TOWN OF ORLEANS
COMPREHENSIVE WASTEWATER MANAGEMENT PLAN

PRELIMINARY SITE INVESTIGATIONS HYDRO GEOLOGIC EXPLORATION AT SITE 241

APPENDIX E

FEBRUARY 2008

WRIGHT-PIERCE
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ORLEANS COMPREHENSIVE WASTEWATER MANAGEMENT PLAN
PHASE 6 - SITE INVESTIGATIONS HYDROGEOLOGIC INVESTIGATION
AT SITE 241

1.0 INTRODUCTION

Assessments conducted as part of the Comprehensive Wastewater Management Plan (CWMP) indicate that the Town of Orleans has a current need for 1.2 million gallons per day (mgd) of effluent disposal capacity, and a potential future need of 1.6 mgd. To date, 26 sites have been identified whose potential aggregate capacity is about 4 mgd. Many of the identified sites are small and privately owned. Wright-Pierce has recommended that the Town defer any study of these sites until more is known about the capacity of the larger sites, particularly those in public or utility ownership.

This report focuses on the results of exploration at Site 241, the site of the Tri-Town Septage Treatment Facility, see Figure 1. This site was selected as the first property for further evaluation because: 1) it has large portions of undeveloped land (as compared to remaining vacant land available elsewhere in town), 2) it is in municipal ownership, and 3) its current land use is consistent with wastewater activities. This report provides a summary of the work completed in the fall of 2007 at the Tri-Town site in support of the Comprehensive Wastewater Management Plan. The scope of this work is summarized as follows.

- Review of existing subsurface information collected during the construction of the septage treatment facility. See Section 2.0.
- Identification of soil conditions in previously unexplored areas of the site, as well as confirmation of soils in the vicinity of the existing effluent disposal system. See Section 3.0.
- Observation of soil permeability through selective percolation testing in areas of the site previously unexplored. See Section 4.0.
- Execution of a hydraulic loading test to determine the hydraulic capacity of the soil at a large-scale and the associated response of the local groundwater to a significant application of water at the surface. See Section 5.0.
- Preliminary determination as to the capability of the site to handle some or all of the Town's future disposal needs. See Section 6.0.
- Simulation of groundwater mounding using computer modeling based on the results of the hydraulic loading test See Section 7.0.
- Recommendation of future steps for exploration at the Tri-Town site. See Section 8.0.

Legend

Hydrant
Monitoring Wells
Parcels

Test Pits 2007 Percolation Rate

<2 min/inch
>5 min/inch
None

Test Pits by Others Percolation Rate

<2 min/in
2-5 min/in
>5 min/in
* Estimated based on Soil Logs

0 50 100 Feet

SOURCE:
Well Cluster Sites and Test Pits created by Wright-Pierce.
All other data obtained from Town of Orleans and MassGIS.

Orleans CWMP
Site Investigations
Site 241
Site Plan

PROJ NO 10645E	WRIGHT-PIERCE	FIGURE: 1	
DATE Feb. 2008			
SCALE AS SHOWN			

2.0 REVIEW OF EXISTING INFORMATION

Extensive subsurface information for portions of the site is available from the original design and permitting of the septage treatment facility. All available information on soil exploration, permeability testing, borings, test/monitoring wells, and subsurface documentation was reviewed by our staff and used to shape this most recent investigation.

The review of documents included information developed by GZA in 1984 to support the permitting of the existing rapid infiltration basins. That exploration included a hydraulic loading test (in an area adjacent the existing basins) that demonstrated the soils are capable of infiltrating approximately 270,000 gpd.

Also reviewed were test pit data and boring logs compiled as part of USGS study (1995) of the migration of nitrogen-enriched groundwater moving away from the Tri-Town site and its potential impacts on surface water in Namskaket Marsh.

More recently, the Namskaket Marsh system has been studied by the Massachusetts Estuaries Project. Preliminary results indicate that this salt marsh system is not currently nutrient stressed, and therefore not negatively impacted by the nitrogen in the groundwater contributed by all sources in the watershed including the nitrogen that reaches the groundwater from the disposal of effluent at the Tri-Town site.

3.0 SUBSURFACE EXPLORATION

Based on all of this existing information, we developed and then executed a program of additional test pits to define soil types and to determine percolation rates to get preliminary estimates of soil permeability in areas that were, until now, unexplored.

A review of existing subsurface information exposed the lack of information in several portions of the site. These areas include the portion of the site south and southeast of the existing rapid infiltration basins (RIB), and northeast of the treatment facility buildings but south of the compost shed, see Figure 1.

Several test pits were excavated in the southern portion of the site, and more test pits were located as feasible in the region northeast of the treatment buildings, but no exploration occurred at the highest elevation of the knoll. The soil constituting the knoll would likely be removed as part of any construction, and therefore any information collected on surface soils would not be useful in the design of effluent disposal beds. Test pits were also located in the area defined as: west of the infiltration basins, south of the treatment buildings and east of the access road. While there is some information available in that area, it is derived from the logs of the monitoring wells installed by USGS. A better classification of soil in that area was needed. No work was done on the portion of the site to the west of the access road, since that land is low-lying and least suited for effluent disposal. Test pits were also located immediately adjacent to the best- and worst-performing RIBs to better characterize the extent of different soil types, and serve as an additional benchmark in the evaluation of the hydraulic loading test.

A total of thirteen test pits, numbered 1-07 through 13-07, were excavated on the site. The location of these test pits can be seen on Figure 1. The test pit logs are in Attachment 1.

Each test pit enabled the visual observation of soil from the ground surface to a depth 10 or 12 ft below ground surface (bgs). Groundwater was not encountered in any of the test pits. Previous mapping of the site indicates the groundwater table is generally 30 to 40 ft bgs in the areas of exploration. This information was also confirmed when recording the groundwater elevation in monitoring wells surrounding the hydraulic loading test.

