Exhibit 5

(Phase Three Hydrogeological Report)

Subsurface Wastewater Disposal Feasibility Study

Whitten West Site
Wolfeboro, New Hampshire

Phase 3 Hydrogeologic Report

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SUBSURFACE WASTEWATER DISPOSAL FEASIBILITY STUDY

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SECTION 1

EXECUTIVE SUMMARY

SECTION 1

EXECUTIVE SUMMARY

Over the past 8 months, Wright-Pierce has conducted various hydrogeological investigations to assess the feasibility for subsurface disposal of treated wastewater effluent at multiple sites in the Town of Wolfeboro, New Hampshire. This work was part of an effort to determine the most appropriate location for the disposal of treated effluent from the existing wastewater treatment facility (WWTF) for the Town. The Town is under an Administrative Order (A.O.) from the NH Department of Environmental Services (NHDES) to implement a new solution to handle the effluent flow. This work is intended to support a Groundwater Discharge Permit (GDP) application to be submitted concurrently with this report.

Wright-Pierce began its initial hydrogeologic investigations in March 2006. A total of six sites were initially identified throughout the Town and evaluated on a preliminary basis. The most promising sites included: the Caswell; Hershey; and two Whitten sites (Whitten East and West). Following the preliminary evaluation phase, the Whitten West site proved to be the most promising site for in-ground wastewater disposal and is the focus of this report.

For this study, Wright-Pierce has completed test pits, test drilling, water level measurements, infiltrometer, pumping, percolation and slug tests, wick and load cell construction along with long-term loading tests. A preliminary numerical groundwater model was performed during the winter of 2006/2007 along with a final numerical groundwater model.

The Whitten West site is a remote hilly forested upland area located in northwest Wolfeboro. A glaciofluvial landform known as a kame delta occupies a large portion of the site. The kame delta was formed by glacial melt water flowing from a down wasting ice mass and depositing well sorted sand and gravels between the ice mass and the till bedrock upland. Where the melt water velocity decreased, fine grained sediments were deposited within the landform resulting in a variable distribution of sediment grain sizes. A large portion of the kame delta's sand and gravel sediments are unsaturated to depths of 80 feet or greater. This tremendous thickness of unsaturated well sorted sediments is highly unusual for New Hampshire and is ideal for in-ground wastewater disposal. The landform is oriented in an east to west direction parallel to Nineteenmile Brook and the brook abuts the southern toe of the kame delta. Till contacts the kame delta to the north and exposed bedrock outcrops are located to the southeast. Wetlands exist in southern portion and toe of the kame delta.

Wright-Pierce has identified on the Whitten West site approximately 10 acres of area suitable for siting of a land based disposal system. A long-term load test lasting 10 days was performed on a 10' x 10' test cell basin and a three day loading test was performed on a test/production wick 55 feet in depth. The cell loading test was performed at a loading rate of 25 gpm (578 inches per day) and the wick loading test was conducted at a loading rate of 28 gpm. Water levels were monitored in 11 monitoring wells, one wick and two staff gages.

The USEPA recommends the design infiltration rate for a rapid infiltration basin (RIB) should be no more than 10% to 15% of the rate observed during the successful load cell test. The load cell test indicates a vertical infiltration rate of 13.6 inches per day (the maximum rate that NHDES approved for North Conway's rapid infiltration system) or higher is feasible. The results of the load cell test would support a maximum design infiltration rate as high as 58 to 87 inches per day

Using the results of the field investigations, Wright-Pierce along with Watershed Hydrogeologic Inc. developed and calibrated a three dimensional single layer numerical unsaturated groundwater flow model using MODFLOW and the preprocessor program Groundwater Vistas. The steep bedrock surface underlying the kame delta and significant topographic elevation differences along with unsaturated sand deposits presented modeling challenges. The final model was calibrated to the observed ambient and 10 day load test groundwater levels. The numerical model simulated sustained flow rates of 600,000, 800,000 and 1,000,000 gallons per day (gpd) to a hypothetical 62,000 square foot in area simulating a RIB disposal system located on top of the kame delta. The models were run for periods long enough to reach steady state conditions. For the purposes of wastewater facilities planning, the numerical model sustained flow rates are considered equivalent to the annual average flow rate. The actual discharge flows to the facility will vary and could be higher or lower than the modeled flow on any given day, week, or month.

The model predicted an annual average discharge rate of 600,000 gpd will not result in excessive mounding beneath the site. Also, "outbreaks" will not occur at non-existing groundwater discharge areas at this discharge rate. Approximately 50% of the effluent discharge to the site would flow vertically upward through silt, clay, sand and organic deposits underlying the wetlands discharging to the surface within the wetlands. The wetlands along with Nineteenmile Brook serve as existing natural groundwater discharge areas for any precipitation that falls upon or infiltrates laterally through the site. Significant additional treatment should occur as the groundwater travels downward and horizontally through the sand sediments and vertically upward through silt, clay, sand and carbon rich organics that underlie the wetlands and Nineteenmile Brook. The proposed discharge will not increase ponding and flooding of the wetlands due to existing drainage channels and site slopes.

Minimum travel times for effluent impacted groundwater prior to reaching discharge areas (i.e. wetlands and surface waters) are estimated to be one to three months. Ambient groundwater flows on-site are relatively small compared to the proposed discharge flow, and will therefore not provide significant levels of dilution within the site boundaries.

No private or public water supplies exist within 0.5 miles of the site.

Wright-Pierce and Watershed Hydrogeologic conclude that an annual average treated effluent discharge of 600,000 gpd on the Whitten West site is feasible.

SECTION 2

INTRODUCTION

SECTION 2

INTRODUCTION

The Town of Wolfeboro, New Hampshire wastewater treatment plant is a 0.6 mgd design flow secondary treatment plant which currently uses spray irrigation fields and snow making facilities to dispose of the treated effluent. A large effluent storage pond provides storage capacity for periods of time when the spray irrigation and snow making systems cannot be operated. The spray fields were originally permitted for 2-inches. per week of wastewater application and have been in operation for over 30 years for wastewater flows that are less than the plant capacity (approximately 0.3 mgd). The existing effluent storage and disposal facilities lack sufficient capacity to accommodate current and projected future flows, which has led to discharge permit violations. The Town is now under Administrative Order (A.O.) from the NH Department of Environmental Services (NHDES) to address discharge permit non-compliance. In response to the A.O., the Town developed a Wastewater Treatment and Disposal Management Plan dated December 30, 2005 and updated January 24, 2006. Based on the Town's proposed Management Plan, future effluent disposal may consist of one or more of the following elements; 1) continued and/or expanded use of spray fields, 2) use of ice crystallization (E-Snow) and 3) rapid infiltration (RI) or effluent reuse. Wright-Pierce conducted a comprehensive hydrogeological investigation to identify potential sites for RI of Wolfeboro's wastewater effluent.

2.1 In-ground Effluent Disposal Concept

The conceptual plan for the in-ground disposal is through a series of rapid infiltration basins at the Whitten West site. However, the landowner did not initially approve of the rapid infiltration basin concept due to aesthetic concerns therefore the study also included a preliminary evaluation of the potential for Wicks. Utilizing RIB's, highly treated effluent would flow vertically and horizontally as groundwater through a significant thickness of unsaturated soils located on the kame delta then into the saturated finer grained soils located along the base of the kame delta and discharge vertically upward within the wetlands, natural existing seasonal discharge areas for precipitation infiltrating the kame delta and as a base flow to Nineteenmile Brook.

Several key factors to determine whether the preliminary design concept can be successfully implemented on the Whitten West site relative to hydrogeological and environmental standpoints are:

1. Whether or not sufficient soils exist capable of receiving the proposed design hydraulic loading rate for the project;

2. Whether or not the proposed design hydraulic loading rate will cause breakouts to occur along the toe of the kame delta;

3. Whether or not the unsaturated thickness of the soils will accommodate the predicted mound beneath the Rapid Infiltration Basins;

4. Whether or not a hydraulic connection exists between the kame delta geologic deposits and the geologic deposits that underlie the existing wetlands and Nineteenmile Brook; and

5. Whether or not the prevailing water quality standards can be met at the compliance boundaries.

Wright-Pierce conducted the hydrogeological investigation and design activities to be consistent with the approved work plan; NHDES Regulation Env-Ws 1500, Groundwater Discharge Permit Regulations and the USEPA's Process Design Manual for Land Treatment of Municipal Wastewater (1984).

2.2 Previous Work

In December 2005, Wright-Pierce identified six potential sites for RI of Wolfeboro's wastewater. Site WOLF-1 was identified as having the highest potential and within close proximity to the existing infrastructure for in-ground disposal of the Town's present and future treated wastewater effluent. The field inspection identified three distinct areas (Area 1, Area 2 and Area 3) within WOLF-1 that appeared to have very permeable soils with an appropriate depth to groundwater. These parcels of land are remotely located and within the Nineteenmile Brook Drainage Area in the Central-Northwestern part of Wolfeboro.

A Phase 1 investigation of the three sites was initially evaluated through the interpretation of historical color and black/white orthophotography, soil, surficial and hydrogeologic mapping and a windshield survey followed by a more detailed evaluation resulting in the ranking of two sites having the best potential to serve as in-ground subsurface disposal location. Following the installation of test borings and monitoring wells along with land size and availability considerations, the most promising two sites were identified as Wolf 1 Area 1 (Whitten West site - Wolf 1A) and Wolf 1 Area 2 (Whitten East site - Wolf 1B).

Numerous meetings were held between Wright-Pierce, Town of Wolfeboro officials and NHDES regarding the use of the Whitten West site for in-ground effluent disposal. A field site inspection was conducted with NHDES and a work plan submitted to NHDES for comments and insight. NHDES comments were incorporated into the document.

2.3 Phase 1 Results

Wright-Pierce conducted a Phase 1 feasibility investigation beginning March, 2006 consisting of test pitting, test boring and monitoring well installation. The memorandum concluded Wolf 1 Areas 1 and 3 (Whitten West and East) were the most favorable locations and the results warranted further investigation. Test pits and test boring data indicated the site contained soils having suitable permeability characteristics however, the greatest depth of unsaturated soils existed on the kame delta portion of the Whitten West site. Based upon test borings, unsaturated soils along the top of the kame delta ranged from 54 to 63 feet in depth (December 12, 2006). Areas are likely to exist on the Whitten West site where the unsaturated soils are as a much as 80 feet in depth. Well sorted permeable sand and fine gravels extend between the kame delta and wetlands thereby providing a hydraulic connection for groundwater discharge base flow to Nineteenmile Brook, existing wetlands and natural discharge areas. Results from double ring infiltrometer tests indicated of the two sites evaluated (Whitten East and West) the vertical infiltration rate velocities into the soils for the Whitten West site would not be a limiting factor for the proposed volume of in-ground effluent disposal.

2.3 Phase 2 Results

The goal of Phase 2 was to build on the preliminary work performed during the Phase 1 investigation in order to perform a feasibility analysis of the Whitten West site and to recommend whether to proceed with additional studies. The Phase 2 results indicate that the Wolf 1 Area 1 (Whitten West/Wolf 1A) had the best potential to serve as an in-ground subsurface disposal location.

Wright-Pierce investigations indicated that the bedrock surface beneath the kame delta slopes steeply downward to the south and west toward Nineteenmile Brook. The till-stratified drift contact bordering the kame delta to the north was defined during test pitting activities. Overlying the bedrock and within the kame delta are well sorted permeable sand and gravel sediments. The sand and gravel sediments comprise the majority of the unsaturated portion of the kame delta. Poorly sorted sand sediments extend from the toe of the kame delta to Nineteenmile Brook. Field measured double ring infiltration velocity rates indicated that, even when using a conservative 2% to 4% (of field observed rates) design factor as recommended by the USEPA, the on-site sand and gravel deposits would allow design infiltration rates in excess of the maximum NHDES previously approved design rates used previously in New Hampshire. The observed infiltration velocity rates for this site are comparable to those observed for the North Conway Water Precinct's rapid infiltration basin study (Roy F. Weston, 1993). However, although the soils are capable of high vertical infiltration rates other factors such as horizontal hydraulic conductivity of the sediments, topography and potential breakouts may limit the volume of subsurface disposal applied to the area.

A conceptual preliminary groundwater model was developed to: 1) preliminarily evaluate the potential impacts of the wastewater effluent discharge on seasonal high groundwater levels and 2) to preliminarily evaluate the discharge of groundwater originating from the Whitten Site. Watershed Hydrogeologic Inc., Amherst, Massachusetts assisted with and provided technical review of the model. The preliminary modeling results suggested an annual average flow of 600,000 gpd of effluent can be discharged through a land based discharge system on the Whitten West site without breakouts occurring at non-existing groundwater discharge areas.

SECTION 3

STUDY AREA DESCRIPTION

SECTION 3

STUDY AREA DESCRIPTION

3.1 Site Description

Wright-Pierce has completed an initial evaluation of two areas (Whitten west and east) located on the Wolf 1 site (see Figure 1-Appendix A) in the Town of Wolfeboro, New Hampshire for the purpose of in-ground treated wastewater rapid infiltration disposal. The Whitten east property is on Tax Map/Lot 96-14 and the Whitten west property is on Tax Map/Lot 96-13.

An examination of historical aerial photography, USGS topographic and soils maps and field reconnaissance identified a glacio-fluvial geomorphologic landform known as a kame delta. This landform is bisected by the Wolfeboro/Tuftonboro town line in the towns of Wolfeboro and Tuftonboro and forms the head of a much larger stratified deposit. The kame delta is essentially unsaturated on a large portion of the Whitten west site. The site is very remote, wooded and is over 1100 feet north of Nineteenmile Brook. The Whitten east site is located as close as 300 feet from Nineteen mile Brook and its proximity to the brook was an important factor in the final site selection. The closest residential well is located over 3000 feet to the west in the Town of Tuftonboro. This residential well is regionally located hydraulically upgradient of the groundwater discharge zone. Public water supplies are not located within one mile of the site. The Whitten West site is the preferred site due to its distance from Nineteenmile Brook, significant unsaturated thickness, permeable sand, remoteness and sufficient acreage. Figure 2-Appendix A shows the modeled rapid infiltration area on the Whitten West property. The area of well sorted unsaturated sand and gravel on the Whitten west site is approximately 10 acres as shown.

