City of Grand Island

Tuesday, May 15, 2012

Study Session

Item -1

Grand Island Dewatering Study 2012 Update

Staff Contact: John Collins, Public Works Director

Council Agenda Memo

From: Terry Brown, P.E., Manager of Engineering Services

Meeting: May 15, 2012

Subject: Grand Island Dewatering Study 2012 Update

Item #'s: 1

Presenter(s): John Collins, P.E., Public Works Director

Background

On December 21, 1998 the City entered into an agreement with the Central Platte Natural Resources District (CPNRD), which provided for the installation of test and monitoring wells to study lowering groundwater levels. The study concluded September 2000 with a recommendation to implement a dewatering program.

On June 28, 2011, by Resolution 2011-162, the City Council was presented with information on the costs and construction on a dewatering system for the City of Grand Island.

On July 26, 2011 City Council approved an Interlocal Agreement with the Central Platte Natural Resources District (CPNRD) to update the September 2000 Groundwater Study. The study and cost are being shared equally between the City and the CPNRD.

Discussion

Results of the updated study are being presented in draft form to both the City and the CPNRD. A final study will be presented to City Council upon collection of input from both the City and CPNRD.

Conclusion

This item is presented to the City Council in a Study Session to allow for any questions to be answered and to create a greater understanding of the issue at hand.

It is the intent of City Administration to bring this issue to a future council meeting for direction on how to proceed with a dewatering project for the City of Grand Island.





Grand Island Dewatering Study 2012 Update

OLSSON ASSOCIATES

Grand Island City Council Study Session
May 15, 2012

Grand Island Study Session Outline

- Project Background
- Dewatering Areas of Concern
- Project Scope
- Previous Dewatering Options
- 2012 Groundwater Model
  - Comparison to 2000 Model
  - Model Development
  - Model Results
- Conveyance and Discharge Options
- Preliminary Opinion of Probable Cost
- Financing Options
- Implementation Recommendations

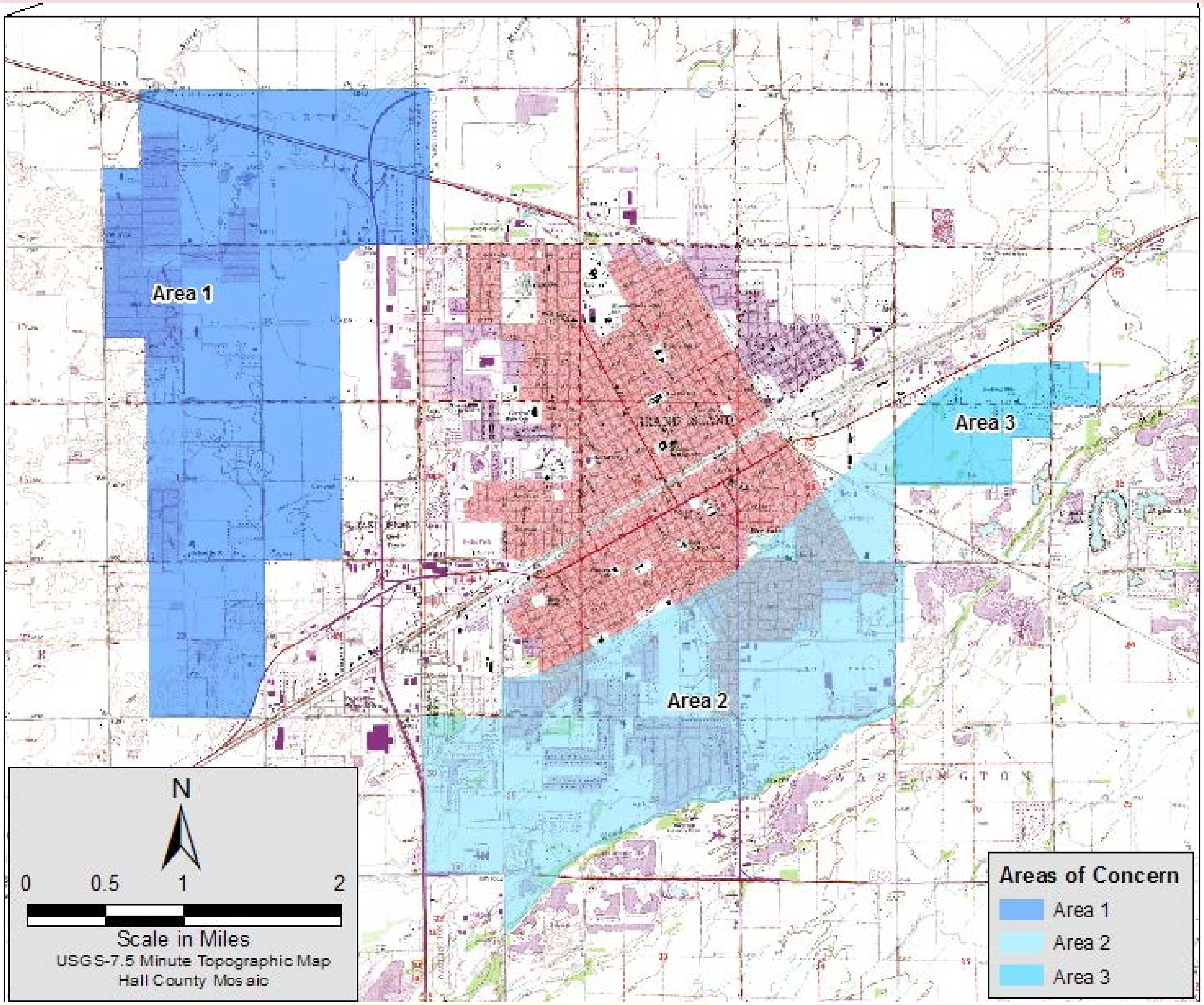
Project Background

- 1998 Record High Groundwater Levels
- Olsson Associates Completed Dewatering System Study to lower Groundwater Levels
- Engineers Opinion of Costs $9,943,000
- Annual Project Costs w/Operations $1,574,100 (20 yr., int.7%)

- 2008 High Groundwater Levels
- Indexed Dewatering System Costs to 2007 Dollars
- Engineers Opinion of Costs $17,802,000
- Annual Project Costs w/Operations $1,895,602 (20yr., Int. 5%)

- 2012 Groundwater Model update – Dewatering System Layout and Opinion of Costs Update

Dewatering Areas of Concern



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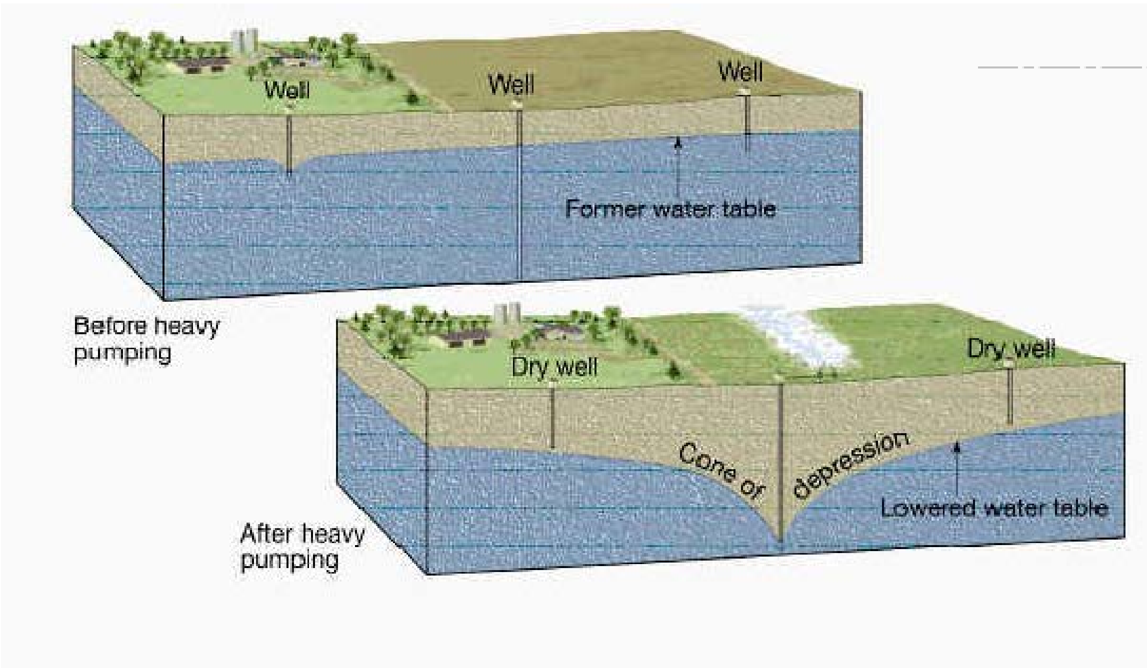
2012 Update – Project Scope