The soil in the test pits has been classified as follows. In general, silty fine sand, ranging from 4 ft bgs to 9 ft bgs, was observed in test pits 1-07, 3-07, 6-07, 7-07, 9-07, 11-07 and 13-07. Orange tan medium sand ranging from 4 ft bgs to 9 ft bgs, was encountered in test pits 2-07, 4-07, 5-07 and 8-07. This soil stratum was also observed in test pits 11-07 and 13-07 from a depth of 6 ft to the bottom of the excavation (11-12 ft). Gray silt and clay was encountered in test pits 3-07 and 10-07, ranging from 5 to 6 ft bgs to the bottom of each excavation. Water infiltrating through the soil as part of the hydraulic loading test was observed weeping into test pit 1-07 (approximately 10 ft south of RIB No. 2, the worst performing RIB) at a depth of 7 ft bgs, on top of the dense silty fine sand strata. Soil mottling, apparently from perched groundwater, was observed as several depths.

4.0 SOIL PERMEABILITY

A single percolation test was performed in test pits 4-07 through 9-07 and 12-07, and two percolation tests were performed at different depths in 2-07 (See Figure 1 for locations and Table 1 percolation rates). Conducting a percolation test includes the following: excavating the top layers of organic soil to expose lower horizons, digging a 1-ft by 1-ft hole 1.5 ft deep, filling the hole with water to a depth of 1 ft and allowing the water to soak in while maintaining that water elevation for 15 minutes by adding more water as needed, and measuring the drop in water elevation for the next 15 minutes while no additional water is added. The key to the percolation test is the change in water level elevation during the second 15-minute interval. The change in elevation divided by the time it takes to make the change is called the percolation rate. For example, if the water level dropped 3 inches in 15 minutes, the resulting percolation rate is 5 minutes per inch (mpi). The smaller the value in mpi, the faster the water moves through the soil.

Table 1

Percolation Test Results

Test Pit Number	**Depth** ft	**Percolation Rate** min/in
2-07	4.5	1.5
2-07	9	0.5
4-07	5	0.75
5-07	4	0.5
6-07	6	6.0
7-07	5	6.0
8-07	5	0.5
9-07	5	6.5
12-07	6	0.5

Percolation test results ranged from 0.5 mpi to 6.5 mpi. Percolation tests were not performed in test pits 3-07 and 10-07 due to the presence of silt and or clay. A percolation test was not performed in test pit 1-07 due to the presence of water weeping into the pit from the ongoing hydraulic loading test.

5.0 HYDRAULIC LOADING TEST

A third and significant segment of the study was a hydraulic loading test to stress the soils for a long duration to quantify the upper limit of the local soil to infiltrate water. As this report details, these 3 aspects of our field work: test pits, percolation tests, and a hydraulic loading test, have significantly increased our knowledge of the site capacity, and provides the Town with timely information needed to further their comprehensive wastewater management planning.

Given the land available at the Tri-Town site, and the lack of large, vacant, publicly owned property elsewhere that could be used for future wastewater activities, a large-scale hydraulic loading test at the Tri-Town site was warranted.

5.1 Design Philosophy

The test pits and percolation tests conducted as part of this study have provided some indication of the soil types and soil permeability in several unexplored areas of the site. Relying exclusively on percolation testing allows conservative design loading rates. This is because these evaluations measure the permeability of only a small area of soil to accept a small volume of water. Based on DEP guidance for the type of effluent disposal being considered at the Tri-Town site, soils with good permeability (percolation rates less than 10 mpi) generally correlate to design loading rates of 4 to 5 gallons per day per square foot (gpd/sf) using only the results of percolation testing. However, when considering disposal of large quantities of effluent, a large-scale hydraulic loading test is appropriate to reduce the very high safety factor inherent in design based only on a percolation test.

To better gauge the amount of land that may be required for effluent disposal and determine if the Tri-Town site could accommodate a centralized disposal system, Wright-Pierce designed a long-term hydraulic loading test to get specific information about the soil capacity. The loading tests must demonstrate an application rates higher than the design rates to account for a factor of safety when converting the test results to full-scale design. The following criteria are critical when considering how the loading test conditions will differ from the day-to-day operating conditions:

- **The size of the test basin.** The larger the test area, the more representative the results and the smaller safety factor that can be applied to determine full-scale application rates.

- **Duration of the test.** The longer the test period, the more likely the test results will approximate actual effluent disposal and the smaller the safety factor that is necessary.

- **Application liquid.** It is most convenient to use potable water for the test, but actual wastewater effluent is more representative of ultimate loading conditions.

Considering the factors above, Wright-Pierce proposed the following test conditions to most closely simulate full-scale conditions with the best available resources.

- **Selecting a test basin.** Effluent from the septage treatment facility is currently discharged to a series of basins each approximately 2,000 sf in area. It is usually not practical to construct basins this large for the hydraulic loading test. The Tri-Town staff was able to dedicate one of its basins to the loading test.

- **Running the test.** With the assistance of Tri-Town staff, the test ran for 25 days, without interrupting the operation of the facility.

- **Applying liquid.** With the assistance of Tri-Town staff, all effluent was directed to the basin being used as the test basin to provide test conditions most representative of design conditions. The daily effluent discharge was supplemented with potable water.