The Whitten West site is located approximately 3.5 miles north of downtown Wolfeboro and the existing Wastewater Treatment Facility (Figure 1-Appendix A). The site proposed for acquisition by the Town of Wolfeboro is comprised of approximately 35 acres of recently logged wooded uplands and is bound to the south by Nineteenmile Brook and to the east by a small unnamed intermittent stream. Land uses for the site consist of timber logging. Only the northern and western portions of the property have suitable soils for subsurface disposal (Figure 2-Appendix A). Wetlands delineated by NHSC, Inc., Newmarket, New Hampshire and existing groundwater discharge areas (springs) identified in the field are shown on Figure 2-Appendix A. Nineteenmile Brook is not classified as a protected surface water. Commercial landowners abut to the west and east. To the north remote undeveloped residential land exists. A power line and associated easement is owned by the Town of Wolfeboro and bisects the site.

3.2 Topography and Surface Water Features

The Whitten West site has hilly south to west sloping topography and till uplands located north of the site slope steeply to south. Wetland areas are located to the south/southwest and along the toe of the kame delta. The ground elevation at the Whitten West site ranges from 688 feet above mean sea level (msl) on the top of the kame delta to 540 feet at the western portion of Nineteenmile Brook. Nineteenmile Brook flows from east to west and is located south of and at the toe of the kame delta. An intermittent stream bounds the site property along the eastern border and in many locations flows directly upon exposed bedrock to Nineteenmile brook (Figure 2-Appendix A). A kame delta forms a flat top ridge beginning immediately west of the intermittent stream and trends from east to west axis coalescing into a large outwash delta deposit to the west in the Town of Tuftonboro. Numerous existing groundwater discharge areas (springs) and wetlands are located along the toe of the kame delta in the southern portion of the project site. Nineteenmile Brook is located south of the wetlands and discharge areas.

The proposed location of the wastewater discharge is not within the Federal Emergency Management Act (FEMA) 100 year flood (Appendix B). The kame delta elevations range from 570 to 688 feet above msl (see Figure 2-Appendix A) and a large portion of the soils within the site is unsaturated. The elevation of wetlands and springs at the base of the esker is approximately 560 feet above msl. Nineteenmile Brook lies at elevations ranging from 505 feet to the west in Tuftonboro to 590 feet in the east in the Town of Wolfeboro. The site is located in the Tuftonboro U.S.G.S. 7.5 minute, 1:24,000 scale quadrangle. Wright-Pierce created a digital base map for the project using public domain GIS (NH GRANIT) files. Detailed topographic survey, test boring/monitoring well locations and monitoring well casing elevations were obtained by East Coast Mapping, Inc. of Bedford, New Hampshire and Lindon Design Associates of Alton, New Hampshire.

Wright-Pierce installed two staff gages (SG-1 and SG-2) along Nineteenmile Brook. The staff gages were equipped with dedicated In-Situ Mini-Troll pressure transducers, for the purpose of monitoring surface water stage variations along Nineteenmile Brook. The majority of the treated effluent discharged into the kame delta would flow vertically downward then horizontal through the unsaturated sand deposits that underlie the proposed location of the RIB's. The horizontal groundwater flow would essentially flow along the south and southwest sloping bedrock topography. A small portion of the discharge would flow to the east and discharge as base flow to the intermittent stream.

3.3 Soils and Surficial Geology

The upland area north of the kame delta is comprised of glacial till. The intermittent stream located to the east of the site flows on top of bedrock and bedrock outcrops were observed on the southeastern portion of the project site forming a defined stratified drift/bedrock contact. Below the 560 foot elevation, the stratified drift deposits become poorly sorted and finer in grain size. The surficial geology for the Tuftonboro 7.5 minute quadrangle was mapped at a small scale by Richard P. Goldthwait (1968) therefore the morphology and delineation should be considered mapped on a macro scale. The sand and gravel deposits where mapped in 1968 by Goldthwait as kame deposits and would normally denote coarse grained somewhat poorly sorted sand and gravel, however, upon closer inspection of the glacial geomorphology, test pits and borings, the deposits where formed in a high energy depositional environment (in Wolfeboro) and pro-grade into a coalescing delta fan (in Tuftonboro) formed in a lower energy depositional environment. The valley width (narrow in Wolfeboro and wider in Tuftonboro) and bedrock slope controlled the depositional environment. The high energy depositional environment resulted in well sorted sand comprising the majority of the proposed RIB site on the Whitten West site. Further to the south and west of the proposed RIB site, the depositional energy environment decreased resulting in the deposition of finer grained and less sorted stratified drift sediments. The glacial till/stratified drift contact for the site is shown on Figure 2 - Appendix A. The surficial geology for the area was mapped by Goldthwait (1968) and is shown on Figure 3 - Appendix A.

The most recent published soils mapping (USDA, 2006) indicates the majority of the site is underlain by Hinckley gravelly loamy sand with a portion of the southern area of the site underlain by Windsor loamy sand and are described as excessively drained, permeable soils formed on terraces or plains of glacial outwash.. Deerfield loamy fine sand, a moderately drained soil, bounds the area to the north. The recently updated soil survey (USDA, 2006) was obtained from public domain GIS (NH GRANIT) files and is shown on Figure 4 - Appendix A. The upland area located to the north of the kame delta is comprised of the Chatfield-Hollis-Canton complex and the parent material is glacial till. The wetlands are mapped as being underlain by Greenwood mucky peat formed in decomposed organic material and are typically very poorly drained.

Underlying the sand deposits is a thin discontinuous veneer of glacial till, a dense heterogeneous deposit that can include clay, silt, sand, gravel, cobbles and boulders. Till was deposited directly by the glacial ice mass and can be further defined as lodgement, ablation or flow till. Because of till's heterogeneity, density and the presence of fine-grained soils, till usually is much less permeable than sand and gravel deposits. The bedrock is overlain by discontinuous areas of glacial till, with the greatest thickness occurring on the upland hilly portion to the north of the site. Geologic cross-sections A-A', B-B' and C-C' depicting the site's subsurface materials are depicted in Figures 5, 6, and 7, respectively (Appendix A).

3.4 Bedrock Geology

Wright-Pierce's bedrock investigation has included a review of published maps and literature; fracture-trace analysis on two sets of aerial photography; the NHDES well database, and boring results from the Phase 1 and 2 Field Activities Summary memorandum, dated May 30, 2006.

According to published mapping (Bedrock Geologic Map of New Hampshire, 1997), the northern portion of the site is underlain by schist and gneiss of the Littleton Formation and Winnipesaukee quartz diorite underlies the southern portion of the site. The bedrock geology in the area of the Whitten site is shown on Figure 8 - Appendix A and derived from public domain GIS (NH GRANIT) files.

Wright-Pierce contoured the surface of the bedrock (Figure 9 - Appendix A) using information obtained from the test borings and field inspection. Although there are some areas with ample data and other areas with no depth to bedrock information, the contour map shows a general south to southwest dipping trend. Bedrock outcrops occur on a small hill located east/southeast of the site. The south to southwest dipping bedrock surface will serve as a controlling groundwater flow path for effluent discharge to Nineteenmile Brook and the wetlands.

3.5 Private and Public Water Supplies

Private and public water supplies within 1.0 mile of the Whitten west site were obtained from the New Hampshire Department of Environmental Services Geologic Survey. No public water supplies exist within 1.0 miles of the Whitten site. The nearest private well is located 3,000 ft to the west of the proposed discharge area (Figure 10 - Appendix A). The high level of effluent treatment afforded by the existing wastewater treatment facility along with additional treatment within the large thickness of unsaturated terrace soils make deleterious impacts to the private wells highly unlikely. Additionally, all discharged water will eventually reach Nineteenmile Brook within a very short distance from the site boundary and leave the site as surface water and not as groundwater.

3.6 Wetlands

Wetland areas that function as natural existing groundwater discharge areas were delineated by a New Hampshire Soil Consultants wetland scientist on the southern portion (discharge zone) of the Whitten West site and along Nineteenmile Brook. The wetlands were located using sub-meter accuracy GPS ground survey techniques and are shown on Figure 2 - Appendix A. A portion of the discharged effluent will discharge vertically upward to the wetlands through the underlying soils. It is likely the silt, fine sand and organic material found in the vicinity of and beneath the wetlands will provide additional treatment. The proposed discharge will increase the water flow through existing wetlands and likely increase the period of time when the wetlands are wet. However, due to the site topography and existing drainageways between the wetlands and Nineteenmile Brook, the proposed discharge will not significantly increase the size of the wetlands areas, nor will it measurably increase ponding or flooding conditions within the wetlands.

3.7 Soil and Groundwater Laboratory Results

Grab soil samples were collected from the bottom of the load cell basin and treated effluent samples were collected from the Wolfeboro wastewater treatment plant located at Filter Bed Rd and submitted for analysis of arsenic leaching potential. The analysis was performed in the laboratory by Kevin H. Gardner, Ph.D., P.E. Department of Civil Engineering, University of New Hampshire.

The arsenic leaching tests results indicate that there are measurable ubiquitous concentrations of arsenic in site soils; however, no measurable arsenic was released in any of the leaching tests. Moreover, the arsenic in these site soils is not environmentally available, even under the reducing conditions experienced in the presence of the WWTP effluent. Introduction of the WWTP effluent is not anticipated to release measurable amounts of arsenic into ground waters or surface waters. The results can be found in Appendix D.

A groundwater sample was collected from the six-inch diameter pumping well located adjacent to MW-14 and submitted to Eastern Analytical, Manchester NH, for analysis of arsenic, copper, sodium, chloride, nitrite, nitrate total phosphorous and biological oxygen demand. Results of the analysis indicate chloride was detected at a very low concentration of 2 mg/L. No other parameters were above reporting detection limits. The analytical water quality results can be found in Appendix D.

SECTION 4

TEST BORINGS AND MONITORING WELLS

SECTION 4

TEST BORINGS AND MONITORING WELLS

4.1 Test Borings

Wright-Pierce subcontractor, New Hampshire Boring of Londonderry NH, drilled a total of eighteen test borings and constructed sixteen monitoring wells on the Whitten East and West properties between July and December 2006. Boring and monitoring well locations are shown on Figure 2 - Appendix A.

The initial site evaluation consisted of the installation of thirteen test borings and the completion of 10 monitoring wells (see Figure 2 - Appendix A). On the Whitten west site, test borings B-6, 8, 9, 10 and 11 encountered excellent (permeable) soil materials to warrant the completion of the borings as monitoring wells MW 5, 6, 7, 8 and 9 respectively. Test borings B-7 and B-9A did not encounter favorable soil materials due to the presence of silty fine sand and shallow depth to bedrock and were not completed as monitoring wells.

On the Whitten west and east sites, monitoring wells MW 1, 2, 3, 4, 5, 6, 7, 8, 9, 10 and 11 were installed within a segmented coalescing glacio-fluvial landform that extends from east to west through the Whitten East and West properties. These test borings indicate the most favorable soil type, depth to bedrock and unsaturated thickness of soil material exist on the Whitten West property. The Whitten West site has the largest land area available, well sorted uniform permeable soil material as well as the greatest thickness of unsaturated soils of the two Whitten sites. On the Whitten West site, unsaturated soils are as great as 80 feet and average around 50 feet. The unsaturated soil thickness on the Whitten East site averages approximately 21 feet and is also favorable for wastewater disposal. The soil materials encountered on the Whitten East site are generally classified as a fine to medium sand with silt lenses and on the Whitten West site fine to coarse sand comprises the proposed in-ground effluent area. The locations of boring and monitoring wells on the Whitten east and west site are shown on Figure 2 - Appendix A, and the soil boring logs can be found in Appendix E and test pit logs in Appendix F.

The monitoring well construction data, depth to bedrock and observed groundwater level are presented in Table 1 (Appendix C).

Test borings were advanced using the drive and wash method with 4-inch diameter flush joint steel casing. Roller cone drilling was used to confirm suspected bedrock, drill through boulders, and to drill through resistant formation where present. During drive and wash drilling, 2-inch diameter split spoon samples and "wash" samples were collected of the formation. At most locations, drilling proceeded until "refusal" was encountered. The boring and monitoring well construction logs can be found in Appendix E. A monitoring well was constructed in each borehole, inside the 4-inch diameter steel drive pipe. Each monitoring well consists of flush joint schedule 40 two-inch diameter PVC screens and riser pipe, installed within the drive pipe. The drive pipe was then withdrawn, with the annular space filled with No. 1 sand pack, bentonite and a protective pipe. Each well screen had 20 slot (0.020") openings and varied in length depending on saturated thickness and overall depth of the geologic formation. A summary of the monitoring well construction can be found in Table 1 (Appendix C).

Drilling and test pitting results clearly delineated the stratified drift-till contact between the kame delta and till comprising the upland area to the north (Figure 2 - Appendix A). Stratified sand and gravel predominates the study area with the thickest well sorted uniform material found within the kame delta landform located on Whitten West. Confining or semi-confining deposits were not encountered during the test borings. Relatively small proportions of silt and clay can significantly inhibit infiltration capacity and prove problematic for subsurface disposal systems (USEPA, 1984). A very thin veneer of glacial till was sometimes encountered above bedrock.

4.2 Well Development

Following drilling and well construction, the monitoring wells were developed (where saturated) by hand using a submersible pump, surging and Teflon tubing. All of the monitoring wells were developed until free of fine sands and silt. Only MW-7 became dry during development.

4.3 Water Level Measurements

Antecedent water levels were measured both manually and with an electronic data logger starting on December 4, 2006 in MW 1, MW-5 through 9 and MW-11 through 16 and two staff gauges SG-1 and SG-2. Prior to start of the load cell test on December 12, 2006, three sets of water level measurements were obtained in the designated monitoring wells. A Solinst 101 electronic water level meter was utilized to manually measure water levels and In Situ® Level Troll pressure transducers were utilized to record water levels electronically.