- Evaluate Previous Options and Current GW Conditions
- Assess Current Groundwater Contamination Plumes
- Develop updated groundwater model
  - -to evaluate dewatering well layouts
- Prepare conceptual layout of conveyance piping
- Develop preliminary opinion of probable cost
- Identify project financing options

Dewatering Well Influence



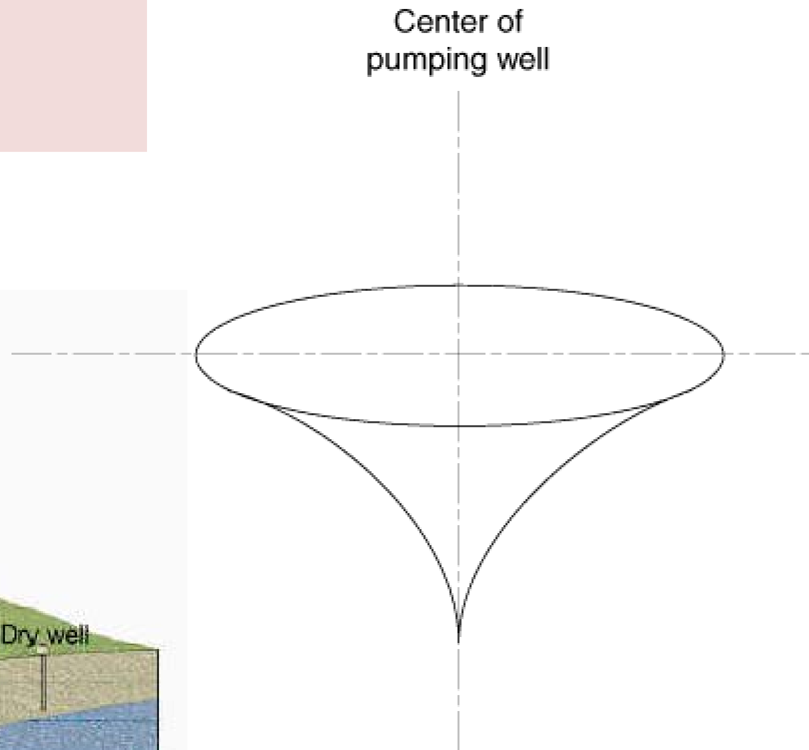
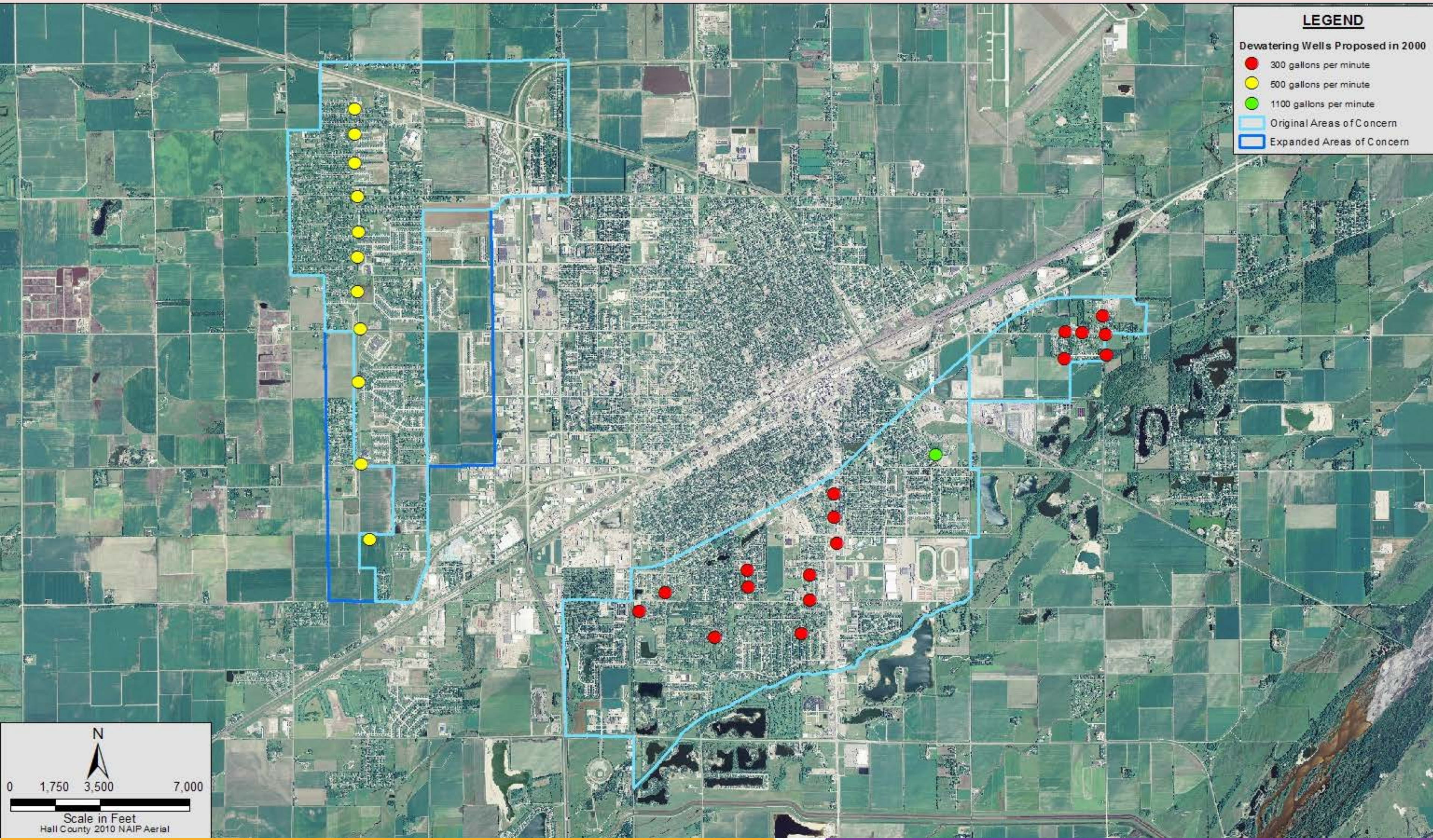


Image sources: USGS.gov and fayettecountygroundwater.com,

2000 Dewatering Option



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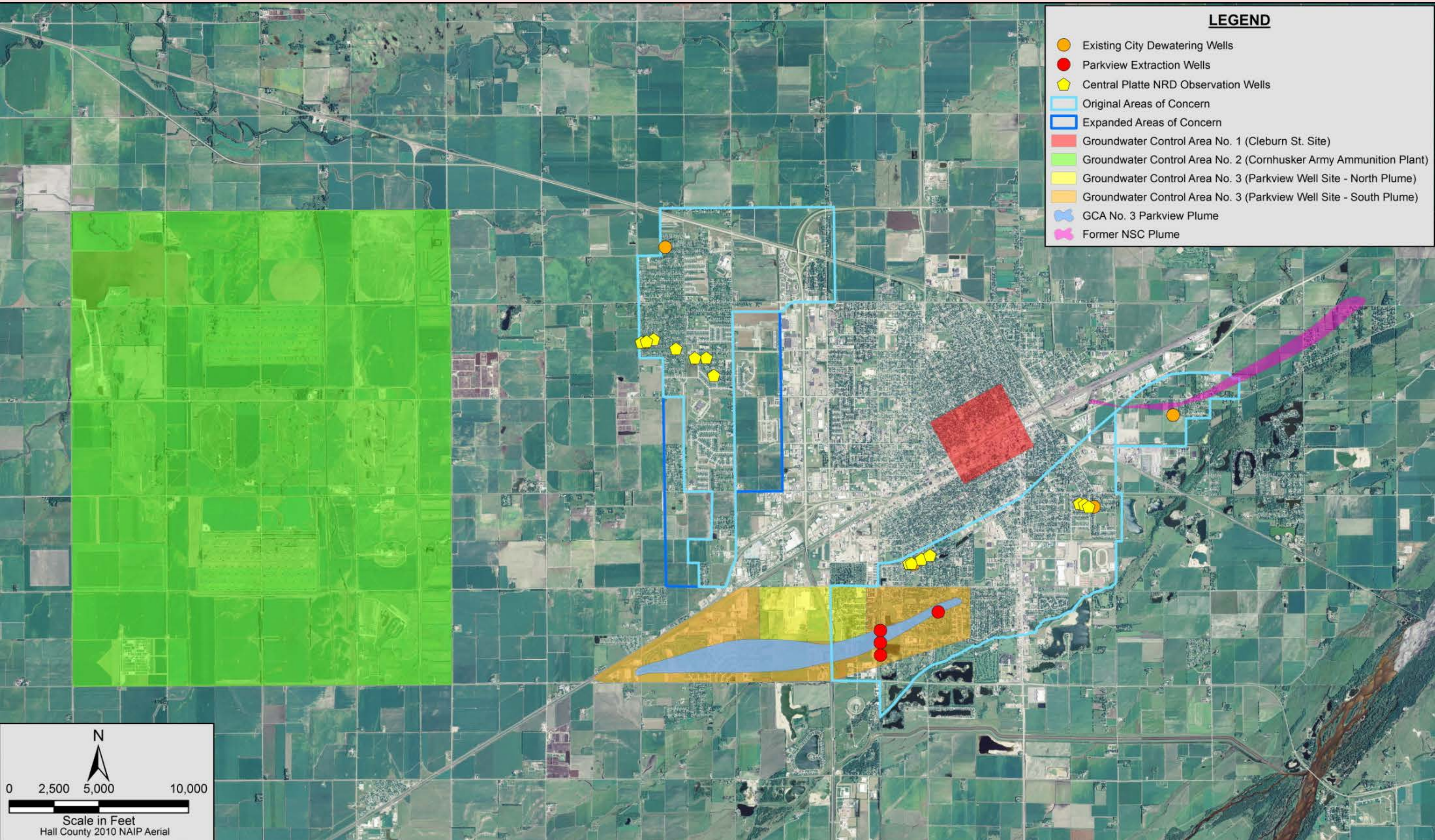
Groundwater Contamination Plumes