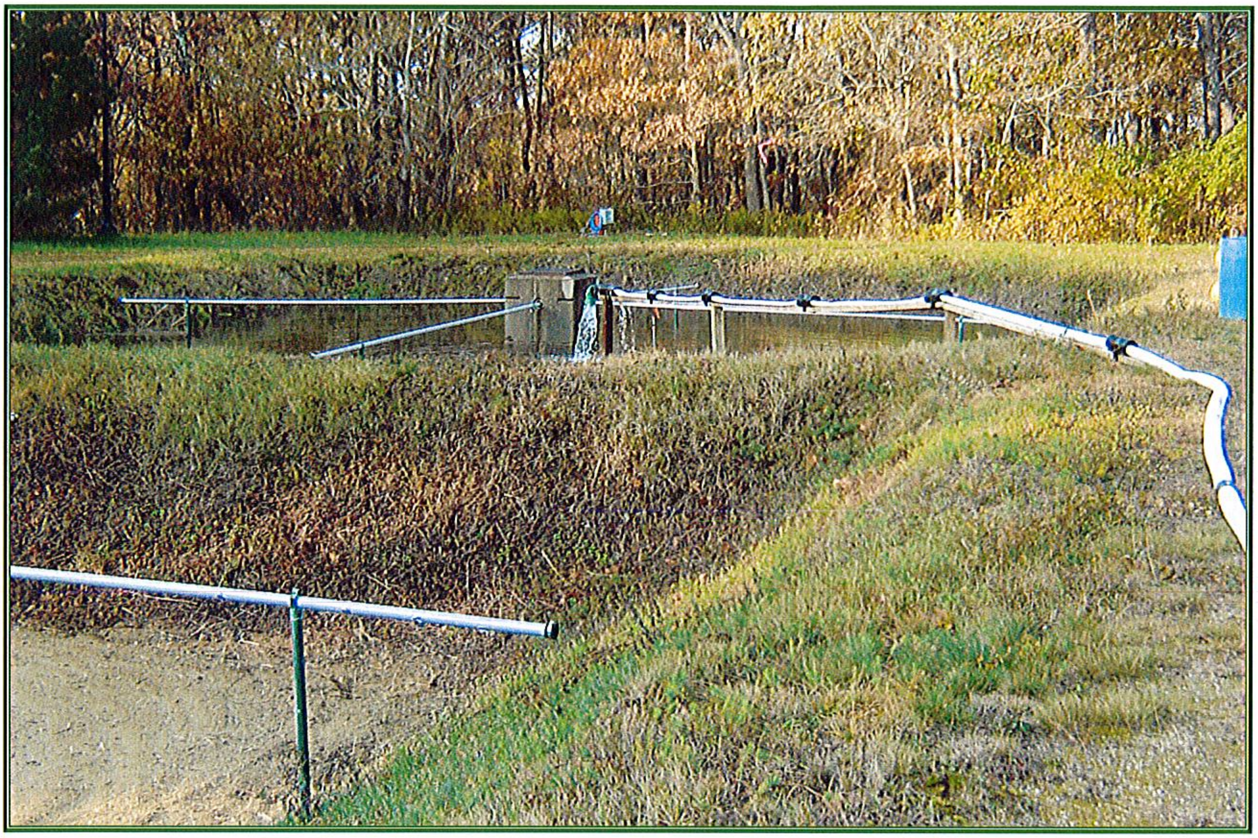
One of the eight existing RIBs (RIB No. 2) was used as the test basin for the hydraulic loading test (see Figure 2). According to Tri-Town staff, RIB No. 2 is one of the worst performing among the eight basins, meaning that effluent percolates through the most slowly. Over the course of the 25-day loading test, 600,000 gallons of treatment facility effluent, and 1,200,000 gallons of potable water was applied to RIB No. 2, resulting in an average loading rate of approximately 30 gpd/sf. The details of the construction, operation, and monitoring of the loading test follows.

5.2 Construction of Loading Test Facilities

Each of the 8 RIBs at Tri-Town is approximately 45-ft by 45-ft (a bottom area of 2,025 sq. ft) and is 5 ft deep. Each of the RIBs contains an effluent dosing system. This system includes a 5-ft concrete post located in the center of the basin, and four perforated pipes that span the distance from the center of the basin to the edge of the basin. When effluent is pumped to the basin from the treatment building, water sprays out the holes in the perforated pipe. During the regular operation of the treatment facility, a basin may be dosed with effluent several times per day for 15 or 20 minutes. Each dose applies a volume of effluent equivalent to a few inches of water if it were applied instantaneously to the basin.

Figure 2

View of RIB No. 2 During the Operation of the Loading Test



RIB No. 2 in the background is partially full. Potable water is being discharged in the center of the bed through the flexible hose. RIB No. 4 (in the foreground) is dry. Only a small earthen berm separates the two beds.

Preparation of RIB No. 2 consisted of clearing vegetation and scarifying the bottom of the bed to expose the sand, and constructing a system to convey potable water the basin. Potable water is available at a hydrant a few hundred feet from RIB No. 2. Flexible hosing ran from the hydrant to the basin. A wooden structure was constructed to support hose from the edge to the center of the basin. This was done to ensure even distribution, to prevent erosion of the side slope, and to keep the hose from coming in contact with the effluent. Figure 3 depicts the test set-up.

5.3 Operation of Loading Test Basin

Tri-Town staff continued with the normal operation of the septage treatment facility during the hydraulic loading test. The hydraulic loading test did not require the staff to suspend or compromise the treatment process. However, the staff did modify the rotation schedule for effluent disposal among the basins. During the hydraulic loading test, all of the effluent was applied to RIB No. 2. During November and December, the facility discharged 15,000 to 20,000 gpd of effluent. During this time frame total suspended solids in the effluent was 5 mg/l or less. The effluent was supplemented with 20,000 to 40,000 gpd of potable water.

Figure 3

Potable Water Being Applied to RIB No. 2

Over the course of the test, 30,000 and 60,000 gpd of applied liquid (potable water and effluent) were required to keep the water level in the basin relatively constant. One goal of the test was to maintain a steady state, that is, the rate at which water was going into the bed was equal to the rate at which water was seeping into the ground.

The discharge of effluent and potable water was measured with flow meters. The effluent flow alternated between zero and a constant rate for a short duration. The discharge rate of potable water could be regulated and therefore varied to maintain a constant level in the basin. A flow meter and regulating valve were attached at the hydrant, and as the water level in the basin rose and fell, the flow rate could be changed.

5.4 Test Monitoring and Coordination

In addition to monitoring the total volume of liquid applied to the bed, and having ready visual observation of water levels in the basin, it was also critical to monitor the groundwater levels below and near the bed. Wright-Pierce provided coordination and monitoring equipment to facilitate groundwater data collection during the test.

Water level data were collected continuously in the 11 wells listed in Table 2 and shown on Figure 1 via InSitu Level Log® pressure transducers. The groundwater monitoring well numbering, elevation range of the well screen interval and distance to RIB No. 2 are summarized in Table 2. Manual water level data were also collected daily in these observation wells. The monitoring wells were chosen based on their proximity to RIB No. 2 and their screened intervals. Wright-Pierce staff identified the location and verified the hydraulic response of the wells. Well BMW-22 was utilized to record ambient water level fluctuations. The manual and electronic water level data are summarized in Attachment 2. Graphical summaries of the manual and electronic water level data are presented in Figures 4 and 5, respectively. The manual readings confirm the accuracy of the electronic measurements.

The shallow screened wells TOMW-3 and -4 were chosen to determine if any mounding would occurred above the discontinuous clay layer located on the western and northwestern portions of the site in the vicinity of Namskaket Marsh.