Lindon Design Associates of Alton Bay, New Hampshire surveyed the top of casings and located the monitoring wells with a global positioning system (GPS). The results of the survey, combined with Wright-Pierce's water level measurements, allow the development of a potentiometric (piezometric) water level contour map (Figure 11 - Appendix A). From this map, ambient groundwater flow directions have been interpreted to show a generally southwesterly groundwater flow direction from the base of the kame delta ridge and discharging as base flow to Nineteenmile Brook. The unsaturated thickness within the kame delta measured prior to the start of the load cell test on December 12, 2006, ranged from 40 to 50 feet (Table 1- Appendix C).

SECTION 5

TEST PITS AND INFILTROMETER TESTS

SECTION 5

TEST PITS AND INFILTROMETER TESTING

5.1 Test Pits

A total of eighteen test pits were excavated on the Whitten West site and eleven test pits on the Whitten East site (Figure 2- Appendix A). Test pits excavated on the Whitten West and East sites helped refine the test boring data in greater detail for depths ranging from 0 to 10 feet below ground surface (bgs). In general, test pits 3 thru 7 and 13 thru 16 for the Whitten West site indicate the soils from 3-10 ft bgs along the kame delta ridge are comprised of well sorted fine to coarse sands and occasional pebble to fine-medium gravel. The gravel forms a topset bed and tended to decrease sharply with depth. In general, test pits 19 thru 29 for the Whitten East site indicate the soils from 2-10 ft bgs along the esker are comprised of alternating layers of well sorted fine to medium sands with frequent stringers (layers) of silt and occasional coarse sand. Soils were interpreted to be coarser grained, uniform and well sorted on the Whitten West site then the soils found at the Whitten East site.

5.2 Double Ring Infiltrometer Tests

Double ring infiltrometer tests were performed at two locations on the Whitten West site and at one location on the Whitten East site. Infiltrometer tests IT-1 and 2 were performed on the Whitten West site adjacent to test pits 4 and 5 and IT-3 was performed on Whitten East site adjacent to test pit 20. The double ring infiltrometer tests were performed to estimate vertical infiltration rates. Double ring infiltrometer tests are considered more accurate than percolation tests because percolation tests have a horizontal flow component whereas the double ring infiltrometer tests the vertical flow component only. All tests were performed at depths ranging from of 3.5 to 4 feet below ground surface where rapid infiltration trenches or basins would potentially be located after grading and bed construction.

Testing and data analysis was performed according to double ring infiltrometer standard method ASTM D 3385-03. For all of the double ring infiltrometer tests, Wright-Pierce maintained the constant head conditions called for in this standard method. The double ring infiltrometer is comprised of an inner and outer ring, 12 and 24 inches in diameter, respectively. Potable water served as the water source and was delivered to the site via a 300 gallon tank mounted on an all-terrain vehicle. Water was allowed to flow by gravity from the tank into each of the rings via a 2-inch diameter fire hose attached to two manifolded flow meters with flow control valves. The outer ring flow meter range was 1 - 10 gpm and the inner ring flow meter range was 0.2 - 2 gpm. A constant head was maintained by measuring the water level in each ring and adjusting the flow control valves as necessary to maintain a 1-inch water level within both rings and flow rate (gpm) recorded.

Results indicate a moderate to rapid infiltration rate ranging from 736 to 1814 inches per day (in/day). The Whitten West site had infiltration rates of 1500 and 1814 inches per day (IT-1 and 2 respectively) and Whitten East rate was 736 inches per day (IT-3). Note that although units for infiltration rates are the same as those for hydraulic conductivity, the two parameters are not the same. USEPA guidance (USEPA, 1984) advises that for RI system design, a maximum of 2-4% of the field-measured infiltration rate should be used. Therefore, the maximum recommended design infiltration rates on the Whitten West site should not exceed 30 to 60 inches per day. The results are presented in Table 2. The field infiltrometer data is presented in Appendix G. Due to the natural variability in the horizontal and vertical distribution of soil material existing on the site, the infiltration rates reported herein is only an estimate of the material tested and do not represent the variability of the site.

SECTION 6

AQUIFER CHARACTERIZATION

SECTION 6

AQUIFER CHARACTERIZATION

6.1 Soil Sieve Analyses

Selected soil samples collected during drilling of the test borings and the excavation of the test pits were delivered to Miller Engineering and Testing in Manchester, New Hampshire for sieve analysis. Soil descriptions based on sieve analysis ranged from poorly graded sand to sand with gravel. The soil sieve analysis reports are provided in Appendix H.

Based on the laboratory sieve analysis, hydraulic conductivity of the samples was estimated using the Hazen method (Freeze and Cherry, 1979) (Fetter, 2001) and can be found in Table 3 (Appendix C). The values shown in Table 3 should only be considered a rough approximation of hydraulic conductivity. Hydraulic conductivity is the rate at which water moves through a porous medium under a unit head gradient. The 10 percent finer (d_{10}) grain size, in millimeters, is squared and centimeters per second calculated. Centimeters per second were then converted to feet per day (ft/d). The Hazen equation (Freeze and Cherry, 1979) is $K= 1(d_{10}^2)$ while Fetter (2001), multiplies the 10 percent finer (d_{10}) grain size, in centimeters, by a constant of 80 to obtain centimeters per second. The Hazen derived hydraulic conductivity for the Whitten site ranged between 16 to 519 ft/day.

6.2 Slug Tests

Slug testing of the saturated zone was performed on monitoring wells completed on the Whitten West and East Sites to obtain aquifer hydraulic conductivity values. The tests were conducted on November 14, 2006 by Wright-Pierce. A solid slug was used to perform the slug tests in wells where groundwater was present. An In-Situ Level Troll® pressure transducer with internal data logger was placed within the well to measure and record the change in water level over time.

To perform the solid slug tests a solid 1-inch diameter PVC cylinder six feet in length was instantly lowered into (falling head test) or removed from (rising head test) the water table to displace the water table in the well. The recovery of the water table to the static water level was then recorded.

The data collected during the slug tests was analyzed using the Hvorslev and Bouwer and Rice methods with the aid of Aquifer Test V4.0 slug test software developed by Waterloo Hydrogeologic (2005). Hydraulic conductivity values ranged from 0.5 to 70 feet per day (ft/day). These results fall within the range expected for the soils encountered and are summarized in Table 4 (Appendix C). The Aquifer Test Analysis reports can be found in Appendix I.

SECTION 7

WICK CONSTRUCTION

SECTION 7

WICK CONSTRUCTION

The Whitten West landowner requested the Town of Wolfeboro to construct an effluent disposal wick (wick) and perform a load test as an alternative to conventional rapid infiltration basins. The landowner had aesthetic concerns regarding rapid infiltration basins on the Whitten West site. An effluent disposal wick is a vertical subsurface structure constructed for the purpose of transporting highly treated effluent to a permeable geologic formation. The formation can either be saturated or unsaturated or a combination of both. A wick is essentially a large diameter borehole filled with pea stone or gravel with an inner liner screen, which is subsequently pulled back partially after the stone is placed inside the hole. Alternatively, the wick can be constructed using a stainless steel slotted well screen and foregoing the stone fill within the casing. Highly treated effluent is discharged into the wick allowing the wastewater to flow through the stone or well screen into the surrounding sand and gravel formation. Wicks allow for very efficient transport and dispersal of highly treated effluent both vertically and horizontally into the permeable sand and gravel formation. In general, as with conventional land based wastewater disposal systems, wick applications require certain favorable hydrogeologic conditions. As described herein, the Whitten West site provides such favorable conditions.

Regular maintenance is required and can readily be performed by conventional Vac Trucks, hydrogen peroxide/chlorine disinfection and swabbing with surge blocks. Due to their relatively low costs, additional wicks can be constructed in very close proximity to one another. The wicks take up very little horizontal area and are very useful when land constraints prevent the construction of conventional subsurface leaching chambers or rapid infiltration basins. Wicks can easily be designed to be hidden from view. At the time of this submittal, wicks have been used at two facilities: the Erickson Retirement Community, Hingham, Massachusetts for over 3 years; and West Island municipal wastewater treatment and disposal facility in Fairhaven, Massachusetts for 9 years.

A single small diameter wick was installed (See Appendix J - Photographs) on the top of the kame delta adjacent to monitoring well MW-5 (Figure 2 - Appendix A). The wick was installed using a 60- L cable tool drill rig by the R.E. Chapman Company of Boyleston, Massachusetts. Twelve inch diameter casing was bailed down (to prevent compaction) to a depth of 55 feet below ground surface. Once all the soil was removed from the casing, formation samples were analyzed for grain size analysis. The wick sieve analysis reports are contained in Appendix K. A No. 3 artificial gravel pack was selected along with 40 feet of 8-inch diameter 304 stainless steel pipe size liner screen. The screen had four ten foot sections of variable slot screen openings. The slot openings are the following from below ground surface (ft bgs): 45-55 feet, 0.030 slot, 35 to 45 feet 0.040 slot, 25 to 35 feet 50 slot and from 15 to 25 feet 60 slot. The wick was constructed in unsaturated soils and the wick as-built can be found in Appendix K.

SECTION 8

LOAD CELL TESTING

SECTION 8

LOAD CELL TESTING

Wright-Pierce conducted a separate loading tests on a single 10' x 10' basin and a 12" x 8" wick located on top of the kame delta. The purpose of the tests was to determine the following:

1. To conduct an in-field assessment of the feasibility, at a scale intermediate between double ring infiltrometer tests and a fully functioning rapid infiltration subsurface disposal system;

2. To conduct an in-field evaluation of the potential of the Whitten West site to receive in-ground effluent disposal through a wick; and

3. To document and calibrate a RIB loading scenario that can be used to verify the numerical groundwater model.

The load test on the basin was designed for the site based on USEPA guidance (USEPA, 1984), the North Conway experience (Roy F. Weston, 1993) and a work plan submitted to NHDES. The load test on the wick was designed for the site based on the NHDES work plan, Erickson Retirement Community experience (Earth Tech, 2003) and Wight-Pierce's recently completed Raymond, New Hampshire study. The planned discharge rate for the test was based on three factors. First, NHDES has indicated that the approved infiltration rate for North Conway's rapid infiltration is 13.6 in/day, and to NHDES's knowledge this is one of the highest approved rates in the country. Second, USEPA guidance (USEPA, 1984) indicates that the maximum design rate for rapid infiltration should be 15% or less of the rate used in a successful load cell test. Rounding 13.6 in/day to 14 in/day yields a test rate (100%/15%= 6.67) of about 93 in/day if the maximum rate is to be permitted. Over a 10-foot by 10-foot basin, 93 in/day is equivalent to 4.03 gpm, so this is the minimum rate that Wright-Pierce may use for the load test. Third we wished to use a higher rate in order to create a large enough hydraulic response within the unsaturated sand in order to calibrate the numerical groundwater model.

H & F Construction Inc. of Tuftonboro, NH constructed the load cell basin under the supervision of Wright-Pierce personnel adjacent to B-8/MW-6 and B-14/MW-12 (Figure - 2, Appendix A). The 10' x 10' basin was excavated to a depth of four feet below ground surface. Sheets of plywood were placed on the side slopes of the basin and the bottom lined with 3 inches of 2-inch diameter crushed stone. The load cell basin picture can be found in Appendix J.

A temporary 6-inch diameter gravel developed dewatering well, installed next to MW-14 and equipped with a 4 inch submersible pump, provided the water supply for the loading tests via 1700 feet of 2-inch diameter PVC piping. The piping was fitted with a control valve and connected to a flow meter.

A total of 19 In-Situ Level Troll® pressure transducers were installed in all monitoring wells installed on the Whitten West site, the ambient well MW-1 (Whitten East site) and staff gages SG-1 and SG-2. The staff gages were installed in Nineteenmile Brook to record water levels. Water levels in the monitoring wells were also measured manually with a Solinst water level meter. Precipitation was recorded by an on- site rain gage and a private weather station located in South Wolfeboro, New Hampshire. The weather station data is contained in Appendix L.

8.1 Pre-Load Test

Only a trace amount (0.04") of precipitation was recorded at the Wolfeboro weather station during the antecedent period and did not have an impact on groundwater or surface water levels. During the week prior to the start of the load cell test on December 12, 2006, three sets of field water level measurements were obtained in the designated monitoring wells. Overall water levels in the monitoring wells and the ambient well remained steady or declined slightly during the period leading up to the loading test, with the exception of MW-13 which showed a slight increase (Figure 12 - Appendix A).

8.2 Load Test -Cell Basin

The load test on the basin was started at 11:00 am on December 12, 2006. Discharge water was supplied to the test basin via 1700 feet of 2-inch diameter pvc pipe as discussed in Section 8.0. The load test lasted 10 days ending on December 22, 2006. On December 14 and 16, 2006, approximately 0.2 and 0.08 inches of precipitation, respectively, was recorded in the on-site rain gauge. On December 15 and 22, 2006 approximately 0.12 and 0.08 inches of precipitation, respectively, was recorded at the South Wolfeboro weather station.

At the start of the test, an initial loading rate of 28 gpm was discharged to the test basin and then decreased to 26 gpm on December 13, 2006. On December 14, 2006 the flow rate was decreased to 24 gpm and remained constant until December 19, 2006 when the loading rate increased to 25 gpm and then remained constant until the end of the test. The variation in flow rates is attributed to generator performance. No appreciable ponding was observed in the test basin during the loading period.

EPA guidance recommends that Rapid Infiltration Basin system design loading rates should be limited to 10% to 15% of the field observed loading rates from the successful Load Cell testing results. The observed load cell testing infiltration rate was 578 inches per day. Therefore the maximum design infiltration rate should not exceed 58 to 87 inches per day.

The only discernable effect on water levels associated to the load cell discharge was observed at MW-6, 12 and 16 and is illustrated in Figure 12 - Appendix A. Effects from the discharge at the cell basin were not immediately observed in MW-12, installed next to the cell basin. It was suspected that a false refusal was obtained during the construction of MW-12 and the screen bottom was set some distance above the bedrock/till layer and therefore not responding to the groundwater discharge. To verify this observation B-18/MW-16 was installed on December 13, 2006, approximately 17 ft south of the cell basin. On December 14, 2006, two days into the test, a measurable water level was recorded in MW-12. On December 15, three days into the test water levels began to increase in wells MW-6 and MW-16. The manual water level data is contained in Table 5.