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Comparison to 2000 Model

- Higher resolution topographic survey data
- Higher resolution aquifer data including

  Hydraulic conductivity

  Specific yield
- Refined Recharge areas with four areas defined

  Irrigated land, riparian, urban open and urban developed
- Detailed Platte River and Wood River flow data input
- Model includes all industrial and irrigation wells
- Model period simulates 1999-2011

Model Development

Steady-State Flow Model

METHOD: MF2K/GWV, PEST

GOAL: Estimate aquifer permeability, establish initial water table conditions representing the late 1990s, set initial conditions for transient period.

Transient Flow Model

METHOD: MF2K/GWV, forward calibration

GOAL: Development of a model that simulates changing water levels witnessed from 1999-2011, and is capable of accurately determining water level changes resulting from new dewatering wells.

Design & Test Wellfield Configurations

METHOD: MF2K/GWV, calibrated transient model testing well configurations vs. a 15 ft. bgl elevation.

GOAL: Use of transient model to determine number of wells, appropriate pumping rates and times to reduce water table below a critical depth in the areas of concern underlying Grand Island. Assess timing of water table recovery with wells idle.

Capture Zone Analysis

METHOD: MF2K/GWV, MODPATH

GOAL: Determine spatial area of influence of each new dewatering well over 12 year period, ensure no interference with contaminant plumes by capture zones in south area of concern.

MT3DS Test

METHOD: MF2K/GWV, MT3DS

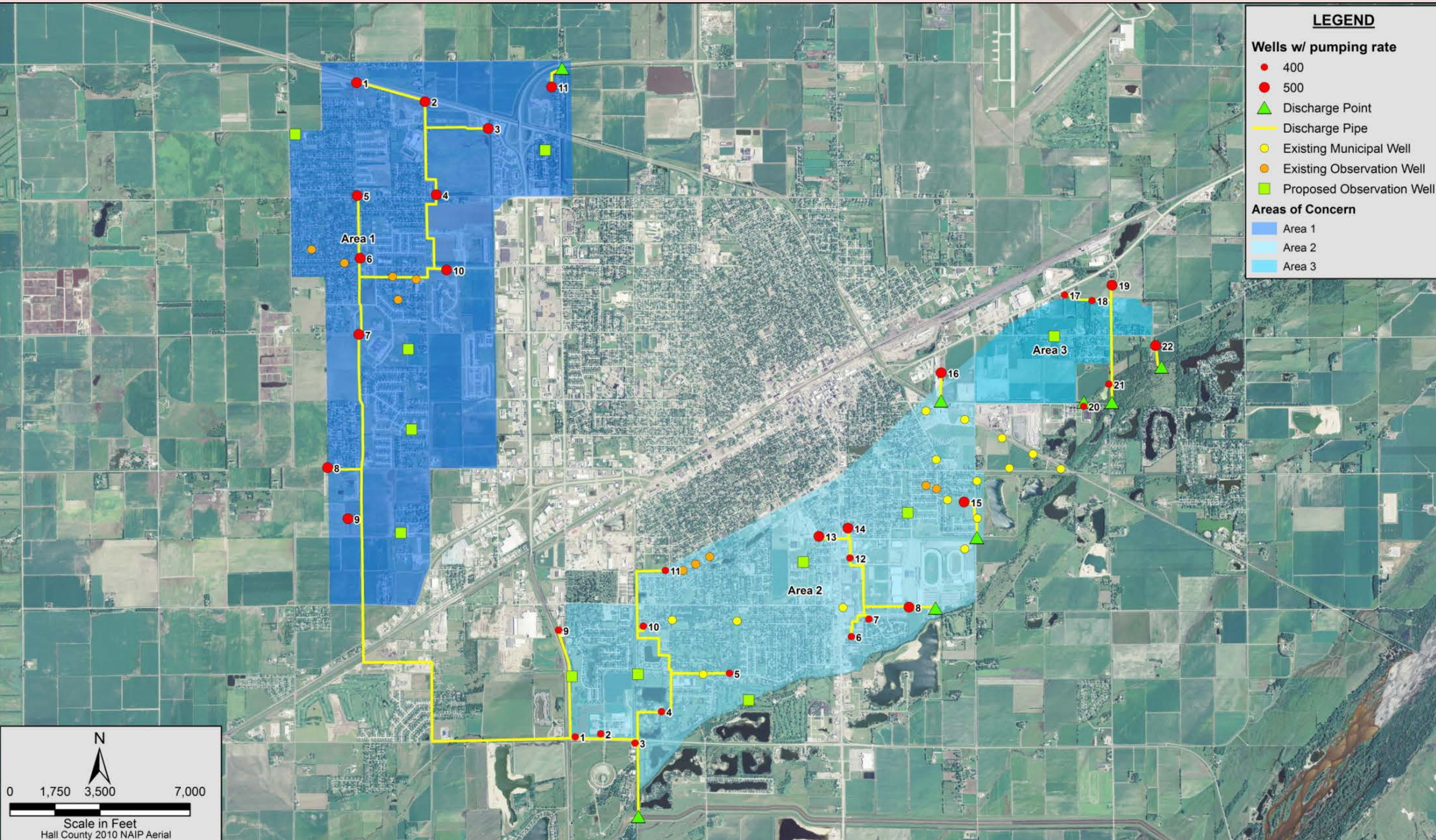
GOAL: Test influence of new wells in south area of concern on hypothetical contaminate plumes in areas where mapped plumes exist under Grand Island.

Final Product

Configuration of dewatering wells in the north and south areas of concern, showing necessary rates and pumping durations to reduce water table elevations below a 15-ft bgl "critical surface", with reasonable confidence that minimal influence on contaminant plumes will occur in south area of concern.