Precipitation data were collected and recorded daily from the existing Tri-Town rain gauge by Tri-Town staff. Precipitation data were also collected by Wright-Pierce through internet sources from the nearest weather station. The precipitation data are shown on Figures 4 and 5. During the course of the loading test, precipitation accounted for less than 1% of the total volume applied to the basin.

Table 2
Monitoring Well Summary

OSW Monitoring Well I.D.	Associated USGS Well Cluster Site	Screened Interval ft msl[1]	Approximate Distance from RIB No. 2 ft
115	99	-12 to -13	100
108	94	6 to -1	25
137	86	10 to -0.5	200
143	148	1.5 to -3.5	275
178	93	3.5 to -1.5	35
GMW-1	126	17 to -2.5	50
GMW-3	128	20 to 0.0	50
GMW-4	129	20 to 0.0	330
TOMW-2	132	11 to -9	500
TOMW-3	133	10 to -10	650
BMW-22	NA	1.5 to -1.5	6,700

1. Source: USGS Open File Report 95-439

5.5 Hydraulic Loading Test

The design, startup and shutdown of the loading test was a collaborative effort among Wright-Pierce, the Town and Tri-Town staff. Wright-Pierce coordinated water level monitoring through the use of automated data collection instruments. Manual back-up measurements were collected by Tri-Town staff.

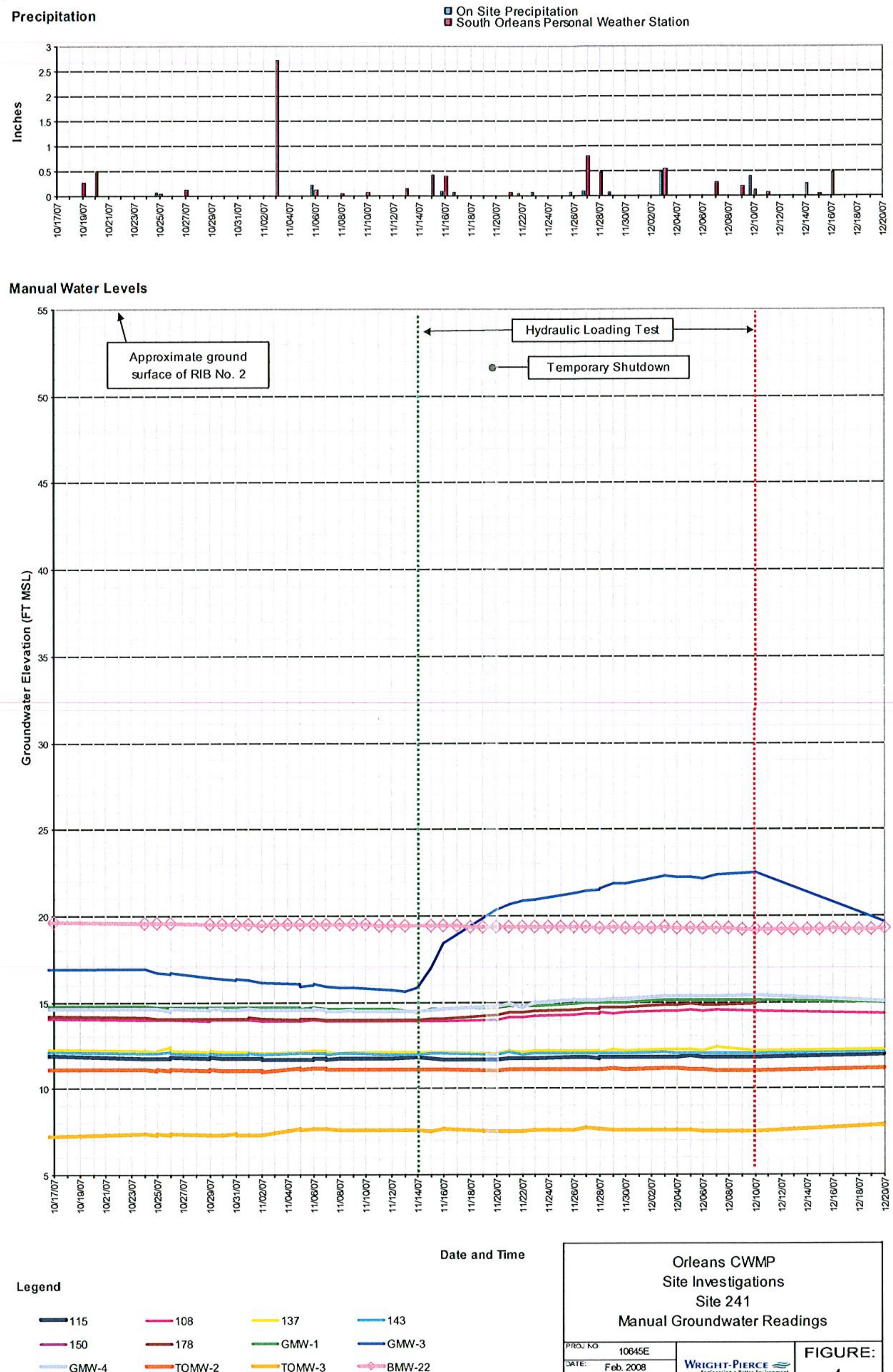
The loading test was comprised of the following monitoring periods: antecedent (26 days); pretest/saturation (2 days); loading (25 days); and recovery period (7 days).

5.5.1 Antecedent Period

Manual water level measurements were obtained by Tri-Town staff twice daily in the monitoring wells listed in Table 1 prior to the start of the loading test from October 17 through November 12, 2007. Water level measurements were also obtained via pressure transducers every fifteen minutes from October 24 through November 12, 2007.