The electronic water level data indicate that:

- The maximum water level change at MW-12 (approximately 2 ft from the edge of the cell basin) was 1.8 ft on December 21, 2006 and then decreased 0.02 ft upon shut down on December 22, 2006. Full recovery occurred on December 25, 2006, three days after shutdown.
- The maximum water level change at MW-16 (approximately 17 ft from the edge of the cell basin) was 5.67 ft on December 21, 2006 and then decreased 1.22 ft upon shut down on December 22, 2006. Please note that the transducer was covered in silt upon removal. (A review of the transducer data at this location shows erratic increasing and decreasing trends. The manual data indicate a maximum rise of 4.19 ft on December 19, 2006). Full recovery occurred on December 29, 2006, six days after shutdown
- The maximum water level change at MW-6 (approximately 75 ft from the edge of the cell basin) was 5.4 ft on December 23 approximately 26 hours after startup of the loading test. Full recovery occurred December 29, 2006, six days after shutdown.
- The water level at ambient well MW-1 increased very slightly (0.02 ft) during the test.

The electronic water level data and a graph of the data collected during the load cell test are contained in Appendix M.

The rate of change for the remaining monitoring wells can be found in Table 5 (Appendix C). By January 8, 2006 all of the monitoring wells had recovered to or were below their pretest levels. Those monitoring wells that had recovered to original pretest water levels were dry at the start of the load test.

Surface water trends recorded at staff gages SG-1 and SG-2 clearly indicate Nineteenmile Brook was influenced by precipitation events during the load cell test. From December 4 to December

12, staff gage SG-1 declined steadily and we conclude from the monitoring well responses the discharge rate applied at the basin did not affect the brook in this area.

8.3 Load Test -Wick

The load test on the wick was started at 12:00 pm on January 2, 2007. Discharge water was supplied to the wick via 1960 feet of 2-inch diameter pvc pipe as discussed in Section 8.0. The wick load test lasted 3 days ending on January 5, 2007. On January 1 and 2, 2007, approximately 0.4 inches and a trace amount of precipitation, respectively, was recorded at the South Wolfeboro weather station. Water levels were monitored electronically in the wick, MW-5, MW-6, MW-7 and MW-8.

During the entire duration of the wick loading test, a constant loading rate of 28 gpm was discharged to the wick. The discharge rate was limited to pump and dewatering well capacity and the not the hydraulic efficiency of the wick. The only discernable effect on water levels from the discharge was observed at the wick and MW-5 and is illustrated in Figure 12-Appendix A.

The electronic water level data indicate that:

- The maximum water level change at the wick was 4.74 ft on January 3, 2006 and then decreased 0.07 ft upon shut down on January 5, 2006. Full recovery occurred on January 5, 2006, four hours after shutdown.
- The maximum water level change at MW-5 (approximately 2 ft from the edge of the wick) was 8.06 ft on January 5, 2006. Full recovery occurred on January 7, 2006, two days after shutdown.
- The greater water level change observed at MW-5 is attributed to the monitoring wells deeper screen setting than the wick screen bottom.

The electronic water level data collected during the wick loading test is contained in Appendix N.

SECTION 9

NUMERICAL MODELING

SECTION 9

NUMERICAL MODELING

9.1 Approach and Goals of Modeling

The overall goal of the numerical groundwater flow modeling is to quantify and predict the physical and chemical impacts of the proposed in-ground effluent discharge by a subsurface disposal system and includes addressing the following aspects of the subsurface system:

- Changes in groundwater directions
- Fate of infiltrated effluent
- Changes in water level including local mounding in the vicinity of the subsurface system
- Potential for "breakout" prior to a receiving surface feature such as the wetlands and Nineteenmile Brook
- Nutrient dilution at receiving surface features such as the wetlands and Nineteenmile Brook
- Optimization of the location, size, and application rate of the disposal facilities

To best achieve these goals Wright-Pierce chose the USGS "Modular Three-Dimensional Finite-Difference Groundwater Flow Model (Harbaugh, 2000), also known as MODFLOW and MODFLOW-SURFACT (MSFT), for several reasons:

- MODFLOW is the most widely used and best documented groundwater flow model in the world
- MODFLOW is flexible when coupled with a graphical processor such as VISUAL MODFLOW or GROUNDWATER VISTAS

- MODFLOW readily simulates key aquifer system characteristics, such as aquifer stratigraphy, partial well penetration, time intervals, groundwater-surface water interactions, etc.
- The standard version of MODFLOW can only simulate saturated groundwater flow and much of the Whitten West site is unsaturated thereby requiring a modified analysis utilizing MODFLOW- SURFACT.

Jesse Schwalbaum, P.G. of ***Watershed Hydrogeologic Inc.***, Amherst, Massachusetts assisted in the development and provided technical review of the preliminary and final numerical groundwater models.

9.2 Preliminary and Final Numerical Groundwater Modeling

A preliminary numerical groundwater model was developed for the site to: 1) preliminarily evaluate the potential impacts of the effluent groundwater discharge within the kame delta upon seasonal high groundwater levels 2) assess the possible ranges in effluent discharge rates in order to determine the economic viability of proceeding with a long-term load test and 3) to compare load rate affects from the basin versus the wick upon the sites hydrogeology. The preliminary model served as the framework for the development of the final model.

9.2.1 Precipitation and Seasonal High Water Table

Precipitation for the region during 2005 and 2006 was well above normal and by December, 2006, the precipitation was 11 inches above average. The USGS maintains a very limited number of groundwater monitoring wells in New Hampshire. The closest well is USGS 434221071051501 NH-OXW 38 located in Ossipee, New Hampshire. Only one data value is available for this well for the month of December, 2006. Since this data is only collected on a monthly basis and the USGS well is sited in a significantly different hydrogeologic (saturated through-valley deposit) setting, applying the Frimpter USGS correction methodology is not applicable to this project. On December 12, 2006, water levels were obtained in all the monitoring wells (with the exception of MW-16 which was not constructed at the time). This time period represents a conservative seasonal high groundwater level condition and the December 12, 2006 water levels were utilized as the starting head elevations in the groundwater model. Of the 40 inches (Moore, 1995) of average annual precipitation for the region, approximately 50% reaches groundwater and the rest is to lost to evapotranspiration and runoff (Harte and Johnson, 1995). This estimate of recharge only applies to precipitation falling on the stratified drift and percolating vertically to the water table.

9.2.2 Model Construction

The groundwater model grid (Figure 13 - Appendix A) extends northward from Nineteenmile Brook in the south, and is bounded by till or bedrock to the north and east. The area of the model is approximately 62 acres (1,500 by 1,800 feet). The model grid consists of 77 rows 92 columns and 1 layer. Grid cell sizes are 20 to 20 feet over the entire model. The proposed subsurface effluent disposal system, as well as the simulated wetlands and river, are all within the upper layer. Nineteenmile Brook serves as specified head boundary on the south. The till and bedrock to the north, and east comprises a no-flow boundary. The source of water to the model area is exclusively through rainfall recharge. The model calibrated best to observed conditions using an annualized average recharge of 22 inches per year and is within published values.

The hydraulic conductivity values were based on data derived from field permeability tests although, to be conservative, values on the lower end of the observed range were used in the model. Model hydraulic conductivity values ranged from 10 ft/day (silty sand) to 100 ft/day (coarse sand) (See Figure 14 - Appendix A). A vertical to horizontal hydraulic conductivity ratio of 0.2 was used throughout the model. Nineteenmile Brook was simulated as a series of river nodes. The gradient of the brook was determined from the topographic survey. The sandy soils of the brook bottom were assigned a vertical hydraulic conductivity of 2 ft/day. The unnamed brook that bounds the site on the east was simulated as a series of drain nodes. The wetlands/seeps were simulated as drain nodes and the hydraulic conductivity values were adjusted slightly from the earlier preliminary model in order to better calibrate to the load test results. The drain nodes were increased in the western and decreased in the eastern wetland areas and the average hydraulic conductivity was 1 feet/day.

The initial steady-state run of the model was compared to observed water levels (December 12, 2006) in six of the on-site observation wells. The observed water levels are assumed in this model to represent seasonal high groundwater table conditions as per the July 2005 NHDES Groundwater Discharge Permitting Guidance Document for Recharging Aquifers with Reclaimed Wastewater. The model-simulated water levels ranged between -1.16 to 2.36 feet from the December 12, 2006 observed water levels (see Table 1 of Watershed Hydrogeologic Inc. February 14, 2007 Memo- Appendix O). This range in values is considered a good match. The model simulated steady state groundwater contours are provided in Figure 15 - Appendix A. Where the model remains unsaturated groundwater contour shown in Figure 15 become irregular in shape and is a function of the unsaturated flow model algorithms.

Discharged effluent wastewater is proposed to be introduced into the kame delta via RI effluent disposal system. The RI effluent disposal system effluent volume is based upon the flow that will not cause significant groundwater breakouts at the toe of the slopes or wetlands and maintaining the effluent mound at least 4 feet below the bottom of the disposal bed. The model was run as a steady state simulation. This means that the simulation is run for as long as it takes to reach equilibrium. The location of the RI effluent disposal system and model grid nodes can be found on Figure 13 - Appendix A. The RI effluent disposal system is located on top of the kame delta and has an assumed area of approximately 62,200 ft^2.

For a detailed discussion on the loading test calibration see Watershed Hydrogeologic Inc. February 14, 2007 Memo Appendix O. The final model calibration process has some inherent limitations because MODFLOW (and virtually all groundwater flow models) is based on a mathematical simulation of saturated groundwater flow. However, much of the flow occurring near the loading tests was occurring under unsaturated conditions. A majority of the sand and gravel in the upper area of the Whitten West site is not saturated with groundwater even under the December 12, 2006 modeled high groundwater conditions. Eight monitoring wells and the wick located on the site were dry prior to the start of the load test on December 12, 2006. The unsaturated soils are attributed to the steep bedrock gradient and the high hydraulic conductivity of the soils comprising the kame delta. This means the groundwater flow will be more rapid and mounding will be less than can be predicted by a groundwater flow model in those areas in which unsaturated flow is occurring. The model predictions are not reliable in those areas where the bedrock gradient is steep. Therefore, Watershed Hydrogeologic focused on calibrating to water levels in areas that were under saturated groundwater flow conditions.

9.3 Sensitivity Analysis

To determine the relative importance of each of the major model parameters a sensitivity analysis was conducted on the model. The model parameters were adjusted in increments up to 50% above and below the calibrated values. Model parameters analyzed were the five hydraulic conductivity zones (Kx1 thru Kx5) and recharge. The sensitivity results can be found in Figure 5 of Watershed Hydrogeologic Inc. February 14, 2007 Memo - Appendix O. The hydraulic conductivity analysis had the greatest impact on water levels for most of the ranges assessed.

9.4 Optimum Effluent Discharge Rate

Following calibration of the model, the discharge to the proposed RIB area was simulated as a conventional subsurface disposal system approximately 62,200 ft^2 in size. The effluent discharge rates were run at 600,000, 800,000 and 1,000,000 gpd. The different rates were run to determine at what discharge "breakouts" would or would not occur. The existing wetlands are the natural discharge area for 50% of the effluent applied on the site. The remaining discharge will be to the unnamed brook and Ninteteenmile Brook. The existing wetlands drainage is to Nineteenmile Brook. Therefore, all applied effluent wastewater eventually reaches Nineteenmile Brook on, or in very close proximity to, the site boundaries, and leaves as surface water.

The final model simulations indicate at a sustained flow of 600,000 gpd no "breakout" would occur at non-existing groundwater discharge areas. At the higher sustained flow conditions simulations of 800,000 gpd and 1,000,000 gpd the model results indicate the potential for "breakout" to occur in the vicinity of Boring B-7. Groundwater contour elevations of the 600,000, 800,000 and 1,000,000 gpd discharge rates are shown on Figures 16 through 18 (Appendix A). The 600,000 gpd groundwater contour elevation is shown in cross-sectional view on Figures 5 through 7.

9.5 Water Balance, Travel Times and Effluent Chemistry Concentration

A water budget, predicted groundwater travel times to the wetlands and Nineteenmile Brook, along with a solute transport model, were analyzed to predict effluent chemistry concentration in the groundwater over time. The water balance for an annual average effluent discharge of 600,000 gpd is summarized in Table 5 of Watershed Hydrogeologic Inc. February 14, 2007 Memo- Appendix O.

Groundwater travel times (Figure 19 - Appendix A) from the assumed land based disposal area located at the Whitten West site were estimated using a particle tracking analysis. The minimum groundwater travel times from the assumed discharge area to the wetlands, Unnamed Brook, and Nineteenmile Brook are estimated to be approximately one to three months.

The solute transport model MT3D (Zhen, 1990) was used to analyze effluent chemistry concentrations in the groundwater. The treated effluent was arbitrarily assumed to have an initial nitrate concentration of 9.0 mg/l prior to discharge into the subsurface disposal system. The model utilized a constant and uniform discharge of precipitation and effluent. The predicted nitrate plume contour map is shown on Figure 20 - Appendix A. The nitrate concentration (Note: the solute transport model does not account for nor incorporate additional treatment within the soil and wetland soils) reaching the wetlands and Nineteenmile Brook, and leaving the site across the northwest boundary line (Town Line) would be approximately 7 to 8 mg/l due to relatively little dilution with ambient groundwater on the site. However, it is reasonable to predict that additional treatment will occur during percolation through the soils on the site and as the groundwater travels vertically upward through the fine silt, clays and carbon rich organics that comprise the bottom of the wetlands areas. All applied effluent will eventually reach Nineteenmile Brook, where stream flows will further dilute nitrate levels.

SECTION 10

LAND BASED DISPOSAL FEASIBILITY

SECTION 10

LAND BASED DISPOSAL FEASIBILITY

Drilling and test pit excavations and slug tests indicate that highly permeable sand deposits are located on the Whitten West site, beneath the wetlands and between the wetlands and Nineteenmile Brook. Double ring infiltrometer (Section 5.2) and the long-term loading tests (Section 8.2) indicate vertical infiltration rates measured on the Whitten West site are acceptable and conservative for the proposed volume of in-ground effluent disposal. The geologic materials are rather uniform and the vertical and horizontal hydraulic conductivity does not vary significantly in the sand and gravel deposits. The unsaturated thickness is greatest on the top of the kame delta where the disposal system is to be sited. The large thickness of unsaturated soils allows for flexibility in the removal of existing site materials to accommodate the rapid infiltration beds.