Groundwater Model Animations

2012 Dewatering System Layout



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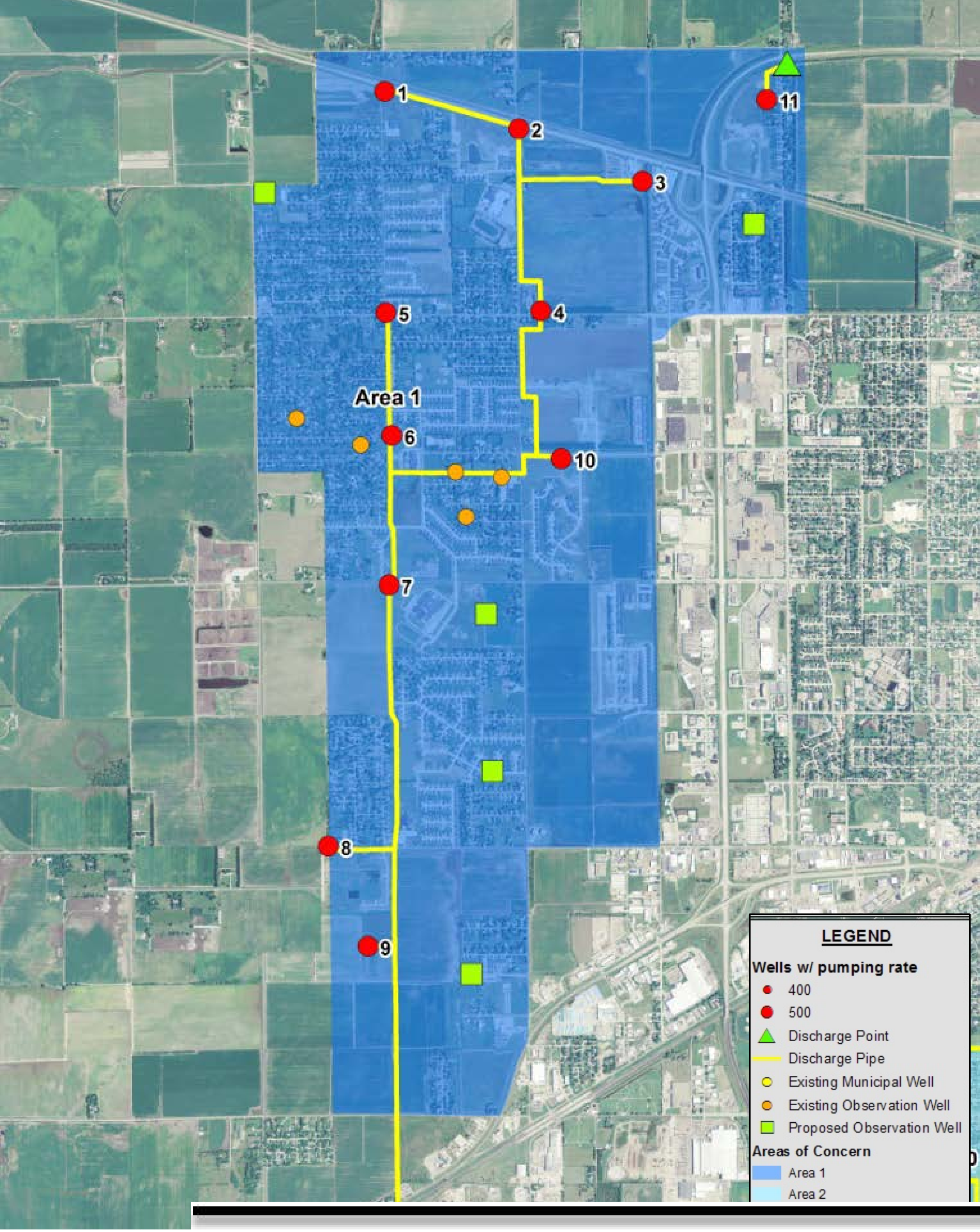

Area 1

- **Eleven dewatering wells**
- **Two discharge points**
- **Five new and Five existing monitoring wells**

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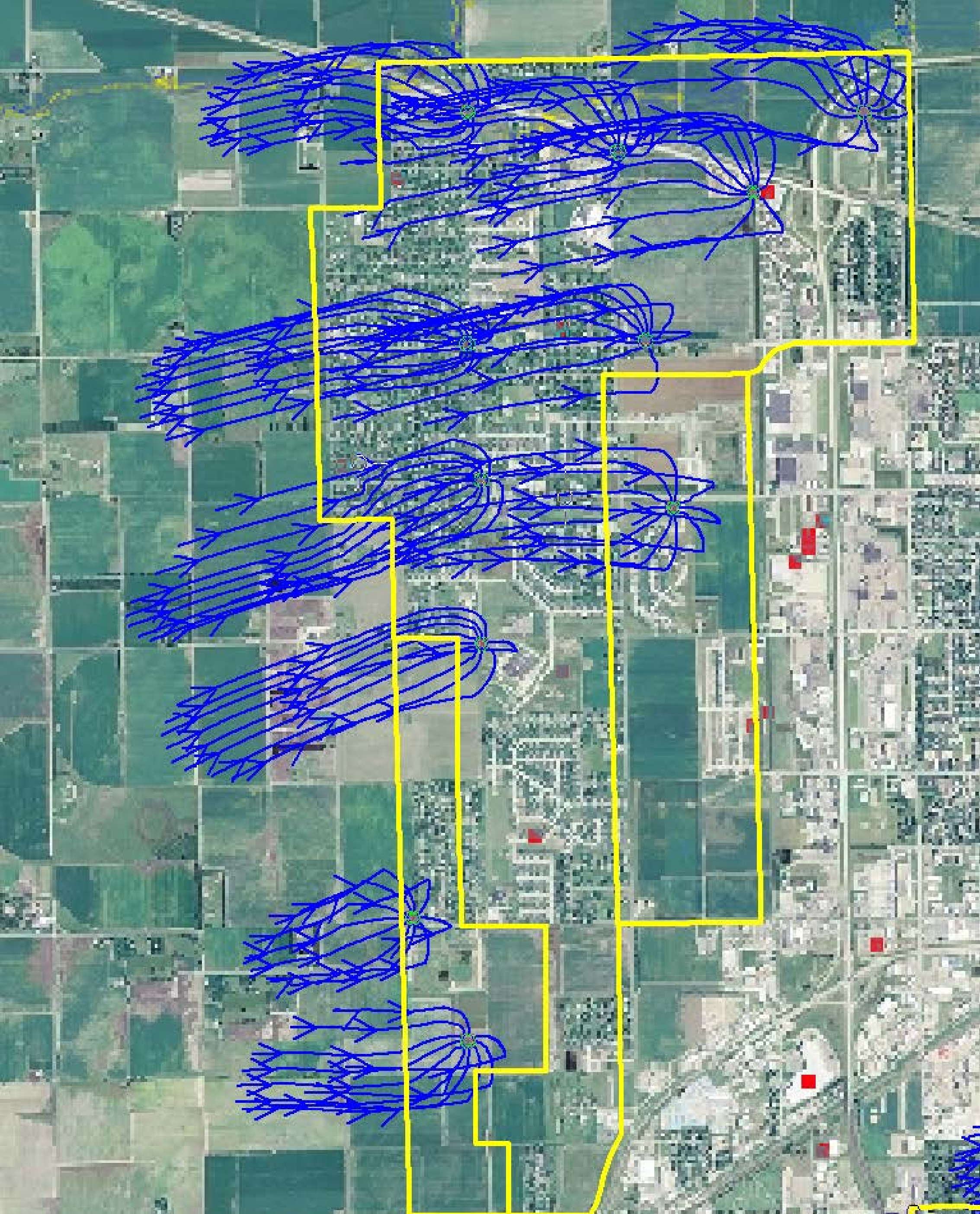
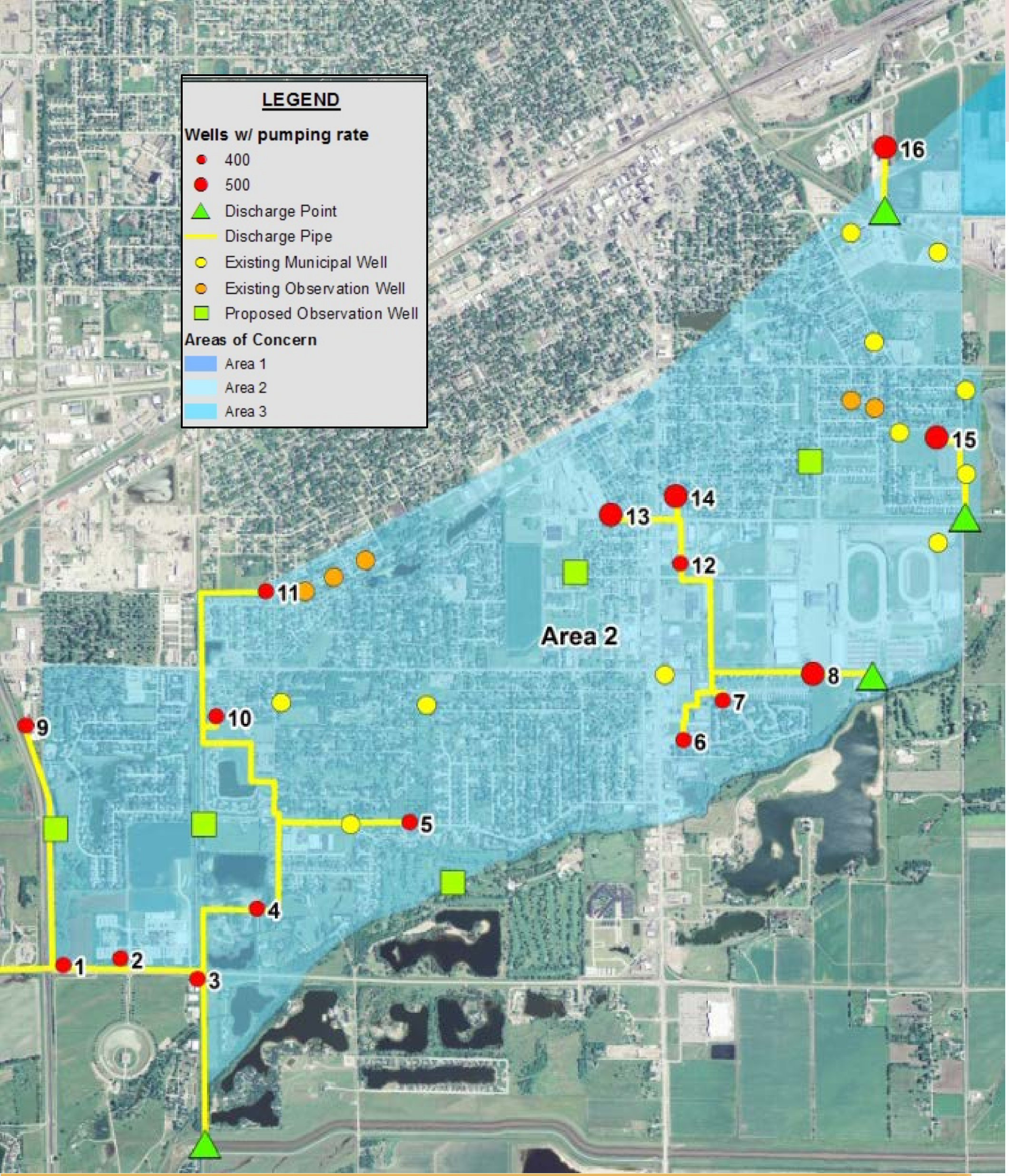