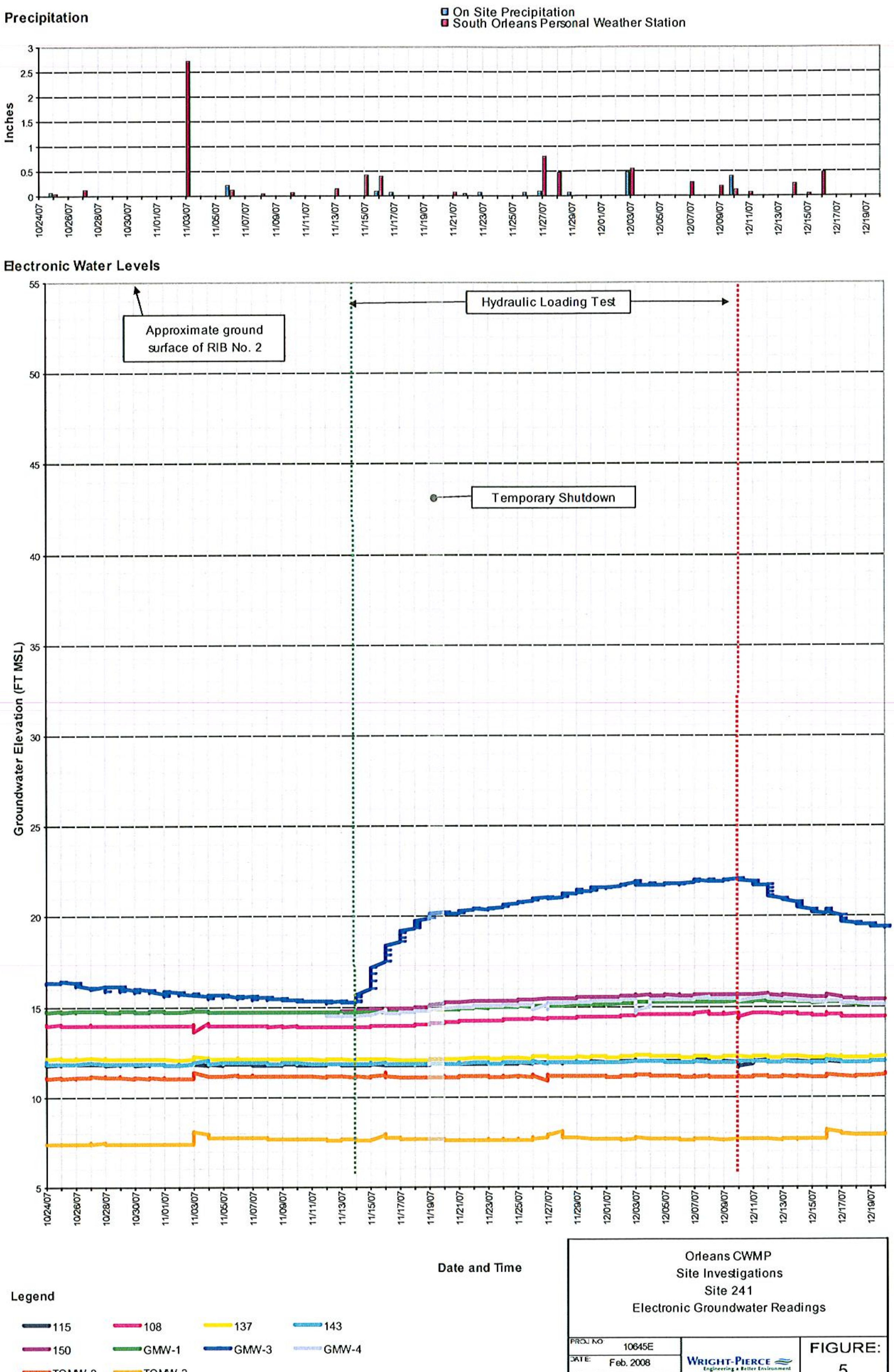
5.5.2 Pre-Test

A pre-test was conducted on November 12, 2007, to determine the appropriate loading rate for the constant rate test. The pretest was conducted at increasing stepped loading rates to



Orleans CWMP
Site Investigations
Site 241
Manual Groundwater Readings

PROJ NO	10645E	WRIGHT-PIERCE Engineering a Better Environment	FIGURE:
DATE:	Feb. 2008		4
SCALE:	As Shown		



Orleans CWMP
Site Investigations
Site 241
Electronic Groundwater Readings

PROJ NO	10645E	WRIGHT-PIERCE Engineering a Better Environment	FIGURE: 5
DATE	Feb. 2008		
SCALE	As Shown		

ensure that flooding did not occur. Discharge rates were adjusted to maintain a constant loading rate and a constant water level over 1- to 2-hour increments. The water levels recorded on November 12, 2007, prior to the start of the pre-test, were used to generate a static water level contour plan. The static water levels and resulting contours are shown in Figure 6. RIB No. 2 was saturated for a period of 24 hours before the loading test began.

5.5.3 Loading Test

The loading test was conducted from November 14, 2007 to December 10, 2007. Water level measurements were obtained via pressure transducers every fifteen minutes in the select monitoring wells listed in Table 2. Tri-Town staff also collected manual water level readings twice a day.

The loading rate varied over the course of the test to maintain an average water elevation in the basin. The water level in RIB No. 2 reached a relatively constant head of 2 ft. Lower volumes of treated effluent are typically generated during the weekends which corresponded to a lower water level in the basin of approximately 1.2 ft.

The loading test was initiated at a rate of 45 gpm of potable water. In addition to the discharge of potable water, an average of 17 gpm of effluent was applied to Basin RIB No. 2. The test was suspended on November 19, 2007 to allow for repair of a nearby gate valve. Over that 5-day period (Nov 14 through Nov 19), a combined average of 60 gpm was discharged to the basin.

The test was restarted on November 20, 2007 at rate of 27 gpm of potable water. After two days, the potable water rate was lowered to 25 gpm. In combination with the application of effluent at an average of 15 gpm, a total of 40 gpm was maintained until the end of the test on December 10, 2007.

At the end of the test on December 10, 2007, the total volume of treated effluent discharged was 595,000 gallons and the total volume of potable water discharged was 1,2010,000 gallons for a combined total volume of treated effluent and potable water of 1,805,000 gallons. Effluent represented about one-third of the total applied volume on average. This test resulted in an average loading rate of approximately 29 gpd/sq. ft.

Legend

Hydrant

Monitoring Well

Parcels

Groundwater Contours (FT MSL)
(November 12, 2007)

0 50 100
Feet

N

SOURCE:
Well Cluster Sites and Test Pits created by Wright-Pierce
All other data obtained from Town of Orleans and MassGIS

Orleans CWMP
Site Investigations
Site 241
Groundwater Levels Pre-Test

PROJ NO. 10645E	WRIGHT-PIERCE Engineering a Better Environment	FIGURE: 6
DATE: Feb. 2008		
SCALE: AS SHOWN		

5.5.4 Recovery Period

From December 10 to December 20, 2007, water level measurements were obtained via pressure transducers every fifteen minutes for ten days in all monitoring wells following shutdown of the loading test.

