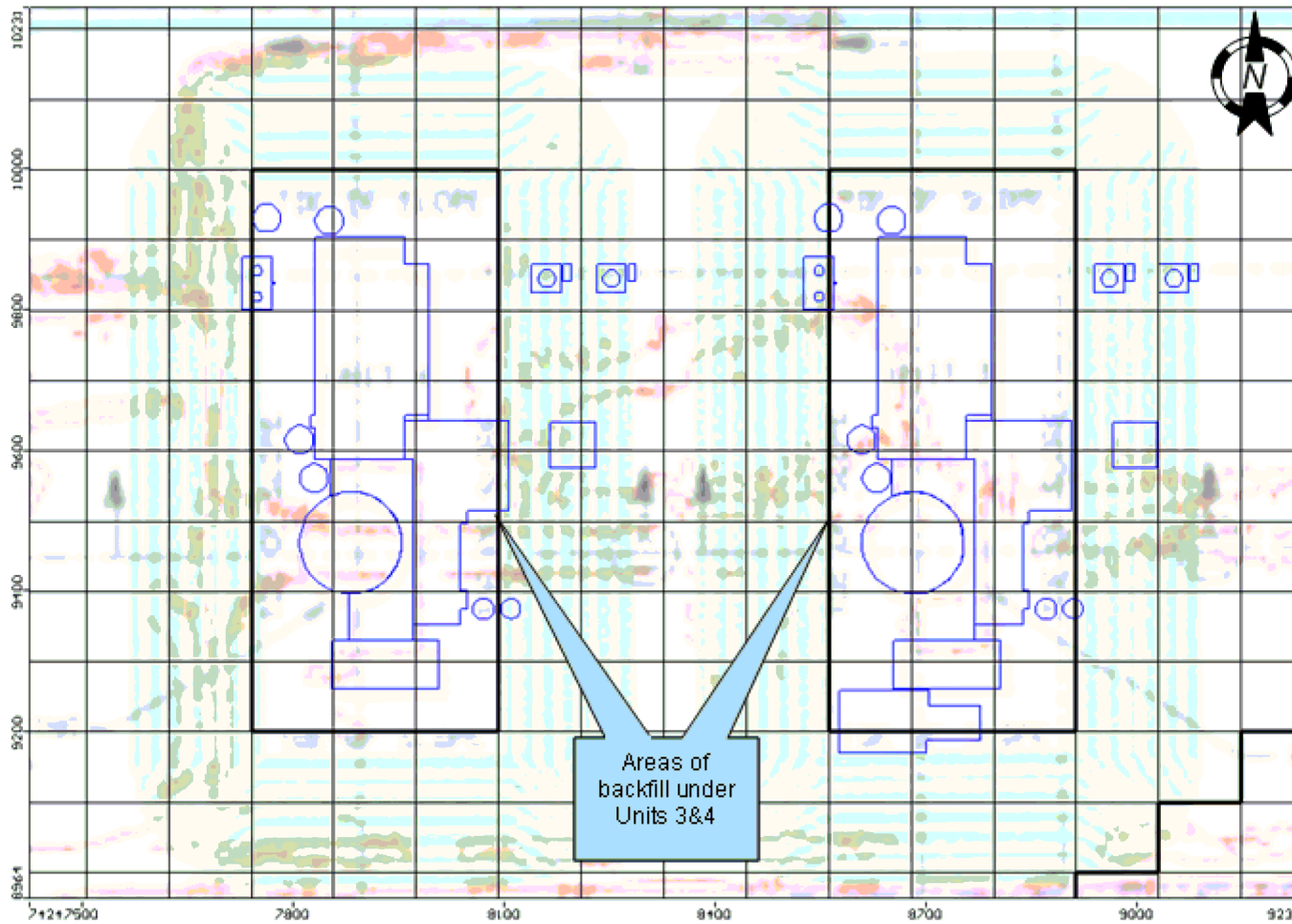
Figure 72:   Excavation Plan Overlaid onto Model Grid to Show Areas of Backfill Around Units 3 & 4
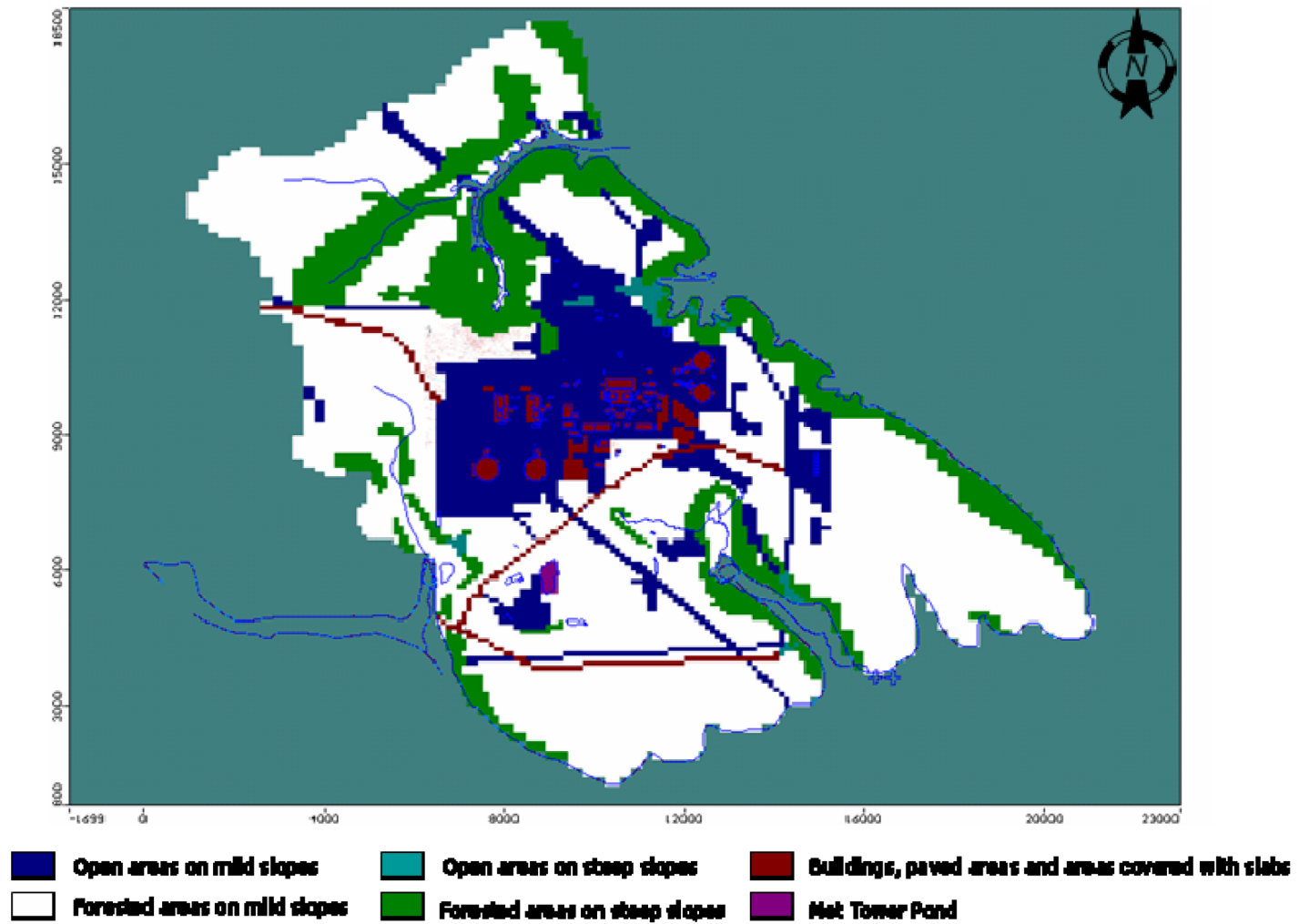
Figure 73: Recharge Zones Used to Evaluate Post-Construction Conditions