Area 1

- **Simulated 12 year well capture zones**

Area 2



- **Sixteen dewatering wells**
- **Four discharge points**
- **Five new and Five existing monitoring wells**

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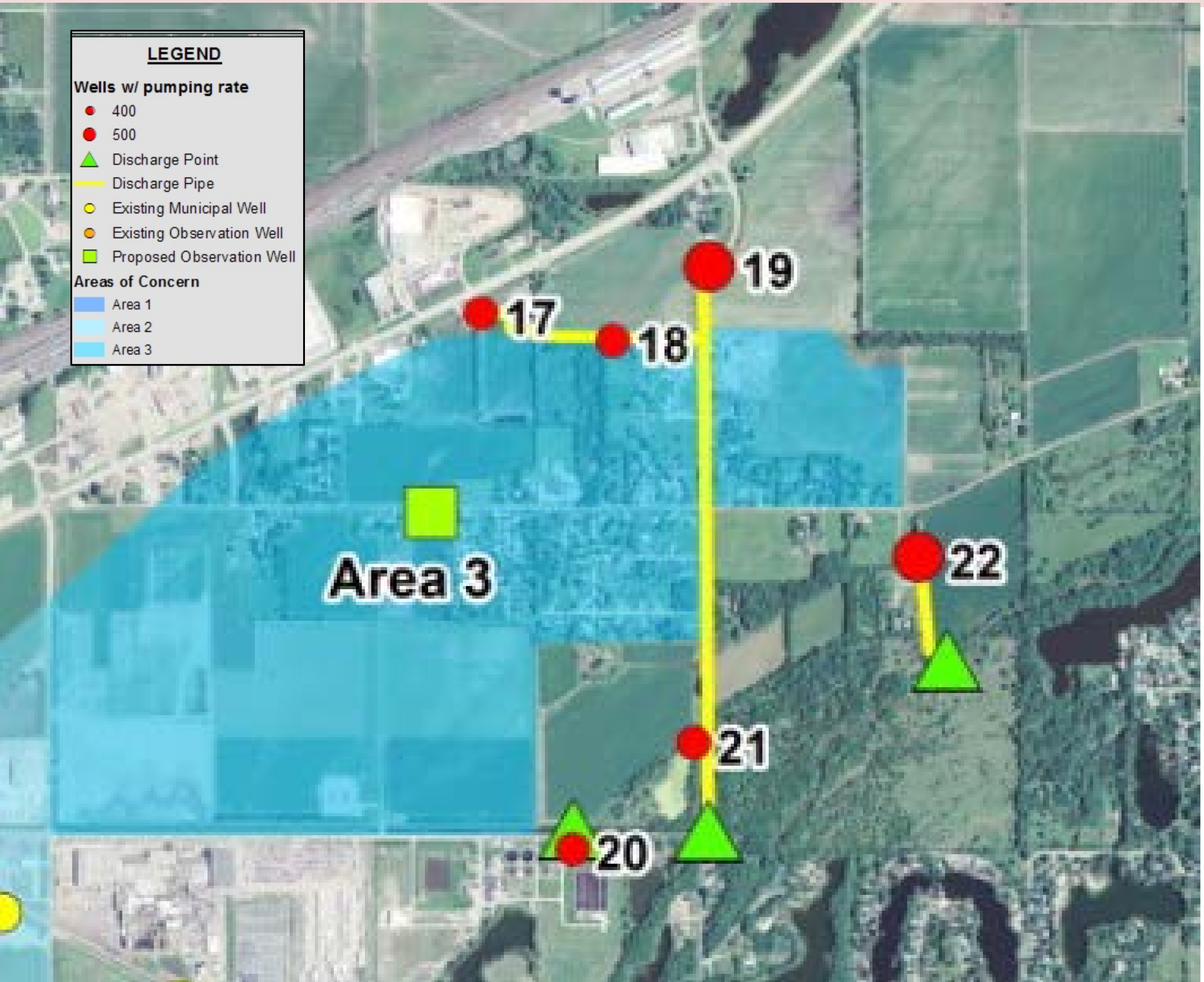
Area 3



- Six dewatering wells
- Three discharge points
- One new monitoring well

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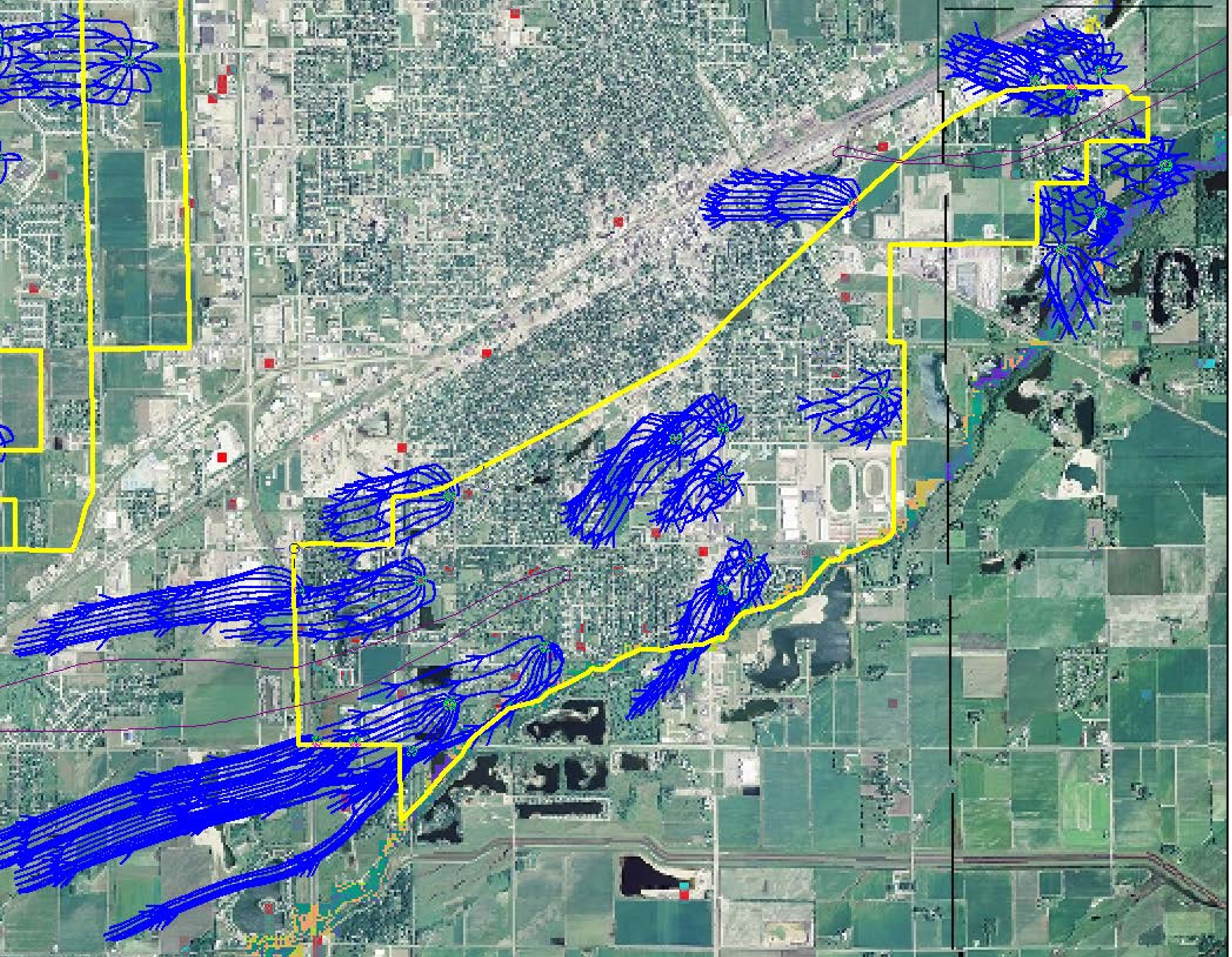