5.6 Observation of Groundwater Response

The observed change at each of the monitoring wells is summarized in Table 3. Groundwater contours of the mounding are shown on Figure 7 from manual water level readings collected on December 10, 2007.

Of the eleven wells, water levels in four of the wells showed a rise ranging between 0.5 ft and 0.9 ft as of December 10, 2007. Water levels in the wells responded in the following categories:

Number of Wells	Change in Elevation
6	< 0.5 ft
4	0.5 to 1.0 ft
1	> 5 ft

The greatest change in groundwater level was observed in well GMW-3. This well is used for routine groundwater sampling and, according to Tri-Town staff, usually goes dry when pumped indicating that the well may be screened in tight soils. The subsurface geology may have directed the flow of the infiltrating discharge toward the area of GMW-3 and artificially raising the groundwater table due to these less permeable soils. Moreover, possible improper construction of the well may result in short circuiting the infiltrating discharge down the annulus of the well casing which would also artificially raise the water table in the well.

Ten days after the cessation of the test, the groundwater had recovered by approximately half the height of the change in elevation between pre-and post-test conditions.

Legend

Hydrant

Monitoring Well

Parcels

Groundwater Contour (FT MSL)
(December 10, 2007)

0 50 100
Feet

N

SOURCE:
Well Cluster Sites and Test Pits created by Wright-Pierce.
All other data obtained from Town of Orleans and MassGIS.

Orleans CWMP
Site Investigations
Site 241
Groundwater Levels Post-Test

PROJ NO 10645E			FIGURE:
DATE: Feb. 2008			7
SCALE: AS SHOWN			

6.0 PRELIMINARY LAYOUT OF DISPOSAL BEDS AND RECOMMENDED APPLICATION RATES

To establish a basis for modeling the groundwater to predict mounding, we made an estimate of the capacity of the site to accept effluent based on surface loading.

First we laid out 23 rapid infiltration beds, each 100 feet by 100 feet. These would be located on the easterly portion of the site to leave room for wastewater treatment facilities near the existing septage treatment plant. This bed layout would leave 100 feet of buffer along Route 6. See Figure 8.

Table 3
Observed Mounding at End of Hydraulic Loading Test

OSW Monitoring Well I.D.	Associated USGS Well Cluster Site	Screened Interval ft msl[1]	Observed Mound at End of Test-Manual ft	Observed Mound at End of Test-Electronic ft	Approximate Distance from RIB No. 2 ft
115	99	-12 to -13	0.12	0.10	100
108	94	6 to -1	0.61	0.53	25
137	86	10 to -0.5	0.08	0.15	200
143	148	1.5 to -3.5	0.03	0.05	275
150	150	3.5 to -1.5	0.57	0.93	7
178	93	3.5 to -1.5	0.90	NA	35
GMW-1	126	17 to -2.5	0.49	0.60	50
GMW-3	128	20 to 0.0	6.82	6.80	50
GMW-4	129	20 to 0.0	0.91	0.92	330
TOMW-2	132	11 to -9	-0.05	0.00	500
TOMW-3	133	10 to -10	-0.04	0.04	650
BMW-22	NA	1.5 to -1.5	NA	-0.20	6,700

1. Source: USGS Open File Report 95-439. 2. NA - Not applicable.

In estimating the disposal capacity of each bed, we considered the soil characteristics determined from this and prior explorations, the percolation testing conducted in 2007 and the large-scale hydraulic test. The basins fall in three categories:

- **9 beds (Numbered 4 through 12) including the area of the existing RIBs and the most favorable percolation testing.** For these beds, we have estimated a long-term capacity of 14 gpd/sf, about one-half the rate demonstrated in the loading test. This is a

Legend

- Hydrant
- Monitoring Wells
- Parcels
- Conceptual Basins

Test Pits 2007 Percolation Rate

- <2 min/inch
- >5 min/inch
- None

Test Pits by Others Percolation Rate

- <2 min/in
- 2-5 min/in
- >5 min/in
- * Estimated based on Soil Logs

0 50 100 Feet

SOURCE Well Cluster Sites and Test Pits created by Wright-Pierce. All other data obtained from Town of Orleans and MassGIS.

Orleans CWMP
Site Investigations
Site 241
Conceptual Diposal Layout

PROJ NO 10645E	WRIGHT-PIERCE Engineering a Better Environment	FIGURE: 8
DATE: Feb. 2008		
SCALE: AS SHOWN		

relatively high percentage of a loading test result, but justified by the scale and duration of the test. The selection of 14 gpd/sf considers the fact that the test results may have been biased downward by the 20-year history of effluent application in existing RIB 2.

- **3 beds (Number 1 through 3) located at the south end of the site**. The percolation testing here is the least favorable on site. We selected a rate of 4 gpd/sf based substantially on the rate allowed by DEP based only on percolation tests.

- **11 beds (Numbers 13 through 23) located in the central and northern portions of the site.** Here we selected a rate of 10 gpd/sf based on some favorable percolation testing, and prior soils characterizations. While we believe that large-scale hydraulic loading tests here might allow a higher rate, we suggest the 10 gpd/sf figure for planning purposes.

Table 4 lists the recommended loading rates for each basin. If all beds were in use, these estimates indicate a total capacity of 2.48 mgd. It would be prudent to base further planning on the fact that 5 beds would be kept idle to allow periodic cleaning and repair of other beds. If the idle beds were those with the highest predicted capacity, then the total design loading rate would be 1.78 mgd.

Table 4 also shows our estimate of the site capacity if the favorable loading test results were ignored. Using DEP-allowed rates based only on the percolation testing, the site would accommodate 1.10 mgd with all beds in service and 0.85 mgd with 5 beds idle.

On the basis of this analysis, we recommend that the Town proceed with site planning based on a short-term peak capacity of 1.78 mgd. Further site testing, including borings on the knoll, should be conducted to confirm and refine these estimates.