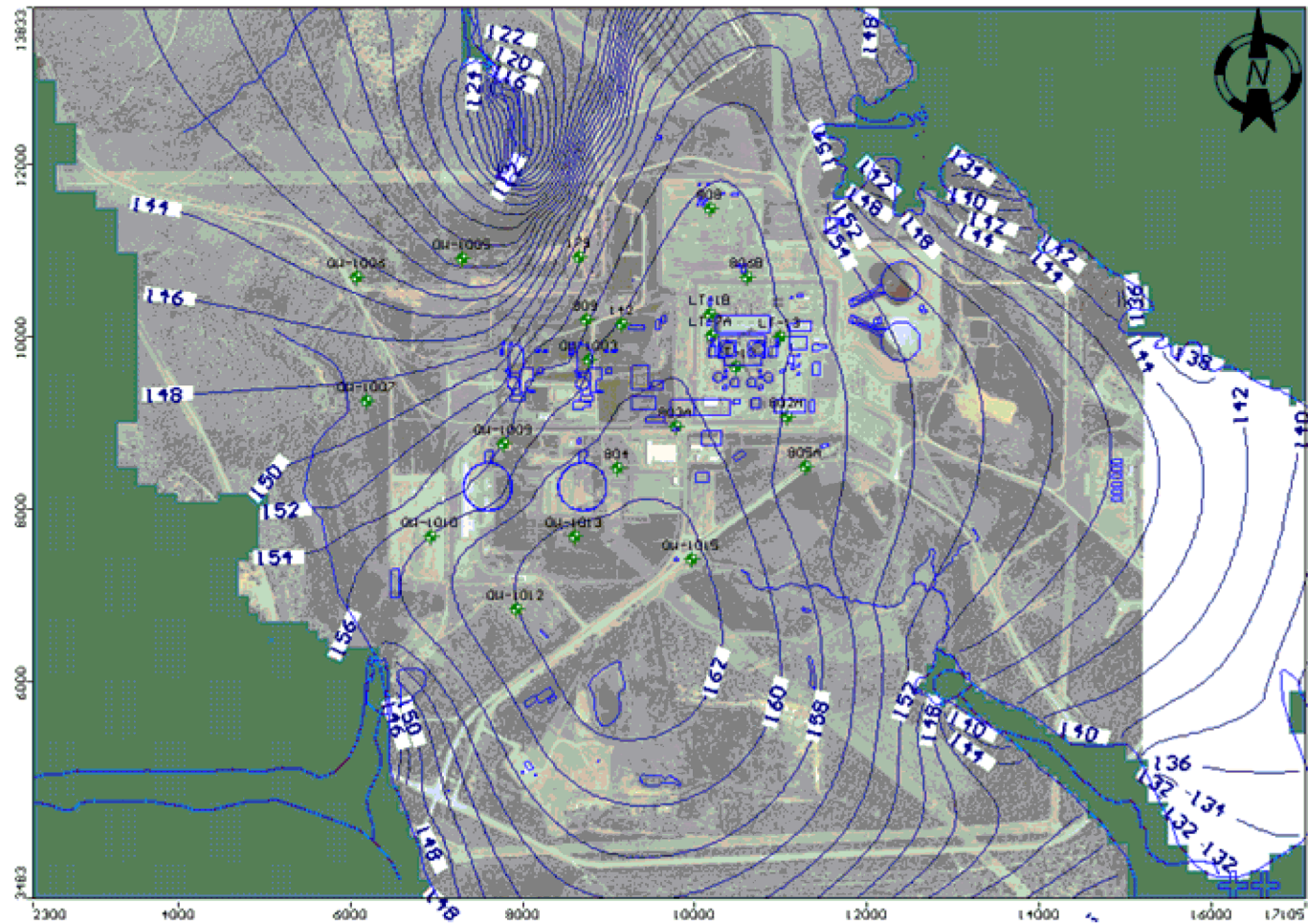
Figure 74:   Simulated Water Levels for Post-Construction Conditions Obtained with Model 7
(K_1=32; K_2=100; K_3=8; K_{fill}=3.3 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