The final model predicts at a sustained discharge flow rate of 600,000 gpd the steady state mounding will <u>not</u> rise to within 4 feet of the bottom of the proposed rapid infiltration basins. The minimum separation distance between the highest groundwater mound elevation and the bottom of the proposed rapid infiltration basins is estimated to be approximately 17 feet. The kame delta deposits are hydraulically connected to the sand and gravel located beneath the wetland as well as Nineteenmile Brook. The model predicts at an annual average discharge rate of 600,000 gpd the steady state mounding will not "break out" at non-existing groundwater discharge areas. Approximately 50% of the effluent discharge will flow vertically upward through the silt, clay, sand and carbon rich organics that underlie the existing wetlands. The remaining 50% will discharge to the unnamed brook and Nineteenmile Brook.

10.1 Potential Impact Assessment

The final model can be used to predict onsite impacts such as to wetlands and Nineteenmile Brook. However, due to the localized affects of the mounding, significant impacts offsite are not anticipated.

10.1.1 Nearby Private and Public Wells

As discussed in Section 3.5, private wells exist west of the site along Route 109A (Figure 10 - Appendix A) and are over 0.5 miles from the site. Public well supplies do not exist within 1.0 miles of the site. The high level of effluent treatment afforded by the wastewater treatment facility, the additional treatment within the site soils, the location of private wells over 0.5 miles from the site and model-predicted flow paths make deleterious impacts to private wells highly unlikely. All discharged effluent reaches Nineteenmile Brook on, or within a very short distance of, the site boundary, resulting in minimal off-site groundwater impacts.

10.1.2 Wetlands

The model predicts approximately 50% of the treated effluent will discharge through the existing wetlands. Significant additional treatment should occur as the groundwater travels through the sand deposits and vertically upward through silt, clay, sand and carbon rich organics that underlie the wetlands. Ponding or measurable increases in the size of the wetland areas will not occur due to the steep site slopes. Adverse impacts of effluent chemistry upon the wetlands are not anticipated, but also difficult to assess at this time. Long term monitoring of the wetlands would be necessary to accurately assess wetland impacts.

SECTION 11

CONCLUSIONS AND RECOMMENDATIONS

SECTION 11

CONCLUSIONS AND RECOMMENDATIONS

As a result of field investigations and numerical modeling, Wright-Pierce concludes that land based discharge using rapid infiltration basins of an annual average rate of 600,000 gpd of treated effluent is feasible on the Whitten West site. Numerical modeling results indicate that the anticipated groundwater level mound under the proposed discharge area would be well below the bottom of the proposed rapid infiltration basins. The highest predicted elevation of the mound at the 600,000 gpd annual average design flow is approximately 643 feet. The rapid infiltration basins floor elevation will be maintained at or above 660 feet, thereby resulting in at least 17 feet of separation between the RIB and groundwater mound. Even at the higher modeled flows of 800,000 gpd and 1,000,000 gpd the separation distance would be at least 13 feet.

Impacts to private wells and public well supplies will not occur. The model predicts "break outs" will not occur at non-existing groundwater discharge areas. Increased flooding or ponding of the existing wetlands will not occur. However, changes to the wetlands due to the chemistry of the effluent discharge although not anticipated, are difficult to predict. Therefore, long term monitoring of the wetlands may be required depending upon the environmental significance of the wetlands.

The load test on the wick proved very successful and demonstrates the ability of wicks to discharge large quantities of highly treated effluent within a very small surface area.

The potential for arsenic leaching from site soils due to the proposed discharge was evaluated. The results indicate that the proposed treated effluent discharge is not anticipated to release measurable amounts of arsenic into ground waters or surface waters.

SECTION 12

REFERENCES

Fetter, C.W., 2001, 4th Edition, Applied Hydrogeology, Prentice Hall, 598 p.

Freeze, R.A. and Cherry, J.A., 1979, Groundwater, Prentice Hall, 604 p.

Goldthwait, R.P., 1968, Surficial Geology of the Wolfeboro-Winnipesaukee Area of New Hampshire.

Harbaugh, A.W., Banta, E.R., Hill, M.C., and McDonald, M.G., 2000, MODFLOW-2000, The U.S. Geological Survey Modular Ground-Water Model-User Guide to Modularization Concepts and the Ground-Water Flow Process, U.S. Geological Survey, Open-File Report 00-92.

Harte, P.T., and Mack, T.J., 1992, Geohydrology of, and simulationof ground-water flow in, the Milford-Souhegan glacial-drift aquifer, Milford, New Hampshire: U.S. Geological Survey Water-Resources Investigations Report 91-4177.

Lyons, J.B., Bothner, W.A., Moench, R.H., and Thompson, J.G., Bedrock Geology Map of New Hampshire, 1997.

Moore, R.B. and Medalie, L., 1995, Geohydrology and Water Quality of Stratified-Drift Aquifers in the Saco and Ossipee River Basins, East-Central New Hampshire: U.S. Geological Survey Water-Resources Investigations Report 94-4182.

Poeter, E.P and M.C. Hill, 1998, "Documentation of UCODE, A Computer Code for Universal Inverse Modeling," U.S. Geological Survey Water Resources Investigations Report 98-4080.

Roy F. Weston, 1993, North Conway Water Precinct Rapid Infiltration Basin Study.

USEPA, 1977, Technology Transfer Process Design Manual for Land Treatment of Municipal Wastewater, EPA 625/1-77-008.

USEPA, 1981, Technology Transfer Process Design Manual for Land Treatment of Municipal Wastewater, EPA 625/1-81-013.

USEPA, 1984, Process Design Manual, Land Treatment of Municipal Wastewater, EPA 625/1-81-013a.

USDA, 1954, Soil Survey of Carrol County, New Hampshire.

USDA,Natural Resource Conservation Service, 2006, New Hampshire Soil Attribute Data, Carroll County, New Hampshire.

Waterloo Hydrogeologic, Inc., 2005, AquiferTest.

Zheng, C., 1991, "PATH3D. A Groundwater Path and Travel-Time Simulator". S. S. Papadopoulos & Associates, Inc., Rockville, Maryland.

Wright-Pierce

Civil and Environmental Engineering Services

Appendixes A-N are Omitted

APPENDIX O

WATERSHED HYDROGEOLOGIC MEMORANDUM

MEMO

Date: February 14, 2006

To: Gary Smith, Wright Pierce

From: Jesse Schwalbaum, Watershed Hydrogeologic

Subject: Wolfeboro, New Hampshire Wastewater Discharge Groundwater Model

As you requested, I have developed a groundwater flow model to evaluate the potential impacts of a proposed wastewater discharge site in Wolfeboro, New Hampshire. This memo describes the development of that model and the results.

The potential wastewater discharge site is located in the western portion of Wolfeboro, north of Nineteenmile Brook and immediately east of the border with the Town of Tuftonboro. The site is on a ridge underlain by relatively coarse glacial outwash sediments that is surrounded to the north and east by glacial till and bedrock. Subsurface borings and test pits conducted at the site show relatively coarse sand deposits near the center of the site, beneath the ridge, but finer sands and silts in the lower elevations to the south. The data from these borings and water levels obtained from on-site wells were incorporated into the groundwater flow model.

Modeling Method

The groundwater flow model used in this analysis is a modification of the U.S. Geological Survey's three-dimensional groundwater flow model MODFLOW. This modified version of MODFLOW is called MODFLOW-SURFACT (MSFT) (HydroGeoLogic, 1996 and 2006). The model was developed by HydroGeoLogic, Inc. This modified version of MODFDLOW was used because it allows for simulation of unsaturated groundwater flow. The standard version of MODFLOW can only simulate saturated groundwater flow. This ability to simulate unsaturated flow was important because much of the area is unsaturated under natural conditions and much of the groundwater flow occurs under unsaturated conditions. Groundwater Vistas (developed by Environmental Simulations, Inc.) was used as a graphical processor for the model.

Conceptual Model of Aquifer

The aquifer is simulated as a simple water table aquifer, although much of the area is unsaturated under existing conditions. The extent of the sand and gravel aquifer was defined by Wright Pierce. Nineteen Mile Brook serves as specified head boundary in the south as does an unnamed brook to the east. There are numerous wetlands and groundwater seeps (springs) at the break in slope about three quarters of the way from the top of the ridge to Nineteen Mile Brook. These groundwater seeps and springs serve as discharge areas for groundwater flow. They begin to appear along the slope at an elevation of about 580 to 560 feet. The till and bedrock to the north and east comprises a no-flow boundary. The western boundary is also a no-flow boundary because it is parallel to the dominant groundwater flow direction. The source of water to the model area is exclusively through rainfall recharge. Water leaves the model through the river nodes simulating Nineteen Mile Brook and the drain nodes that represent the unnamed brook, groundwater seeps and wetlands.

Model Design and Development

The area of the groundwater model extends northward from Nineteen Mile Brook to the northernmost extent of sand and gravel deposits, and from the unnamed brook in the east to approximately 500 feet across the town line into Tuftonboro. The model extent and boundary conditions are shown in Figure 1. The area of the model is approximately 62 acres (1,800 by 1,500 feet). The model grid consists of 77 rows 92 columns and 1 layer. The model nodes are a uniform 20 feet by 20 feet across the model domain.

Because the saturated portion of the aquifer is relatively thin (0 to 20 feet over much of the model domain) the entire vertical thickness of the model was simulated in one layer. The bottom of the simulated aquifer represents relatively impermeable till or bedrock.

Model Input Parameters

Hydraulic Conductivity
The hydraulic conductivity (K) values for the aquifer soils were based primarily on soil grain size analyses conducted by Wright Pierce. K values in the model ranged from 10 ft/day (silty sand) to 100 ft/day (coarse sand). A vertical to horizontal hydraulic conductivity ratio of 0.2 was used throughout the model. The distribution of K values is illustrated in Figure 2.

Aquifer Bottoms
As noted above, the simulated bottom of the aquifer represents the elevation of glacial till or bedrock, both of which are considered to have very low hydraulic conductivities. The bedrock/till surface contours were based on data from borings and were provided by Wright Pierce. The distribution of aquifer bottom elevations within the model is shown in Figure 3.

Recharge
At the time that water levels were measured at the site, groundwater levels throughout New Hampshire were at or near seasonal high levels. Average annual recharge to the aquifer from rainfall would be expected to be in the range of 16 to 18 inches per year. However, to account for the high groundwater levels, annualized recharge rates were assumed to be 22 inches per year. This provided the best fit to observed conditions.

MEMO

Brooks, Wetlands and Groundwater Seeps

Nineteen Mile Brook was simulated as a series of river nodes. The gradient along the brook was determined on the basis of topographic survey data. This survey data was used to assign head values at the river nodes. The brook bottom sediments were composed of sandy soils and were assigned a vertical hydraulic conductivity of 2 ft/day.

The unnamed brook to the east was simulated as a series of drain nodes. The difference between the drain nodes and river nodes is that the river nodes can provide water to the aquifer if water levels drop below the stage of the river. This does not occur anywhere in the model so the drain nodes and river nodes act identically. The elevations of the brook were determined on the basis of a site topographic survey and the vertical hydraulic conductivity of the brook bottom was assigned a value of 2 ft/day.

Groundwater seeps and wetlands begin to appear at the site approximately three quarters of the way from the top of the ridge to Nineteen Mile Brook. These seeps and wetlands serve as locations of groundwater discharge and were also simulated using drain nodes. The wetland and seep locations were provided by Wright Pierce and the elevation of the drains were based on detailed site topographic maps.

Model Calibration

The model was roughly calibrated to observed water levels obtained on December 12, 2006. This was a period of relatively high water levels throughout New Hampshire and is probably representative of seasonal high groundwater levels. The steady state model calibrated best using a annualized recharge rate of 22 inches per year. This is significantly higher than the expected average recharge rate but provides the amount of recharge needed to match the seasonal high groundwater levels.

The model was calibrated to observed water levels in six wells under assumed steady state conditions. The resulting groundwater contours are shown in Figure 4. A summary of observed and model simulated water levels at the six wells is presented in Table 1. Four of the wells were simulated within one foot of the observed level, one wells was almost within one foot of observed conditions and one well was within 2.5 feet of the observed level.

Several statistical techniques are used to evaluate how well a model has been calibrated to observed conditions (Anderson & Woessner, 1992). One is to calculate the mean error, which is the mean of the residuals (the difference between observed and simulated groundwater elevations). Table 1 lists the residuals at the 6 observation wells. The mean of residuals was found to be 0.38 foot.

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One drawback to using the mean residual to evaluate calibration is that positive and negative residuals tend to cancel each other out, making the calibration appear better than it might be. A better overall indication of error is the mean absolute residual. For this, the mean is taken of the

Table 1
Summary of Model Calibration Statistics

Well No.	Observed Head (ft.)	Computed Head (ft.)	Residual (ft.)
MW-15	557.70	555.34	2.36
MW-13	581.37	581.76	-0.39
MW-7	648.69	648.10	0.59
MW-11	553.56	553.22	0.34
MW-6	600.30	601.46	-1.16
MW-8	581.24	580.67	0.57
	Residual Mean	0.38	
	Res. Std. Dev.	1.08	
	Sum of Squares	7.89	
	Abs. Res. Mean	0.90	
	Min. Residual	-1.16	
	Max. Residual	2.36	
	Range in Target Values	95.13	
	Std. Dev./Range	0.011	

absolute values of all the residuals. The mean absolute residual for the model calibration was 0.90 feet.

Another measure of the accuracy of a model calibration is the residual standard deviation divided by the range of head values. The residual standard deviation for the calibrated model was 1.08 ft. The observed range in head was 95.13 ft. Therefore, the residual standard deviation divided by the range in head is 0.011 ft.