7.0 PRELIMINARY GROUNDWATER MODELING

Prior to the hydraulic loading test and the test pit excavations, a desktop analytical evaluation of groundwater mounding was conducted to see if mounding was likely to be a factor at this site. That analytical approach, using the Hantusch method and a peak flow of 1.6 mgd, resulted in an estimated mound height of approximately 10 feet. Given the fact that the ground surface elevation is higher than 40 feet above sea level across most of the upland portions of the site, and existing water table is typically at elevation 10 to 15, this would leave at least 40 feet of unsaturated soil above the mounded groundwater. Therefore, we proceeded with the test pits and loading test, knowing that mounding is not likely to limit the use of this site.

Table 4

Recommended Loading Rates

Basin Number	Basin Area	Loading Rates Based on Percolation Test Results		Loading Rates Based on All Test Results	
		Rate	Total Application	Rate	Total Application
	sf	gpd/sf	gpd	gpd/sf	gpd
1	10,000	4	40,000	4	40,000
2	10,000	4	40,000	4	40,000
3	10,000	4	40,000	4	40,000
4	10,000	5	50,000	14	140,000
5	10,000	5	50,000	14	140,000
6	10,000	5	50,000	14	140,000
7	10,000	5	50,000	14	140,000
8	10,000	5	50,000	14	140,000
9	10,000	5	50,000	14	140,000
10	10,000	5	50,000	14	140,000
11	10,000	5	50,000	14	140,000
12	10,000	5	50,000	14	140,000
13	10,000	4	40,000	10	100,000
14	10,000	5	50,000	10	100,000
15	10,000	5	50,000	10	100,000
16	10,000	4	40,000	10	100,000
17	10,000	5	50,000	10	100,000
18	10,000	5	50,000	10	100,000
19	10,000	5	50,000	10	100,000
20	10,000	5	50,000	10	100,000
21	10,000	5	50,000	10	100,000
22	10,000	5	50,000	10	100,000
23	10,000	5	50,000	10	100,000
Total, gpd		23 beds	1,100,000	23 beds	2,480,000
		18 beds	850,000	18 beds	1,780,000

After the successful hydraulic loading test, further groundwater modeling was conducted to refine the estimate of mounding at the site. The groundwater model was based on the USGS model of the Monomoy Lens of the Cape Cod Aquifer (Walter and Whealan, 2004). A sub-regional model, comprising the Town of Orleans and portions of Brewster and Eastham, was developed from the USGS regional model using a technique known as telescopic mesh refinement (TMR). The model grid was refined in the process. The original USGS model had uniform grid nodes with dimensions of 400 by 400 feet. The revised model has grid nodes ranging from 200 by 200 feet to 50 by 50 feet. This provides for significantly greater detail within the model. The newly-developed Orleans sub-regional model produces groundwater levels and flow directions that are virtually identical to those produced from the regional USGS model.

Following the refinement of the model, it was used to simulate the loading test conducted in November and December of 2007. The last 22 days of the loading test were simulated (the period after the unintended shutdown of the test). The loading test was simulated under steady state conditions. Modeling the discharge under transient conditions (that is, 22 days of discharge) resulted in nearly identical results. This means that the discharge had approached a steady state condition. The model predicted a groundwater mound that was generally higher than the observed mound – by as much as 1 to 2 feet in some areas. This was considered to be a significant deviation.

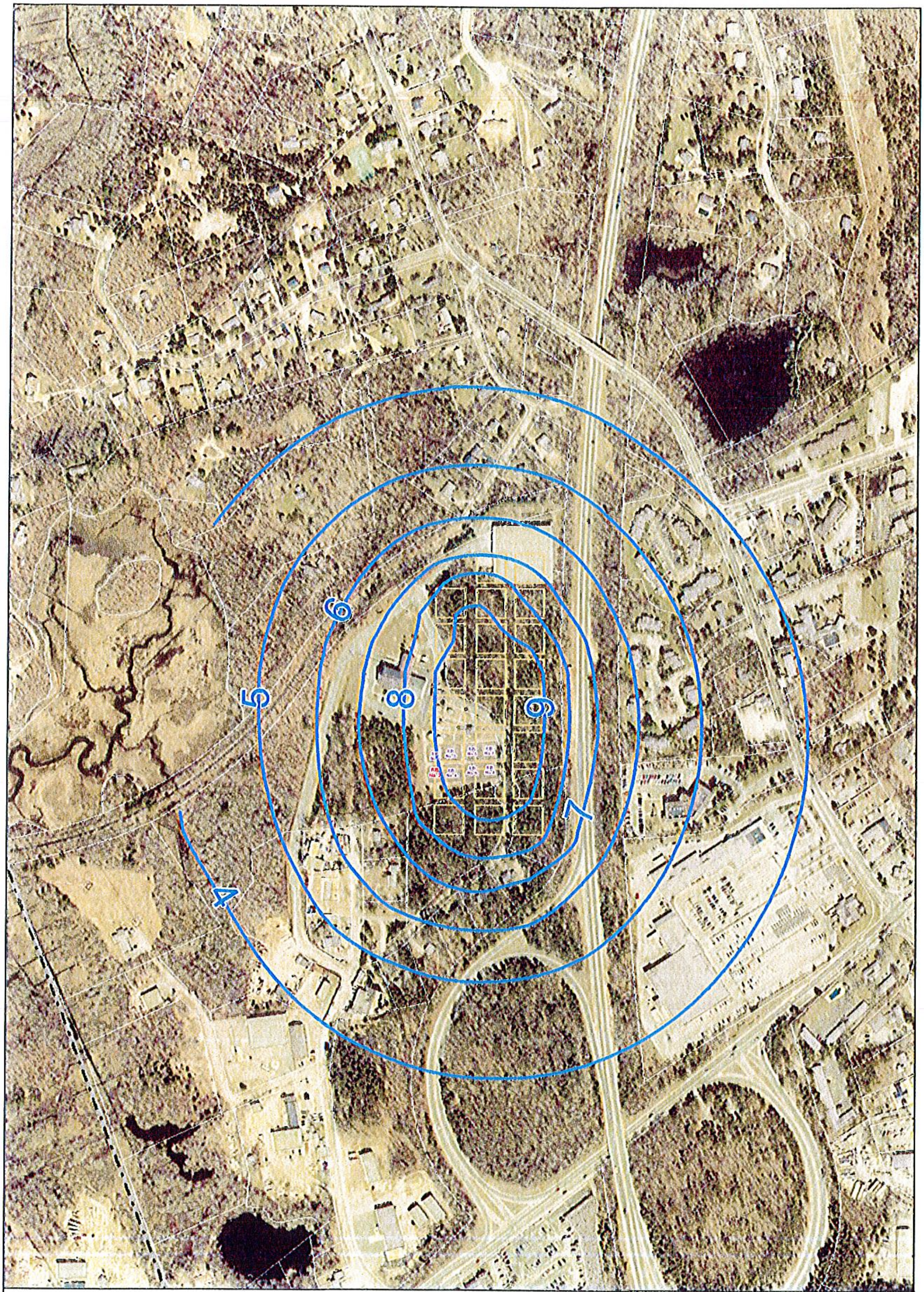
The hydraulic conductivity (K) field within the model was examined to determine the potential cause of the high mound. It was found that the K field in the model did not accurately reflect observed subsurface conditions, as represented by subsurface cross sections developed by the USGS in Desimone and Barlow (1996). The model K values represented a thin layer of relatively coarse material (medium to coarse sand) overlying a relatively fine grained material (fine sand). However, the USGS geologic cross sections through this area indicate that there is a relatively thin layer of fine-grained material overlying a coarser grained material – precisely the opposite of what was simulated in the model. The model K field was adjusted in the vicinity of the proposed discharge area to more closely reflect the actual subsurface data. The simulated groundwater mound resulting from this change was much closer to the mound observed during the loading test.