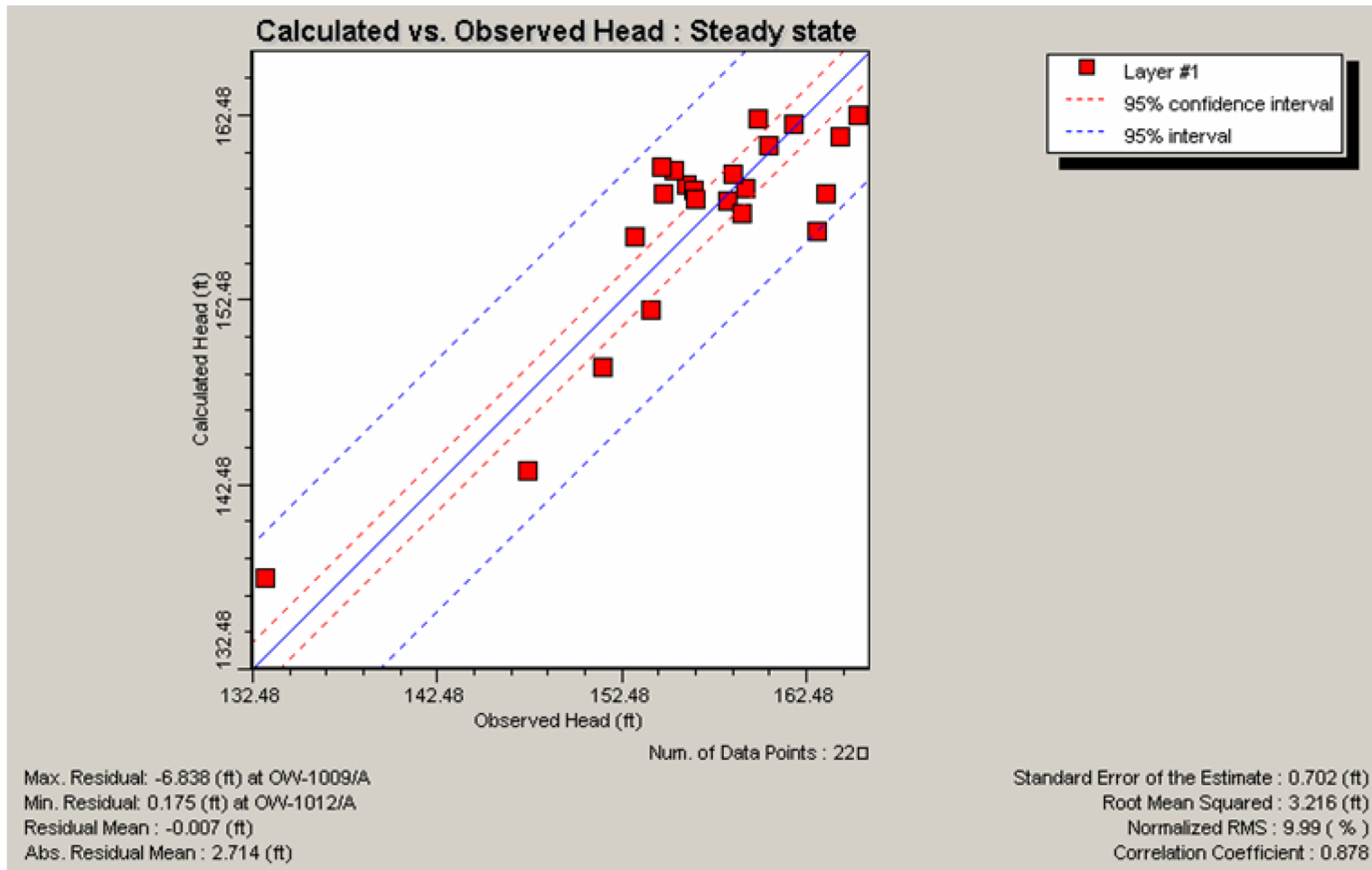
Figure 75:   Simulated Water Levels with Model 7 Accounting for the Backfill Material for Units 1 & 2 as a Different Material with Hydraulic Conductivity Equal to 3.3 ft/day