Area 3

- **Simulated 12 year well capture zones**

Preliminary Opinion of Probable Cost

	Area 1	Area 2	Area 3	Combined Total
Transmission Line	$7,085,000	$1,617,000	$477,000	$9,179,000
Utility Conflicts	$31,000	$28,000	$4,000	$63,000
Dewatering Wells	$939,000	$1,347,000	$498,000	$2,784,000
Control System	$118,000	$135,000	$62,000	$315,000
Construction Cost Subtotal	$8,173,000	$3,127,000	$1,041,000	$12,341,000
Contingency	$818,000	$313,000	$105,000	$1,236,000
Overhead, Legal, Fiscal, Engr.	$981,000	$376,000	$125,000	$1,482,000
ROW Acquisition	$1,877,000	$1,496,000	$496,000	$3,869,000
Total Project Costs	$11,849,000	$5,312,000	$1,767,000	$18,928,000
Annual Costs (20 years, 7%, A/P)	$1,118,462	$501,415	$166,792	$1,786,669
Annual Costs (20 years, 3%, A/P)	$796,439	$357,050	$118,770	$1,272,259

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Annual Operational Cost

Item	Area 1	Area 2	Area 3	Combined Total
Labor	$18,000	$18,000	$18,000	$54,000
Power	$120,000	$94,000	$30,000	$244,000
Miscellaneous Repairs & Supplies	$47,700	$68,900	$26,500	$143,100
Total O & M (Per Year)	$185,700	$180,900	$74,500	$441,100
TOTAL Annual Cost	$982,139	$537,950	$193,270	$1,713,359

Monthly Operational Cost

Item	Area 1	Area 2	Area 3	Combined Total
Labor	$1,500	$1,500	$1,500	$4,500
Power	$10,000	$7,833	$2,500	$20,333
Miscellaneous Repairs & Supplies	$3,975	$5,742	$2,208	$11,925
Total O & M (Per Month)	$15,475	$15,075	$6,208	$36,758
TOTAL Monthly Cost	$81,845	$44,829	$16,106	$142,780

Financing Options

- Dewatering Districts
- User Fees
- Revenue and Various Purpose Bonds
- Water Banking

Implementation Recommendations

- Conduct neighborhood meetings for education and public outreach
- Work with CPNRD for funding options
- Develop Dewatering Districts
- Determine financing schedule
- Complete final design including plans and specifications
- Solicit bids for construction
- Conduct final public hearings on assessment fees Dewatering Districts
- Finalize project financing

Questions

2012 UPDATE

DRAFT
GRAND ISLAND DEWATERING SYSTEM STUDY

GRAND ISLAND, NEBRASKA

PREPARED FOR

CITY OF GRAND ISLAND, NEBRASKA
AND
CENTRAL PLATTE NATURAL RESOURCES DISTRICT

MAY 7, 2012

OLSSON PROJECT NO. 011-2231

OLSSON ASSOCIATES

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EXECUTIVE SUMMARY

In 2000, as part of a comprehensive study to evaluate options for dewatering high water table conditions in Grand Island, Olsson Associates constructed a numerical groundwater model of the aquifer underlying Grand Island to use as a tool to design a dewatering system. The 2000 study used state-of-the-art groundwater modeling and analytical tools, however, in the past twelve years, computer modeling capabilities and computer computation speeds have dramatically increased. Additionally, the issues of high water table conditions in certain areas of Grand Island have remained and even expanded in some parts of town. For these reasons, the City of Grand Island and Central Platte NRD authorized an update to the 2000 study using the latest groundwater modeling tools.

The primary areas within the Grand Island city limits that have recurring high water table conditions are in the west, northwest and southern part of town. The areas have been subdivided into three areas where Area 1 encompasses the northwest part of the City and is located primarily between Highway 2 and Highway 30 and west of Highway 281. Area 2 encompasses the southern part of the City located south of Oklahoma Avenue to the Wood River, west from Highway 281 to the east along South Stuhr Road. Area 3 is located east of South Stuhr Road to the eastern edge of the City just east of North Shady Bend Road.

An updated groundwater model was developed, calibrated and executed to evaluate the optimal locations for a system of dewatering wells in Areas 1, 2 and 3. The Updated Dewatering System includes a total of 33 dewatering wells (11 in Area 1, 16 in Area 2 and Area 3 has 6). The system is designed to reach the optimal groundwater target depth of 10-15 feet below ground surface through the network of dewatering wells without significant impacts to the existing groundwater contamination plumes that are known to exist within the City limits.

The major findings of this study are as follows:

1. The City of Grand Island and the CPNRD should continue to develop this project. The plan proposed in this study consists of a series of vertical wells placed throughout the project area. The wells will be connected together with a system of pipes to transfer the water to the Platte River. The City will be able to monitor and control the system with a centrally located control system.
2. The groundwater levels continue to plague local residents, not knowing the next time the groundwater will enter their basements. The property values in the affected areas would return to the current market values of the City when the projects are completed.
3. The findings of the updated study are similar to that found in the 2000 report, but the modeling is more refined to better understand the groundwater movement. Construction costs have increased but the interest rates have decreased, resulting in only minor annual cost increases from the 2000 study. The project remains affordable to the City of Grand Island.

Implementation of a project of this magnitude is impossible without the support of the public. The following recommendations will provide guidance in the implementation of a comprehensive dewatering system for the City of Grand Island.

- Conduct neighborhood meetings within the three project areas. Determine the local support and discuss with the residents the Dewatering District concept, and how it would be used to fund the proposed capital improvements and OM&R.
- The City should work closely with the Central Platte Natural Resources District (CPNRD) to review the possible local, state and federal funding sources. Determine if any outside funds could be made available to help finance the projects. Use the CPNRD resources to create a water banking program for future income options.
- The City should proceed to create the Dewatering Districts. Three Districts should be created.
- Determine the financing and revenue sources and schedule, to meet the necessary obligations. Obtain temporary financing to begin the development of the Projects.
- Authorize Olsson Associates to complete project plans and specifications and obtain all necessary permits and right-of-way and /or easements in cooperation with the City, as necessary, for the project construction, operations and maintenance.
- Solicit construction bids for the project from local contractors.
- Construct projects per plans and specifications
- Determine Project Costs and Assessments. Conduct hearing for to make the final assessments to each resident within the benefited area.
- Finalize financing based on the amount of bonds required to pay off the temporary financing and pay for the project over a 20 year period.

1.0 INTRODUCTION

This Dewatering System Study Update was prepared to identify the best option available to reduce high water table conditions in three specific areas within the City of Grand Island, Nebraska. This document was prepared by Olsson Associates under contract with the City of Grand Island and Central Platte Natural Resources District (Central Platte NRD). The document was written for the sole use of the City and Central Platte NRD.

1.1 BACKGROUND

In 2000, as part of a comprehensive study to evaluate options for dewatering high water table conditions in Grand Island, Olsson Associates constructed a numerical groundwater model of the aquifer underlying Grand Island to use as a tool to design a dewatering system. The 2000 study used state-of-the-art groundwater modeling and analytical tools, however, in the past twelve years, computer modeling capabilities and computer computation speeds have dramatically increased. Additionally, the issues of high water table conditions in certain areas of Grand Island have remained and even expanded in some parts of town. For these reasons, the City of Grand Island and Central Platte NRD authorized an update to the 2000 study using the latest groundwater modeling tools. Using the results of this updated study, the City will be able to make an informed decision on the best options available to reduce the high water table conditions. This study also includes updated project cost estimates and construction phasing options that will help in the development of dewatering districts to provide funding for the project.