Once the model produced results that closely resembled the results of the loading test, the model was run to simulate the potential groundwater mound that would occur with the discharge of 1.6 mgd. The discharge was spread over an area of approximately 320,000 square feet in an area that has been determined to be suitable on the basis of test pits and percolation tests. (This is the gross area associated with 23 beds shown in Figure 8, whose net bottom area is 230,000 square feet.) The simulated groundwater mound is shown in Figure 9. Around the edges of the application area, the mound is expected to be about 8 feet high, rising to near 10 feet at the center of this area. Under these conditions there will be at least 20 feet from the top of the mound to the bottom of the discharge beds, far in excess of the four-foot minimum requirement.

The model indicates that there will be significantly more groundwater flow to the downgradient wetlands. However, there will be no groundwater discharges in areas that do not already receive groundwater discharges (i.e., wetlands), although in areas where groundwater currently discharges on steep slopes adjacent to wetlands, these may occur at slightly higher elevations.

It is important to note that these model simulations are quite conservative for two reasons. First, we assumed a conservative maximum monthly flow of 1.6 mgd, based on the Needs Assessment report, and not the 1.5 mgd maximum monthly flow that would be associated with the short-term peak flow of 1.78 mgd. Second, the modeling assumes that the maximum monthly flow would continue for three months. The maximum **monthly** flow associated with a 1.78 mgd short-term peak would be about 1.5 mgd, and the **90-day** maximum flow would be even less than that.

In later project phases, the model will undergo further revisions and calibration. Subsequent simulations will be based on the specific flow rates (annual average, monthly maximum and short-term maximum) associated with the wastewater management plan focused on the Tri-Town site (Plan 2). However, it is expected that the general conclusions of this preliminary modeling will hold – the groundwater mound resulting from a maximum-month discharge of up to 1.6 mgd is not expected to result in any excessive mounding.



0 150 300
Feet

N

Legend

- Simulated Groundwater Mound Contours (FT)
- Conceptual Basins
- Parcels

SOURCE:
Well Cluster Sites and Test Pits created by Wright-Pierce.
All other data obtained from Town of Orleans and MassGIS.

Orleans CWMP
Site Investigations
Site 241
Simulated Mounding - 1.6 mgd

PROJ NO	10645E	WRIGHT-PIERCE	FIGURE: 9
DATE:	Feb 2008		
SCALE:	AS SHOWN		

8.0 Conclusions and Recommendations

An extensive evaluation of Site 241, the site of the Tri-Town Septage Treatment Facility, was conducted to assess its capability to accept wastewater effluent through rapid infiltration. These investigations included:

- Review of historic test pit and boring logs from the original plant construction and from the USGS studies on site.
- Excavation of 13 new test pits and performance of 8 percolation tests.
- A 25-day large-scale hydraulic loading test using one of the exiting Tri-Town rapid infiltration basins and application of 1.8 million gallons of potable water and septage plant effluent.
- Refinement of the USGS regional groundwater model and simulation of expected mounding from effluent application.

The hydraulic loading test demonstrated that one of the existing rapid infiltration basins could be loaded at a long-term rate of 29 gpd/sf, with only a moderate rise in the underlying groundwater. This is an encouraging result because of the long duration of the loading test, the presence of suspended solids in the septage effluent, and the near surface soil plugging from 20 years of effluent application.

Test pits and percolation tests demonstrated the generally permeable nature of the site soils, tempered by occasional lenses of less permeable materials. By considering the distribution of soils and the location of the loading test, the site capacity was estimated for a proposed configuration of 23 future rapid infiltration basins:

- In the central, most favorable, area that was extensively tested: 9 beds loaded at 14 gpd/sf (about one-half of the rate demonstrated during the loading test);
- In the high northerly area of the site where permeable soils are expected but deep borings are needed for confirmation: 11 beds at 10 gpd/sf (about one-third the rate demonstrated during the loading test); and
- In the southerly portion of the site that had moderate percolation rates: 3 beds at 4 gpd/sf (the rate DEP would allow based only on percolation testing).

In the aggregate, this bed configuration and estimated loading rates would allow a short-term peak flow of 1.78 mgd, assuming 18 beds on-line and 5 beds in reserve.

Numerical modeling results indicate that the anticipated groundwater mound under the proposed discharge area would be well below the bottom of the proposed rapid infiltration basins. The highest predicted height of the mound at a conservative 1.6 mgd maximum-month loading rate is approximately 10 feet. The separation between the bottom of the rapid infiltration basins and the groundwater mound will likely be maintained at or greater than 20 feet.

The model predicts that "break outs" will not occur. Increased flooding or ponding of the existing wetlands may occur at slightly higher elevations. Effects from the loading test on the water table in the area of Namskaket Marsh were not observed.

Although extensive geological data exists for the site, test borings should be advanced to the elevation of the proposed RIBs to classify and test the permeability of the soil, on the northern portion of the site where soil boring data are lacking. This work is expected to confirm the loading rates estimated for that portion of the site.

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