By all of these measures, the model calibration is considered to be quite good for the scale and head difference within the model. The primary difficulty in calibration was at well MW-15. No reasonable values of hydraulic conductivity, recharge or aquifer bottom elevation (the primary parameters in the model) resulted in a close match to the water level at this well. It may be that the water level at this well reflects a transient condition that was not incorporated in the model. For example, infiltration from a relatively recent heavy rainfall may have been working its way through the aquifer when these water levels were obtained. Recharge events are likely to move rapidly through the upper, unsaturated portion of the aquifer and raise water levels in the saturated lower portion of the aquifer. At any rate, there is some uncertainty with respect to the groundwater flow in the vicinity of this well. As a result, the hydraulic conductivity values in this area of the model are quite low and under discharge conditions the model will likely predict higher heads than might be expected. This makes the model conservative with respect to potential impacts.

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Sensitivity Analysis

A sensitivity analysis is applied to numerical groundwater flow models in order to evaluate which parameters have the greatest impact on the model output.

The sensitivity analysis was undertaken by applying a sensitivity analysis module of Groundwater Vistas (Environmental Simulations, Inc., 2004). This module automatically subjects the model to an extensive sensitivity analysis. Each significant parameter was adjusted by specified increments (up to 50% higher and lower than the calibrated values) and the differences in water levels throughout the model were compared. The sensitivity is represented as a sum of squared residuals, that is, the differences in water levels for each parameter change are summed and squared so that they are positive numbers that can be compared. The parameter sensitivities for the model are summarized in the graph shown in Figure 5. The sensitivity analysis was conducted on the five hydraulic conductivity zones (Kx1 through Kx5) and recharge. Based on the graph in Figure 5 it is clear that the model is most sensitive to the hydraulic conductivity (K_{x1}) in zone 1 which is the area beneath the proposed discharge beds. The second most sensitive parameter was hydraulic conductivity zone 4, which covers the southernmost portion of the model. The model is least sensitive to recharge rates

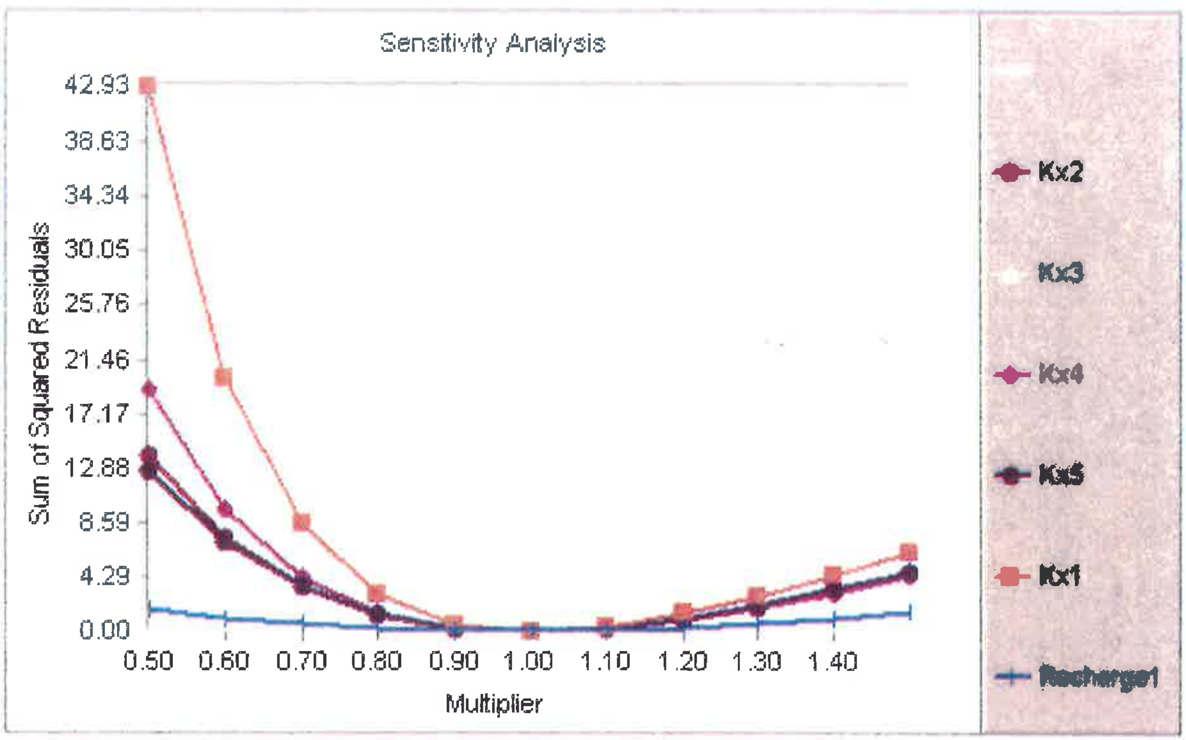

Figure 5
Summary of Sensitivity Analysis

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Model Simulations

Discharged wastewater was simulated as additional recharge over the potential discharge area. It is assumed that treated effluent will enter the ground by means of surface discharge to rapid infiltration beds RIBs). The location of the proposed discharge is illustrated in Figure 6. The total discharge area is approximately 62,000 square feet or 1.4 acres. The proposed maximum discharge rate for the entire area is 600,000 gallons per day, which was simulated as a steady state condition in order to provide a conservative analysis. In addition, the discharge was simulated under high recharge conditions that would be representative of ambient high groundwater conditions.

The model-simulated water levels under a discharge of 600,000 gpd is illustrated in Figure 6. The maximum groundwater elevation resulting from the waster discharge is approximately 643 feet. This is well below the corresponding ground surface of approximately 680 feet. The model indicates that there will be no break-out of groundwater along the steep south slope of the ridge. Groundwater will continue discharging to the existing groundwater discharge areas associated with wetlands and groundwater seeps at the break in slope. There will, however, be greater rates of discharge to these areas.

The model was also used to simulate potential discharge rates of 800,000 and 1,000,000 gpd. The primary difference between these simulations was the height of the mound beneath the discharge beds and the amount of water discharging to the discharge areas. The maximum groundwater elevations beneath the discharge beds were 645 and 647 feet, respectively. There was no break-out of groundwater along the steep slope of the ridge for the 600,000 gpd scenarios but there does appear to be some minor potential for breakout at discharges of 800,000 gpd and greater. The quantity of discharge to wetlands and the brooks was proportional to the effluent discharge.

A particle tracking analysis, using PATH3D (Zheng, 1991), was used to evaluate the where effluent-impacted groundwater would eventually discharge and the associated groundwater travel times. The results of the particle tracking analysis for the 600,000 gpd scenario are shown in Figure 7. The particle tracks shown in this figure illustrate groundwater flow over the course of two years. It was determined on the basis of particle tracking for shorter time periods that the effluent-impacted groundwater would reach discharge areas (groundwater seeps and wetlands and the unnamed brook) in a minimum of one month, and would reach Nineteenmile Brook in a minimum of about 3 months.

Solute transport modeling was also conducted in order to predict the nitrate concentrations in groundwater as it travels through the aquifer and discharges to surface waters. The program used for this analysis was MT3D (Zheng, 1991). It was assumed that the concentration of nitrate within the discharged effluent was 9 mg/l. A map showing the steady state distribution of nitrate within the aquifer at a discharge rate of 600,000 gpd is shown in Figure 8. MT3D does not simulate unsaturated flow so the model only shoes nitrate movement through saturated portions of the aquifer. Fortunately under discharge conditions most of the aquifer is saturated. However, it should be kept in mid that the solute transport model does not predict nitrate concentrations in unsaturated areas. This accounts for the difference between the shape of the nitrate plume and the shape of the particles tracks in Figure 7.

The solute transport model predicts that the concentration of nitrate will remain relatively high throughout most of the flow path of the effluent-impacted groundwater because there is relatively little dilution within the aquifer. Concentrations at the outer edges of the plume will be lower because in these areas there will be more dilution. The concentration of nitrates in groundwater that discharges to the wetlands and brook is predicted to be on the order of seven to eight mg/l.

A comparison of quantity of groundwater flowing to the discharge areas with and without the proposed wastewater discharge was conducted. The discharge areas were divided into five separate areas – three areas of groundwater seeps and wetlands, Nineteen Mile Brook and the unnamed brook to the east. The three groundwater seep/wetland areas are: 1) the southeastern area between Nineteen Mile Brook and the unnamed brook, 2) the central seep/wetland area south of MW-15 and 3) the western seep/wetland area southwest of MW-13. A summary of model-simulated ambient flows and model-simulated flows with 600,000 gpd of wastewater discharge is presented in Table 2.

Table 2
Summary of Groundwater Flows to Surface Water (Gallons Per Day)

Surface Water Body	Model Simulated Ambient Condition (GPD)	Model Simulated Condition with 600,000 GPD Discharge
Nineteen Mile Brook	89,000	166,000
Unnamed Brook	3,000	181,000
Western Wetland/Seep	0	208,000
Central Wetland/Seep	0	60,000
Eastern Wetland/Seep	3,000	74,000

Summary and Conclusions

In our opinion, the groundwater flow model simulates existing and potential groundwater conditions at the site reasonably well. The unsaturated model was necessary to simulate existing conditions but once the wastewater discharge was simulated the majority of the model functions as a normal MODFLOW simulation.

M E M O

There is some uncertainty incorporated in the model (particularly in the area of MW-15) that is a reflection of existing hydrogeologic data. On the other hand, the model is conservative with respect to predicting potential impacts. In particular: 1) the hydraulic conductivity values are relatively low for the observed sediments and this will result in higher predicted groundwater levels, 2) the proposed discharge is simulated under high recharge (and groundwater level) conditions and 3) the model is simulated as a steady state condition when in practice the discharge will not be kept continuously at the maximum rate. In addition, the model reflects the relatively simple geometry of the aquifer – the source and ultimate discharge areas of groundwater are well-defined.

The model predicts that the site can serve as a discharge area for up to 600,000 gpd of treated wastewater effluent and still maintain an appropriate level of separation between the discharge beds and the top of the water table mound. The model also shows no significant potential for new locations of groundwater discharge along the steep southern slope of the ridge. There will be significantly more discharge to the groundwater seeps, wetlands and brooks surrounding the site. However, the effluent-impacted groundwater will have traveled at least one to three months through the groundwater system before discharging at the surface.

Simulations at higher discharge rates were also run and there appears to be some potential for breakout at higher discharge rates. Therefore, we would recommend that the site not be considered for higher rates until after the system has been operating for some time and new data has been collected to confirm the model results and the carrying capacity of the brooks and wetlands.

REFERENCES

Anderson, M.P. and W.W. Woessner, 1992, Applied Groundwater Modeling, AcadPress, Inc., San Diego, California.

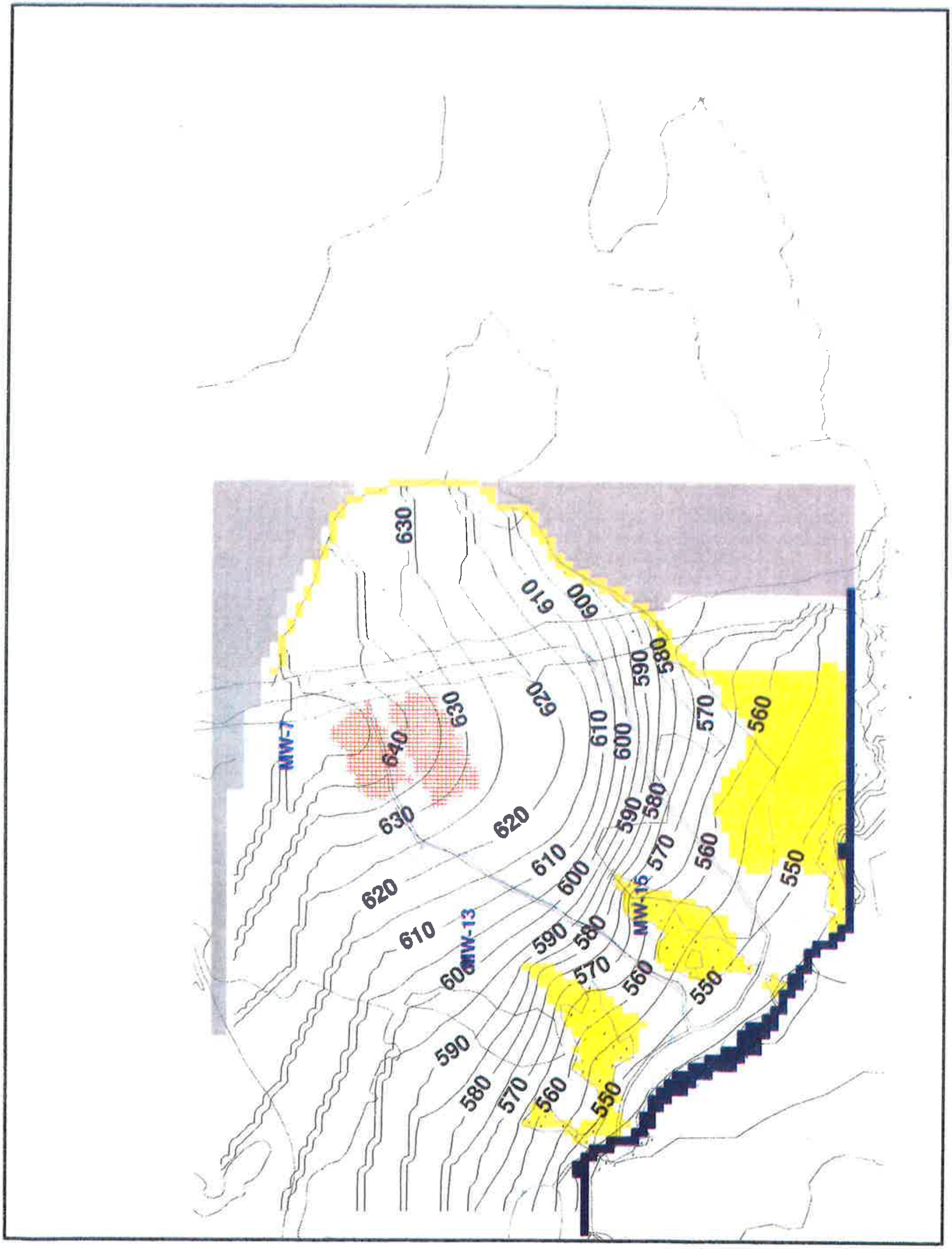
Environmental Simulations, Inc., 2004, Groundwater Vistas Manual.