1.2 STUDY AREA AND AREAS OF CONCERN

The City of Grand Island is located in an area where the depth to groundwater varies from less than 10 to 30 feet below the ground surface. The City lies within the relatively flat sandy river deposits of the Platte River Valley. There is very little topographic relief across the City with one prominent exception. There is a broad topographic plateau that elevates the center of the City. The plateau lies roughly parallel to the railroad tracks that cut through the center of the City from the southwest to the northeast. This broad topographic feature elevates the center of town approximately 12 feet above the rest of the City. Because of the relief that this plateau provides, the issues of high water table conditions in the central portion of the City are not as significant. Conversely, as the City has grown westward and to the south, there are areas of the City that now directly overlie the sandy alluvial river deposits with recurring high water table conditions.

The three primary areas within the Grand Island city limits that have recurring high water table conditions are illustrated in Figure 1. Area 1 encompasses the northwest part of the City and is located primarily between Highway 2 and Highway 30 and west of Highway 281. Area 2 encompasses the southern part of the City located south of Oklahoma Avenue to the Wood

River, west from Highway 281 to the east along South Stuhr Road. Area 3 is located east of South Stuhr Road to the eastern edge of the City just east of North Shady Bend Road.

In these three area, the citizens of Grand Island have had repeated incidents of groundwater inundation into their basements. This causes significant property damage, health issues related to mold and mildew, and for many home owners, the annual problem directly affects property values since it is difficult to sell homes with recurring water problems.

1.3 DOCUMENT PURPOSE, SCOPE, AND ORGANIZATION

The purpose of this project is to provide the City with the most cost effective and efficient option to reduce the recurring high water table conditions in Areas 1, 2, and 3. As stated above, this is an update to the study completed by Olsson Associates in 2000. The scope of the current project included:

- Evaluating the current groundwater hydrogeologic conditions in Grand Island (Section 2)
- Reviewing the previously proposed dewatering options (Section 3)
- Preparing a transient numerical groundwater model that simulates the aquifer conditions in and around Grand Island (Summary in Section 4 with the full analysis presented in Appendix A)
- Preparing an optimized dewatering system layout (Section 4)
- Describing system conveyance and discharge options and a preliminary opinion of probable cost (Section 5)
- Describing the City's financing options (Section 6)
- Recommendations for project implementation (Sections 7)

2.0 GEOLOGY, HYDROGEOLOGY AND GROUNDWATER PLUMES

2.1 GEOLOGY

Grand Island is underlain by sands and gravels deposited by the braided Platte River as it shifted across what is now Hall County. These sands and gravels were deposited on top of the Seward Formation which in contrast is composed of silts and clays. In the Grand Island area, the Platte River sand and gravel deposits are from 70-175 feet thick with some isolated interbedded silts and clays within the thick sequence of sand and gravel.

2.2 HYDROGEOLOGY

In Grand Island, as in much of central Nebraska, the movement of groundwater occurs through the pore spaces between uncemented grains of clay, silt, sand, and gravel. The depth to the water table ranges from less than 10 feet to greater than 30 feet below ground surface depending on the location within the study area and the prevailing climate conditions. The sand and gravel aquifer beneath Grand Island ranges in saturated thickness from 50 to 150 feet. The coarse sand and gravel deposits provide for excellent water bearing capacity and as such, Grand Island has some highly productive wells that can generate over 2,000 gallons per minute.

Because of the relatively shallow water table conditions combined with the coarse sand deposits, groundwater levels rise and fall in response to precipitation events. Figure 2 illustrates the water table changes over time in comparison with annual precipitation. Differences between the two plots are mostly due to a slightly delayed response time in the aquifer conditions and as well, the affects of pumping from nearby irrigation wells. What this means for the citizens of Grand Island living in Areas 1, 2 and 3 is that after significant rain events, there is an almost immediate rise the water table such that basements are inundated until the high water table conditions subside.

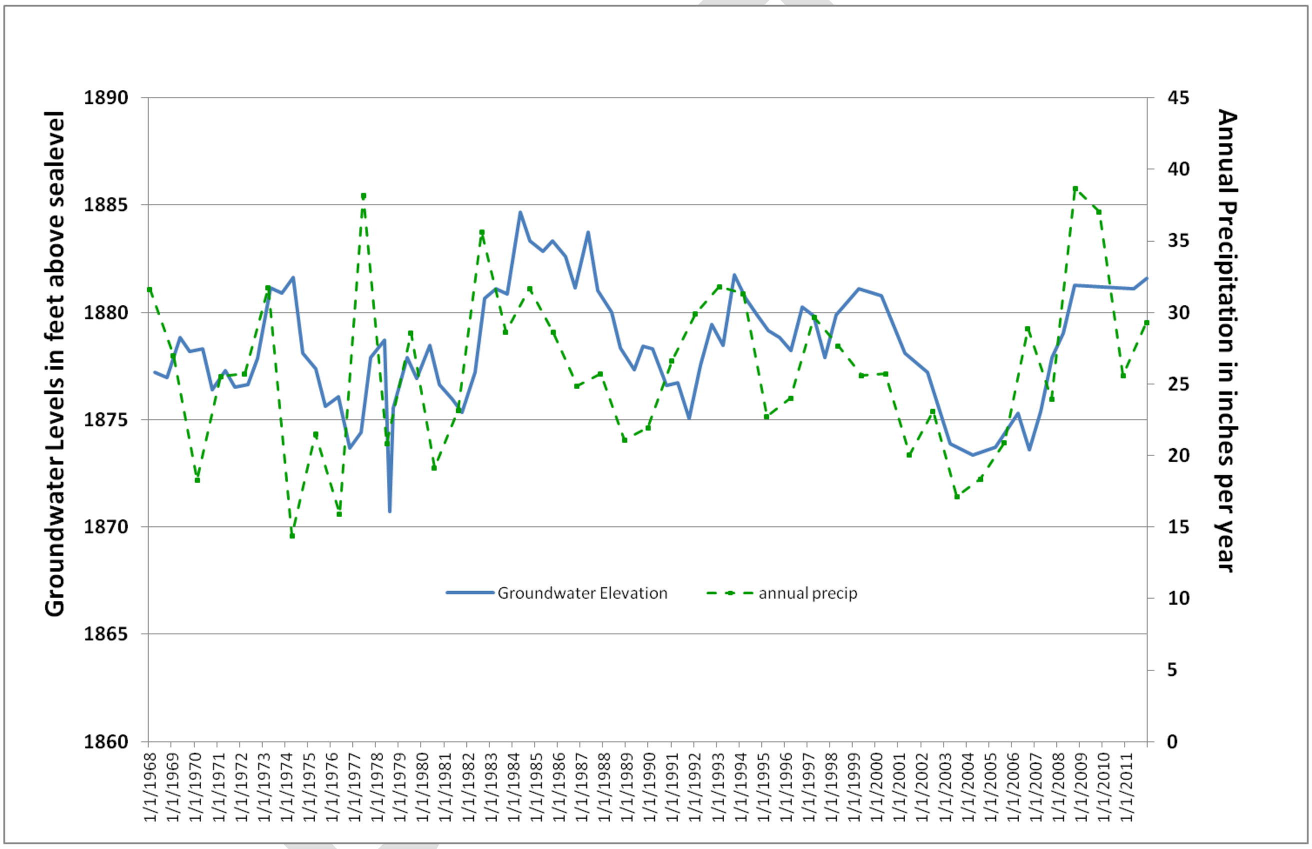


Figure 2 Groundwater levels versus annual precipitation at USGS monitoring well 405318098252202

2.3 GROUNDWATER PLUMES

The high water table conditions in Grand Island also cause other problems across the city. There are numerous plumes of contaminants that occur in the aquifer due to the fact that any spills or leaks of hazardous liquids at the surface migrate directly to the aquifer with little to no natural attenuation of the contaminants. As shown in Figure 3, there are three Groundwater

Control Areas (GCA) and two primary groundwater plumes that have been mapped beneath Grand Island. In the center of the City is groundwater control area No. 1. The control areas were designated where there has been significant deterioration of groundwater quality and to ensure the protection of human health and the environment, regulations were put in place to restrict the installation of wells and use of the groundwater. All three of the GCAs are undergoing groundwater remediation and cleanup. GCA No. 1, Cleburn Street Site and GCA No. 2, Cornhusker Army Ammunition Plant, are not within the Areas of Concern specified for this project. Furthermore, the plume that had previously emanated from the Cornhusker Army Ammunition Plant has been remediated to the required groundwater cleanup standard. Therefore, these two areas of groundwater contamination will not be discussed in further detail in this report.