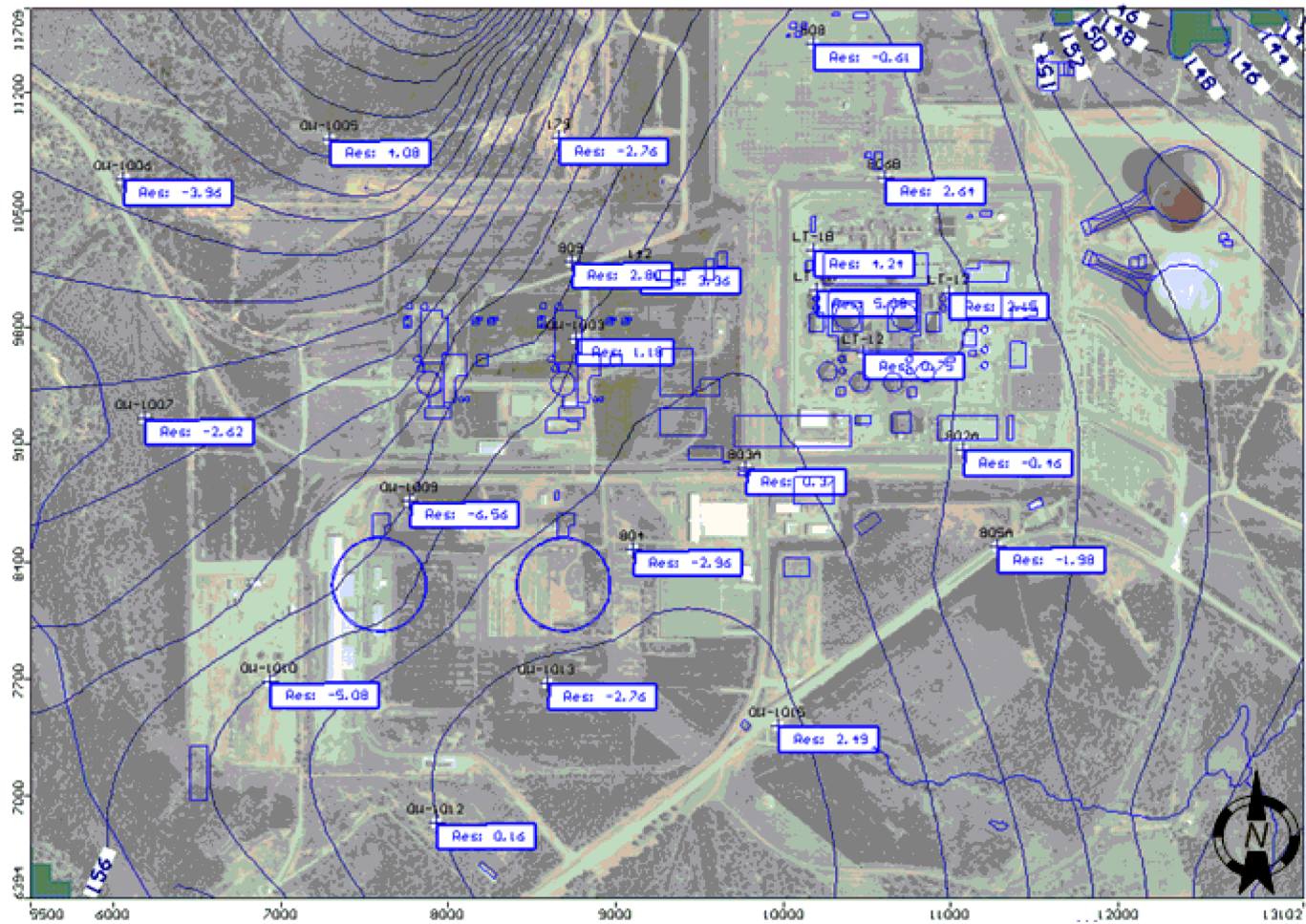
Figure 76:   Simulated Water Levels with Model 7 Accounting for the Backfill Material for Units 1 & 2 as a Different Material with Hydraulic Conductivity Equal to 3.3 ft/day