HydroGeoLogic, Inc., 2006, MODFLOW-SURFACT (Version. 3), The HydrGeoLogic Enhanced Version of the U.S. Geological Survey Modular Groundwater Flow Model, Code DFocumentatin Report.

McDonald, M.G. and A.W. Harbaugh, 1988, "A Modular Three-Dimensional Finite Difference Groundwater Flow Model", Techniques of Water Resources Investigations 06-A1, USGS.

Zheng, C., 1989, PATH3D: A ground-water path and travel-time simulator, S.S. Papodpulos and Associates, Inc., Rockville Maryland.

Zheng, C., 1990, MT3D: A modular three-dimensional transport model for simulation of advection, dispersion and chemical reactions of contaminants in ground-water systems, U.S. EPA, R.S. Kerr Environmental Research Laboratory, Ada, Oklahoma.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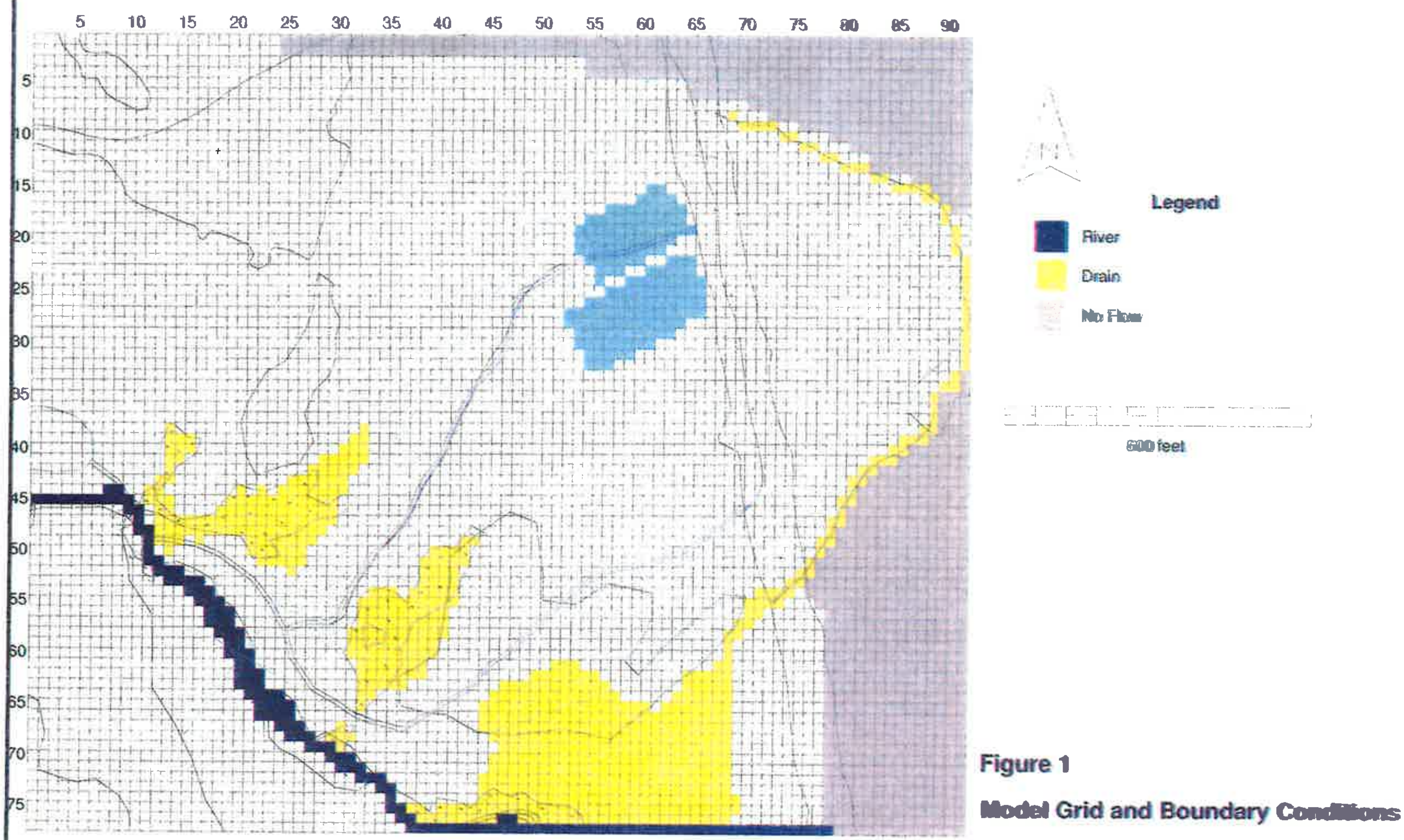

Figure 1

Model Grid and Boundary Conditions

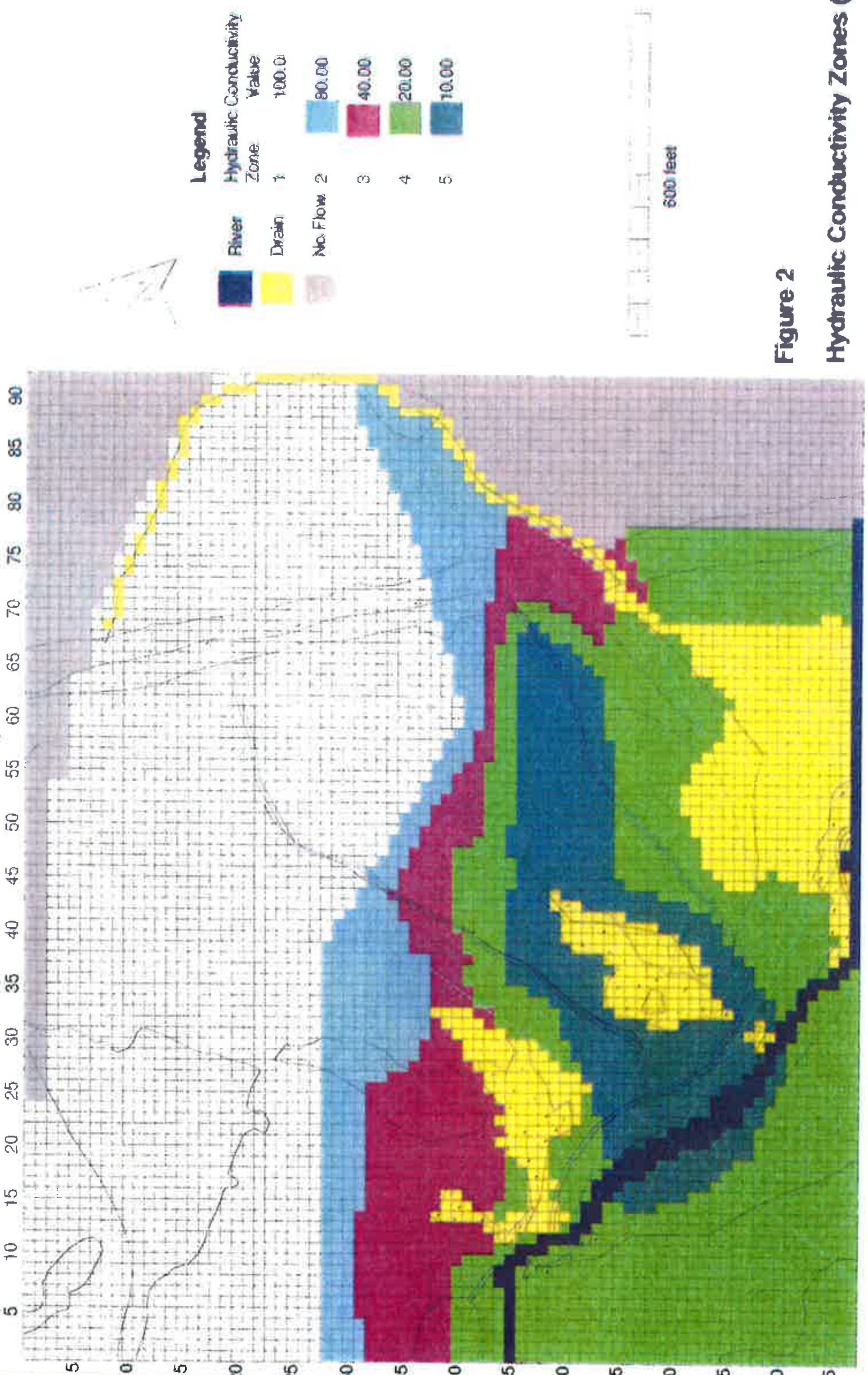

Figure 2

Hydraulic Conductivity Zones (ft/day)

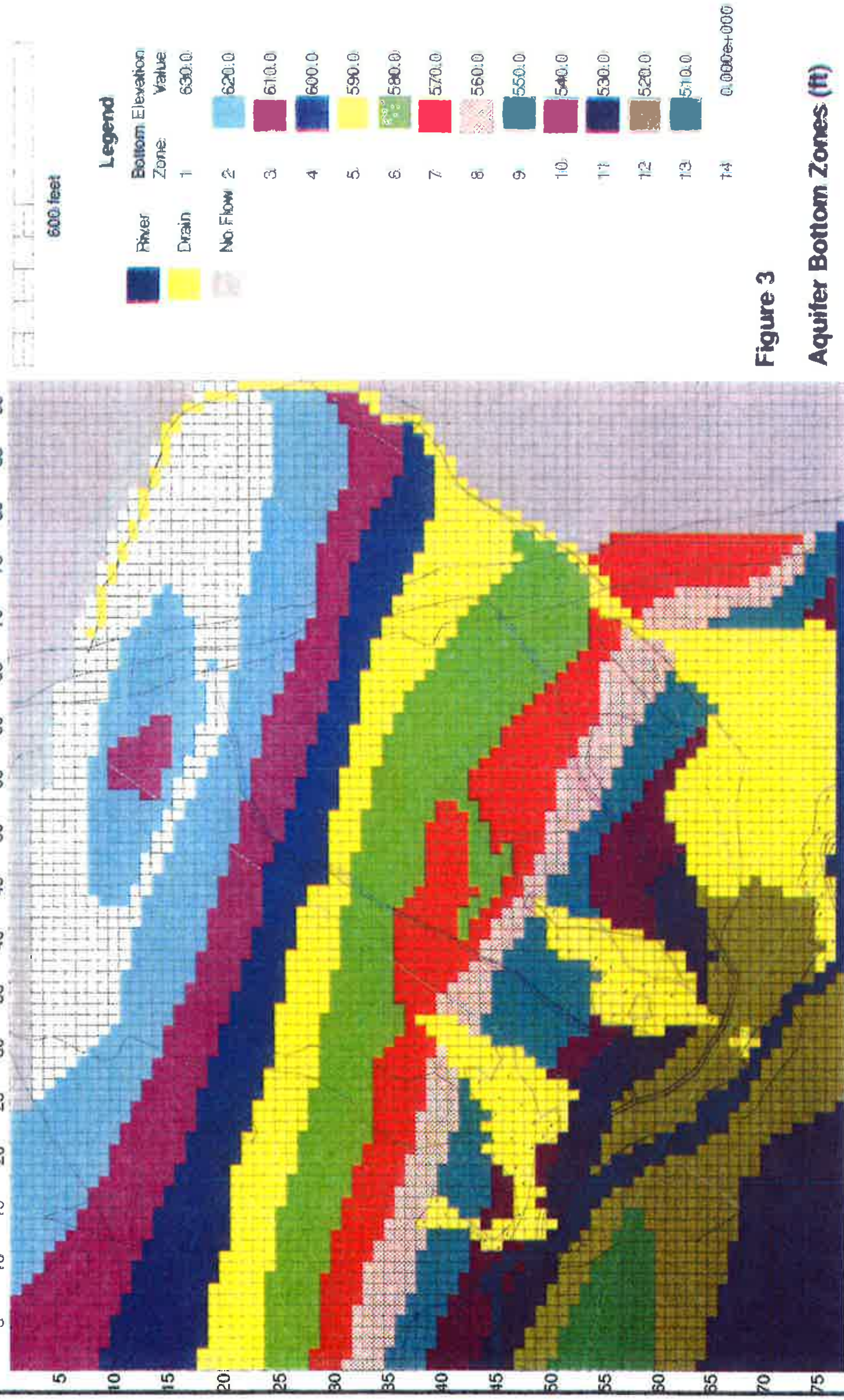

Figure 3

Aquifer Bottom Zones (ft)