Figure 77:   Simulated Water Levels and Particle Tracking for Post-Construction Conditions Obtained with Model 7 (K_1=32; K_2=100; K_3=8; K_{fill}=3.3 ft/day; R_1=10; R_2=6; R_3=6; R_4=4; R_5=0 in/yr)
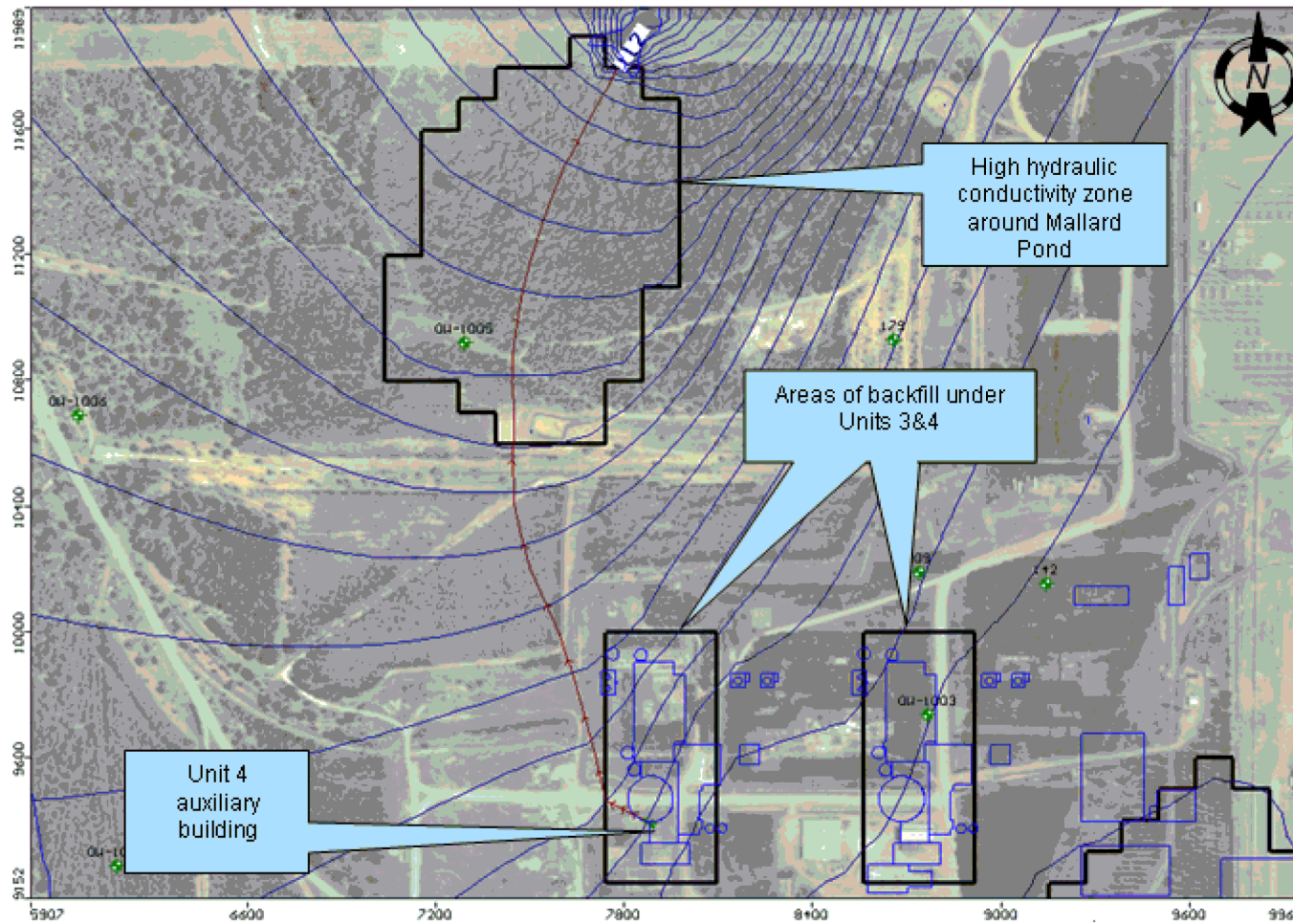
Figure 78: Pathway of Particle Released from Auxiliary Building of Unit 4

ATTACHMENT 1

Relevant Open Items

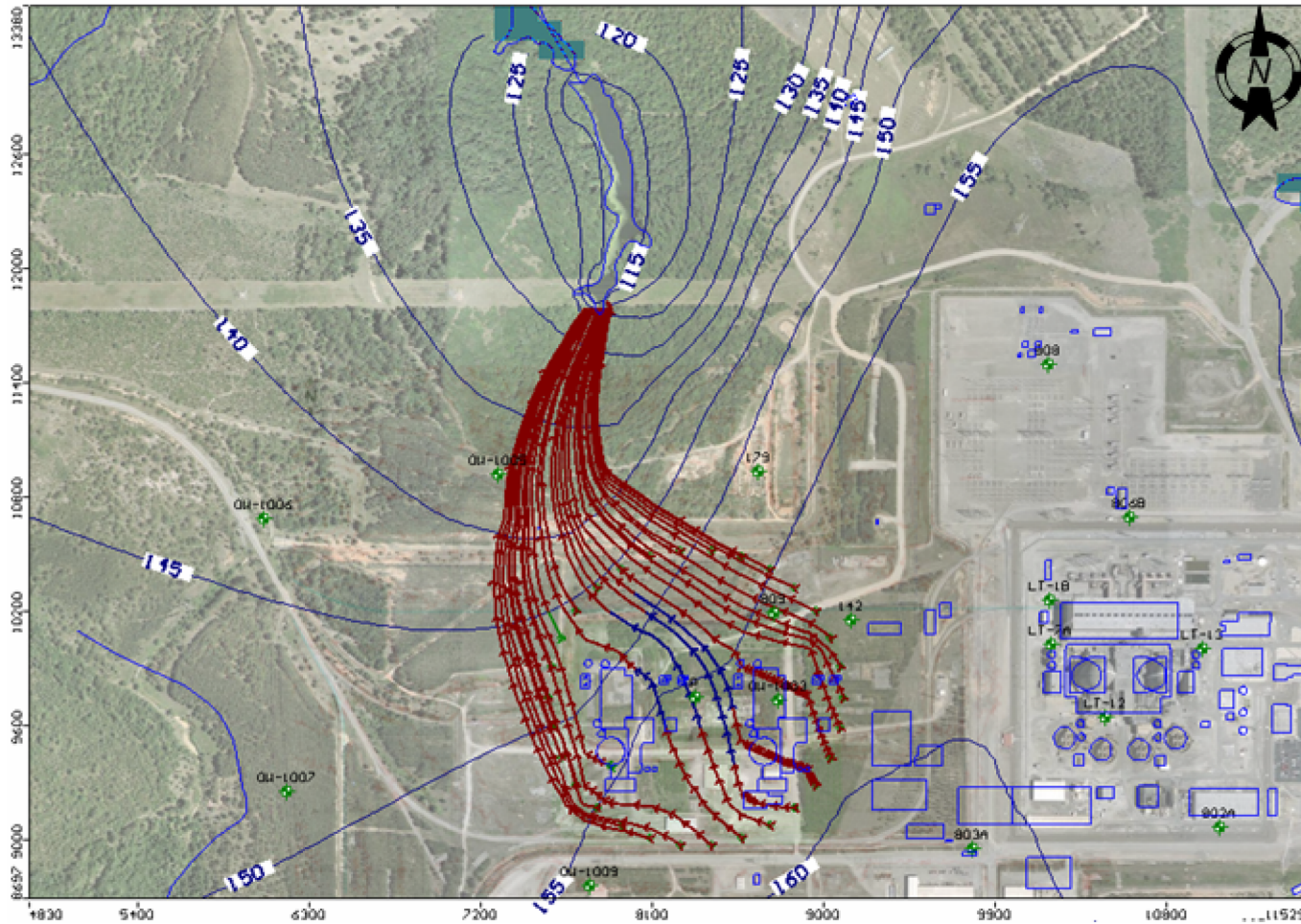Open Item 2.4-2 in SSAR 2.4-12

The applicant should provide an improved and complete description of the current and future local hydrological conditions, including alternate conceptual models, to demonstrate that the design bases related to groundwater-induced loadings on subsurface portions of safety-related SSCs would not be exceeded. Alternatively, the applicant can provide design parameters for buoyancy evaluation of the plant structures.

Open Item 2.4-3 in SSAR 2.4-13

The NRC staff found the applicant's analysis in the SSAR to be incomplete; because it did not include consideration for the inevitable change in hydrology, and, hence, the potential changes in flow direction within the Water Table aquifer for some release locations within the protected area (PA). The applicant's analysis provided no assurance that an adequate number of combinations of release locations and feasible pathways had been considered.

Open Item 2.4-4 in SSAR 2.4-13

The NRC staff's review of the release location, migration, attenuation, and dilution of the radioactive liquid effluent inventory was incomplete because, as stated in Open Item 2.4-3, the applicant has not considered a sufficient number of alternate conceptual models to identify potential release points and pathways. Therefore, the applicant needs to specify the nearest point along each potential pathway that may be accessible to the public.

ATTACHMENT 2

NRC's Comments on the Groundwater Model

Date: 21 February 2008

To: Hosung Ahn

From: Charley Kincaid

Subject: Review of South Nuclear Company's submitted groundwater modeling of the
             Water Table aquifer at the VEGP Site

The following general notes and comments summarize our initial review of the modeling
achieved by SNC.

General Notes and Comments:

1. Vogtle Groundwater Model installed and reviewed – per files received January 23 via e-mail
   message from Chris Cook, NRC. The files consist of three simulation "cases" that are
   described in Section 2.4.12 of the application plus two simulation variants that are not
   included in the document. All simulation cases have both "Existing" and "Future" variants.
   Thus, a total of 10 simulations were received. The cases that are not discussed in Section
   2.4.12 were viewed, but not assessed in detail because of the fact that they are not described