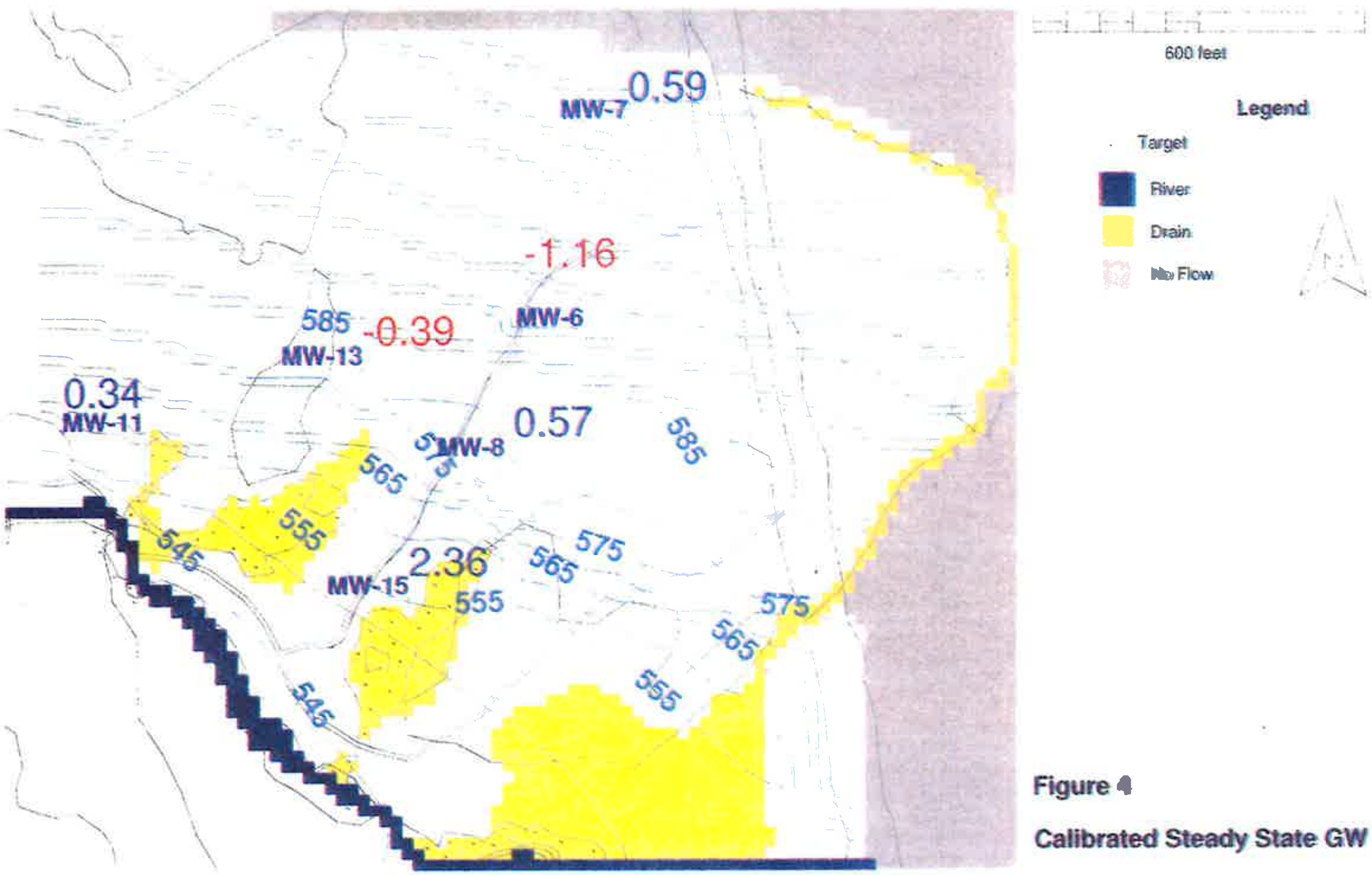

Figure 4

Calibrated Steady State GW Contours

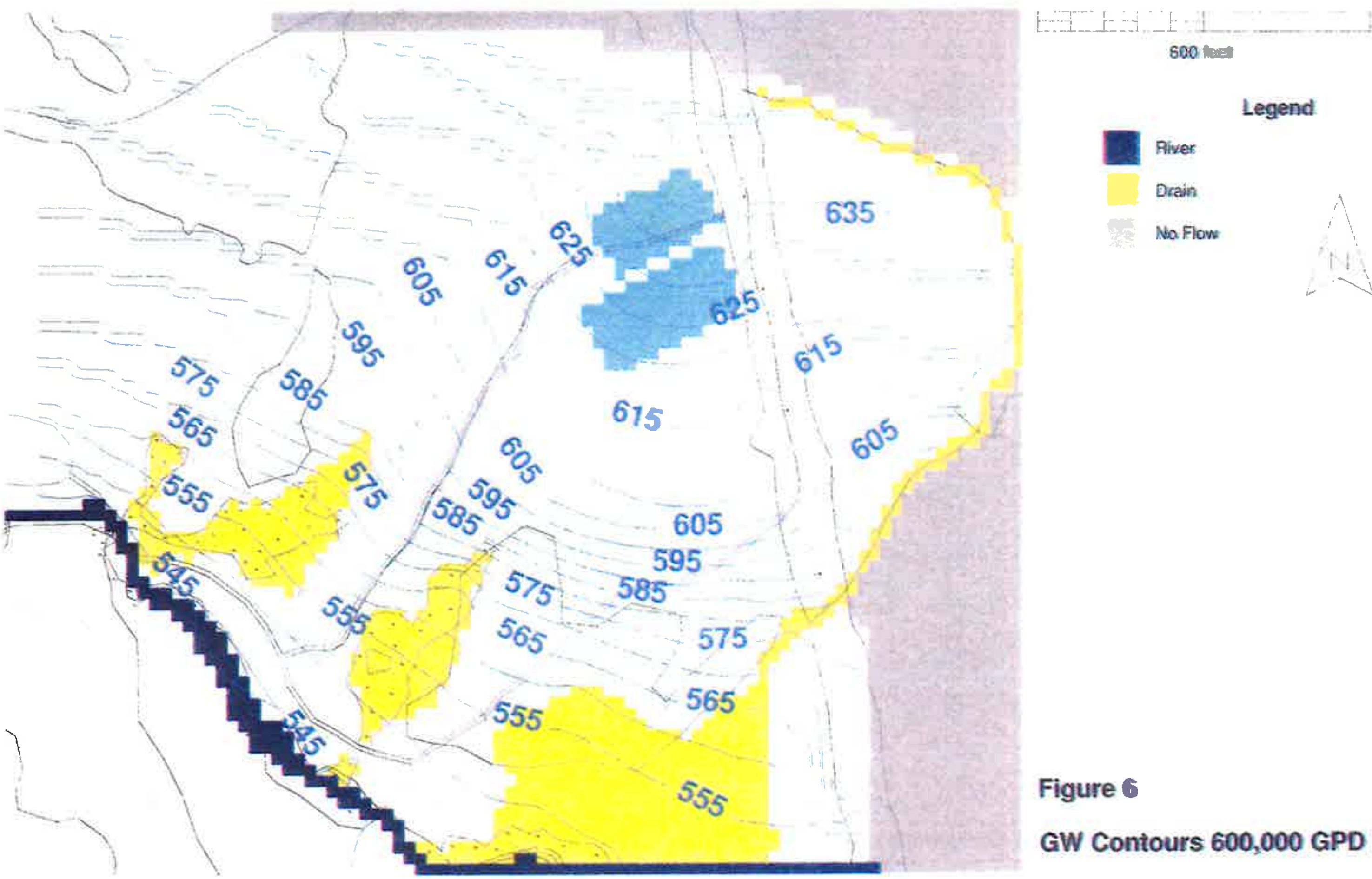

Figure 6

GW Contours 600,000 GPD Discharge

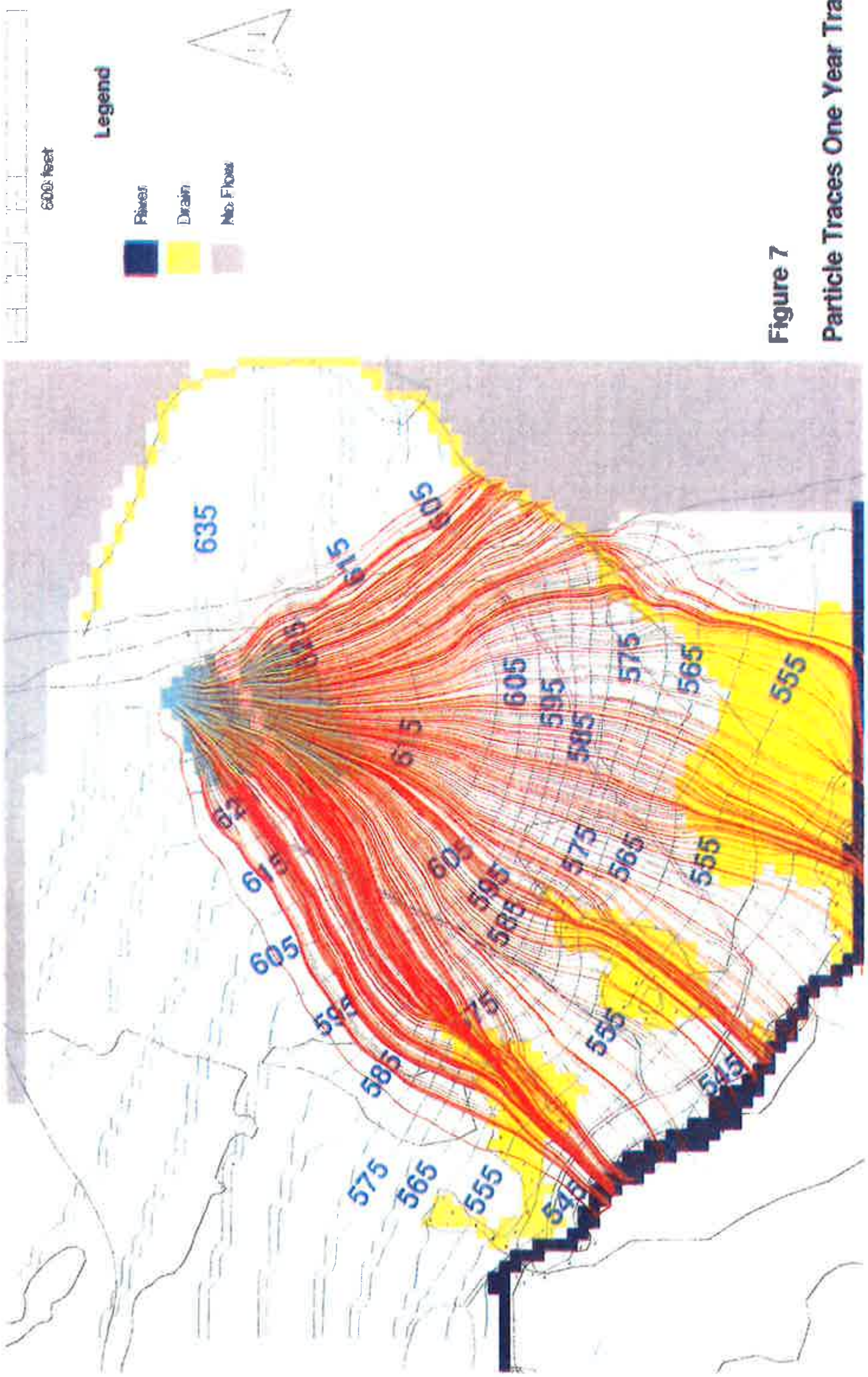

Figure 7

Particle Traces One Year Travel Time

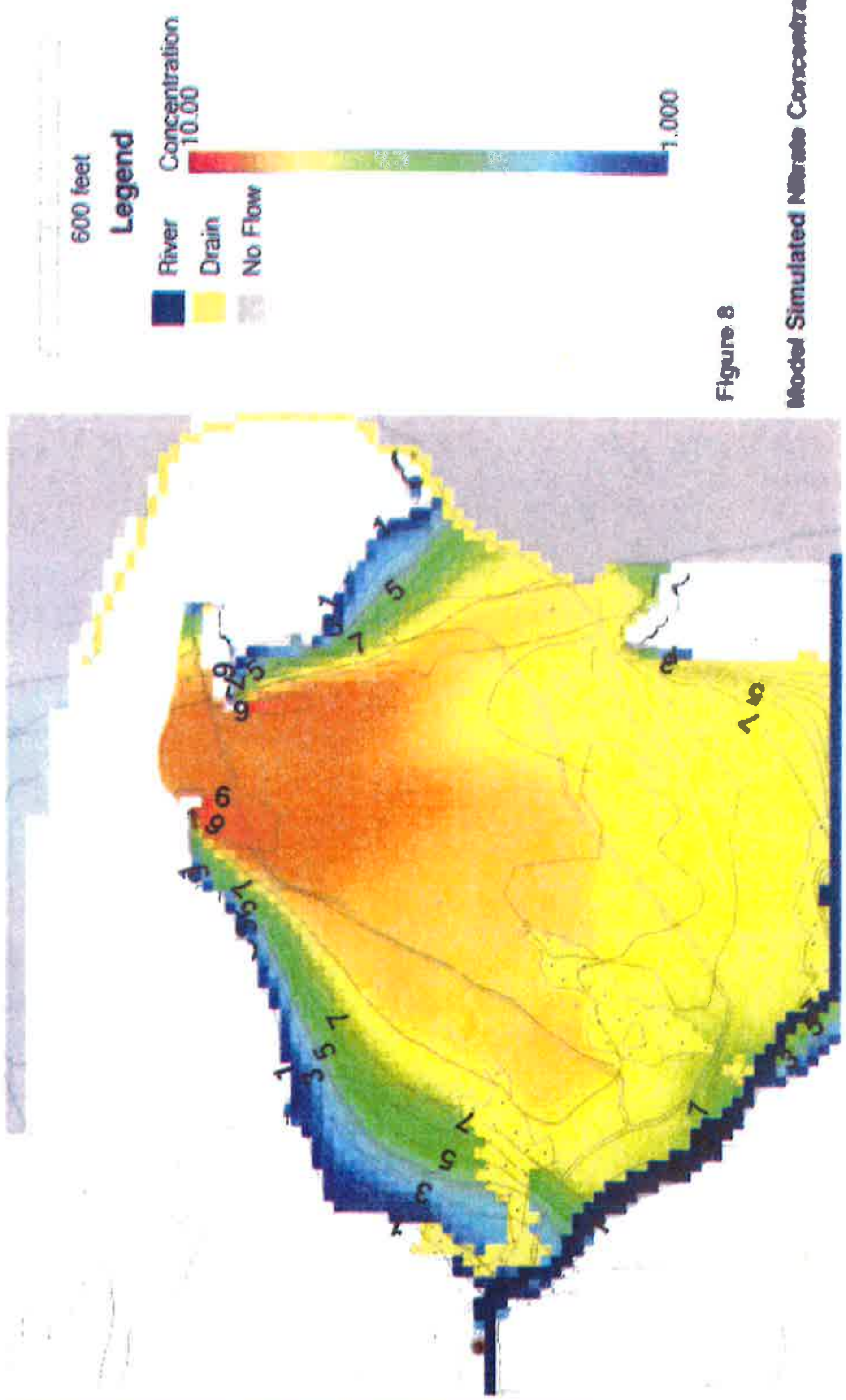

Figure 8

Model Simulated Nitrate Concentrations