   in the document. Files were generated by Visual Modflow version 4.2.0.151 (Schlumberger
   Water Services, Waterloo, Ontario, Canada) and were reviewed with version 4.2.0.153.

2. All simulation variants consist of a single layer model with non-uniform lateral grid spacing
   and a deformed grid in the vertical direction. However, based on the shapefile "map" data set
   for the top of Blue Bluff Marl (included in some simulation model files, such as Case 1
   Existing) and the model data set representing the bottom of the Water Table aquifer, it would
   appear that the model does not duplicate in good fidelity the primary structural feature of the
   VEGP Site. It is also noted that an expected subcrop of Blue Bluff Marl above the water table
   to the north of VEGP Units 1 & 2 is not included in the model configuration.

3. The cases presented would appear to be a sequence of successively improved calibrations
   or attempts to do so. These might be described as a sequence of calibrations leading to a
   preferred model. However, the goal is to consider plausible alternative conceptual models
   that are fundamentally different than a preferred model in some way, but nonetheless
   consistent with the available data and observations of system behavior. The fundamental
   differences that should be considered are those that might influence our judgment regarding
   the safety of the proposed facility, (e.g., that might result in faster transport or transport in a
   different direction).

4. The saturated conductivity zonation represented in the cases involves a single-layer aquifer
   and ranges from (1) two zones (i.e., high conductivity for engineered backfill, and lower
   conductivity elsewhere, to (2) three zones (the third zone being one added in the immediate

vicinity of Mallard Pond), and to (3) a single layer model with the "everywhere else" zone divided in two along a line through the center of the VEGP site from southwest to northeast. Note, the latter case was provided in the files, but not used in the Rev. 3 of the application. The "Future" cases modify the hydraulic conductivity distribution by adding zones for high conductivity backfill material under reactors for Units 3 & 4. The sequence appears to be an attempt to successively improve the calibration and is not a suite of plausible alternative conceptual models.

5. The saturated conductivity assigned to the engineered fill could be conceptualized in at least two ways; (1) assigned saturated conductivity as measured and previously presented in the ER and SSAR because measurement scale and model scale (e.g., cell size) are similar, and (2) scaled saturated hydraulic conductivity to represent the hypothetical scale-up value proposed by SNC.

6. The infiltration zonation is represented in the cases with a single value for the entire model except at the post-construction areas of Units 1 & 2 (including the cooling towers), which are assigned zero recharge. However, no changes to the recharge zonation are implemented in the proposed construction zones of Units 3 & 4. Thus, the "Future" cases fail to examine alteration of recharge after construction of Units 3 & 4. One simulation variant does assign zero infiltration to areas where Units 3 & 4 reactors and cooling towers are proposed to be located, but this simulation variant is not included in the Rev 3 application. None of the alternatives examine the potential for gravel covered and essentially vegetation-free regions having substantially higher recharge than that associated with pre-construction conditions.

7. There is no discussion of model calibration in the Section 2.4.12 of the application, and there is a systematic error in each of the calibrations. All comparisons between field observations and modeled values of groundwater levels on the Mallard Pond drainage side of the model show a higher modeled value than observed value. All comparisons of groundwater levels between field observations and modeled values on the Telfair drainage side of the model show a lower modeled value than observed value. One may assess this calibration error as being conservative with respect to the direction of flow (i.e., if flow goes toward Mallard Pond even when the simulation head results in the direction of Mallard Pond are higher that the observed value, then a more correct calibration would undoubtedly flow in that direction). However, there are two problems with such an assessment. First, this systematic error in calibration is an indication that the model is not correctly conceptualized. Second, the systematic error in calibration means that the hydraulic gradient produced in the simulation is too low, and, hence, any travel time calculations will be substantially incorrect and non-conservative (i.e., the simulated travel time will be longer than the observed gradient would imply.)

8. In the three cases presented, there are model cells that go dry during the simulation. Similarly, there are cells that indicate flooding in some areas. The dry cells are in the vicinity of the expected Blue Bluff Marl outcrop above the water table, which is north of VEGP Units 1 & 2; however, the dry cells do not appear to represent the entire outcrop. Cells that go dry and those that flood are indications that the conceptual model being simulated needs to be reworked. Note also that the appearance of "dry" and "wet" cells within the simulation may indicate a misrepresentation of the structure.

9. The flow balance of the three cases is quite varied, and does not appear to represent solutions to well posed models. For example, the flow balance of Case 1 Future exhibits a

70% discrepancy, that of Case 2 Future exhibits a less than 1% discrepancy, and that of Case 3 Future exhibits a 75% discrepancy. Such information would argue against these cases being plausible alternative conceptual models of the Water Table aquifer. Note that incremental changes in starting and boundary conditions result in widely varied flow balance results, indicating an incorrect model configuration.

10. There are questions about the boundary conditions used to represent the streams in the model. The unnamed stream and Daniels Branch may be appropriately modeled by a "drain" boundary condition provided the applicant has data or knowledge of its ephemeral character. However, two other streams on the Telfair watershed side of the model appear to be spring fed (see the 1971 Water Table aquifer contour map in the FSAR of Units 1 & 2). These streams may be better represented by "river" boundary conditions, especially during a "March" or spring period of the year. Also, the location and topography of the upper reaches of the unnamed stream that feeds the Daniels Branch could be greatly improved based on local topography and would likely lead to improved calibration to nearby field observations.

11. What we want to see in plausible alternative conceptual models are models that do not violate the data, and, given the simplifications they embody, models that are arguably conservative representations of the site. The three cases presented are not plausible alternative conceptual models of the site.

12. Regarding the movement of tracer particles from the proposed VEGP Units 3 & 4, it is important to examine origins anywhere within the entire power block area. Examining start locations only in the immediate vicinity of proposed Units 3 & 4 is not adequate. The ESP assumes reactor facility locations anywhere within the power block area.

13. The model is insufficiently documented (e.g., basis for boundary conditions, calibration, sensitivity, etc.). For example, assuming calibration has been done and needs to be documented, then the accepted procedures followed to complete the calibration need to be described. This would include some quantitative assessment of the results of the calibration.

14. The model fails to consider the transient nature of the system. There needs to be a valid technical rationale for accepting a steady state model. No rationale was provided for using a steady-state model, nor for the selection of March 2006 as the appropriate observed hydraulic heads to use for calibration. Does the seasonal variation in recharge drive this aquifer system to be dynamic or transient over an annual cycle?