GCA No. 3, Parkview Well Site, north and south plumes are on the western edge of Area 2 and the Former NSC plume cuts across Area 3. These plumes have significant impact on the placement of dewatering wells since any dewatering within these areas must be accomplished without impacting the direction or flow regime of the groundwater plumes. The City of Grand Island must avoid withdrawals of water from within these plumes, otherwise, the City will have to treat the water prior to discharge.

Both the GCA No. 3 Parkview and the Former NSC plumes contain levels of chlorinated solvents that are above the US EPA Drinking Water Standards. These types of contaminants are typically more dense that water and therefore sink to the bottom of the aquifer. For this reason, the dewatering scenarios described in this report include a series of smaller capacity wells that withdraw water from the top of the water column to ensure that the deeper contaminants are not mobilized toward the dewatering pumps.

3.0 PREVIOUS DEWATERING OPTIONS AND PROPOSED ANALYSIS

In 2000, the selected alternative for dewatering the northwest and southeast portions of the City included the well configurations illustrated in Figure 4. In the northwest area, 11 dewatering wells pumping at 500 gallons per minute (GPM) were proposed in a north south configuration along Independence Avenue. In the southeast area, a total of 17 new wells pumping at 300 GPM and 1 existing City well pumping at 1100 GPM were proposed to dewater the southeast part of the City. Although this was the best option at the time, the following items have changed since that time rendering the recommendations invalid:

- The northwest area of high water table conditions has expanded to include parts of the City that have new residential and commercial development. In order to meet achieve dewatering across this expanded area, the wells need to be strategically placed across the expanded area.
- The US EPA has implemented a cleanup program for the GCA No. 2 at the Parkview Well Site and several extraction wells have been installed. If the previously suggested 11 wells were installed as described in the 2000 report, they would interfere with the US EPAs pump and treat system that was installed since the time of the initial report.

- In 2000, the NSC plume was not defined. If the five wells were installed as originally designed, they would intercept the plume and the water pumped from the wells would require treatment prior to discharge.

Due to these issues with the 2000 dewater system design, a new analysis of the best alternative for dewatering the high water table conditions in Grand Island was proposed. The proposed analysis had the same objectives as the original study in that the intent was to lower the static water levels within Areas 1, 2, and 3 1-2 feet below the levels of residential basements. Therefore target level set for dewatering in Areas 1, 2, and 3 was 10 to 15 feet below the ground surface.

4.0 GROUNDWATER MODELING

The following section provides a summary of the methodology and results of the groundwater modeling analysis performed for this dewatering study update. A detailed description of the modeling techniques, all hydrologic parameters and calibration statistics is included in Appendix A. This summary is intended for a general audience. Those that would like to review a detailed description of model development, calibration and implementation are directed to Appendix A.

4.1 COMPARISON TO 2000 MODEL

Before delving into a summary of the model development and results, it is important to consider the differences between the 2000 and current groundwater models. The results of the two analyses are different and reflect the dissimilarities listed in Table 1. It is also important to note that the primary reasons for these dissimilarities is because of the increased computational and storage capacity of computers currently available for engineering work. Additionally, there have been several technological advances in the science of groundwater modeling including the use of parameter estimation simulation techniques (PEST) that test various input parameters and optimize selection of appropriate values during the calibration phase of model development. The list in Table 1 provides a summary of the differences between the two groundwater models.

Table 1 Grand Island Dewatering Model Differences – 2012 versus 2000

2012 Groundwater Model	2000 Groundwater Model
• Land surface based on recent high resolution LiDAR topographic information	• Land surface based on topographic contours from the City of Grand Island
• Lower mode boundary, base of the aquifer, replicates natural conditions	• Lower model boundary set to a uniform elevation across the entire model
• Variable hydraulic conductivity across modeled area were based on inverse calibration of steady state model using Parameter Estimate Simulation Technique (PEST)	• Uniform hydraulic conductivity across the entire model
• Four recharge zones were used in the model – irrigated land, riparian, urban open and urban developed areas.	• Two recharge zones were used – undeveloped and urban
• The Platte River and Wood River were simulated using the stream package which allows for simulation of actual recorded flow rates including no flow during drought conditions.	• The Platte River was simulated using the River Package and the Wood River as a drain. The simulations were based on water table contours from regional maps.
• Transient model incorporated all registered industrial and irrigation (high capacity) wells and including the Grand Island municipal wells whose pumping rates were calculated based on electrical usage.	• No industrial or irrigation wells incorporated into the model.
• Transient model uses a specific yield term that is very similar to values reported by CPNRD from pump tests in the study area.	• Steady state model calibration to high water table conditions observed in 1994.
• Transient model simulates 12-year period from 1999 to spring of 2011. This time period witnessed excessive fluctuations due to drought in the first half of the decade and water table recoveries in the second half.	• Transient simulation calibrated to observed drawdowns at one point in time (144 days). Thus the model did not replicate temporal variations in the water table across the model area.

4.2 MODEL DEVELOPMENT

The new groundwater model was developed in a series of eight separate steps. Table 2 lists the steps and description of the types if information gathered and simulated during each step of the way. Groundwater modeling is done in a step-wise fashion that goes from simple data gathering and analysis to additional complexity in the model simulations. Specifically, the model goes from simulating one point in time to a series of simulations that represent a twelve year modeling period with the proposed extraction wells and groundwater plumes. Detailed information on each step is provided in the Groundwater Model Report along with a description of the steps presents a flow diagram illustrating the process in graphical format (Figure 3.1-1 in Appendix A).

Table 2 The Eight Step Process of Groundwater Model Development

Model Development Step	Description/Comments
1. Data gathering and site conceptualization	Data such as high resolution survey topography, water level measurements, registered well locations, pumping rates, and geologic /hydrogeologic regime defined.
2. Development and calibration of a steady state model to conditions observed in the late 1990s	An initial groundwater model was developed to simulate conditions at one point in time. This was done to evaluate the initial model input parameters and their ability to simulate natural conditions
3. Development of a transient model that simulates the period from 1999 to 2011	Include water pumping data from irrigation and municipal wells as well as changes in recharge across the model to simulate changes in climate and pumping
4. Test and evaluate dewatering well configurations in the Northwest area of concern	Assess the optimal location of dewatering wells and minimize piping lengths where possible.
5. Test and evaluate dewatering well configurations in the South area of concern	Assess the optimal location of dewatering wells and minimize piping lengths and avoid impacts to the Parkview plume.
6. Assess the time of recovery of the water table when wellfield is idle following initial dewatering	This was completed at the request of the City in order to assess operational needs of the system
7. Develop capture zones for new dewatering wells over a 12 year period	An initial check on the impact to groundwater plumes in Areas 2 and 3
8. Assess influence of dewatering well pumping on simulated contaminant plumes on the west and east sides of the South area of concern where known plumes have been mapped and are the subject of federal and state mitigation programs and operations	Further evaluation of the impact of the proposed dewatering wells on the groundwater plumes in Areas 2 and 3.

Figures 5 and 6 present the important groundwater, surface water and pumping well components built into the groundwater model. The model grid is illustrated in Figure 5 and it represents the area covered by the groundwater model. The horizontal and vertical grid lines indicate the size of the grid across the model. The grid is 250 by 250 feet in the center of the City and is larger at the edges of the model boundary. This is to ensure that the accuracy of the model is focused on the central portion where Area 1, 2, and 3 are located. Figure 5 also illustrated the location of the boundary conditions that define the edges of the model, specifically the constant head, stream and drain boundaries built into the model to represent active groundwater flow and surface water influences.

Figure 6 illustrates the location of active high capacity pumping wells. The pumping records for all municipal wells were incorporated into the groundwater model. There were no pumping records available for industrial or irrigation wells and therefore the pumping rates introduced into the model were based on the registered capacity of industrial wells. For irrigation wells, the pumping rates were based on the registered pumping rate on the well registration records along with the number of acres irrigated and climatic conditions for each year of the simulation period.

4.3 MODELING RESULTS

The objective the groundwater model was to optimize the well configurations in the 2000 dewatering well study for Areas 1, 2, and 3. Figure 7 illustrates the updated well configuration based on numerous simulations that included variations on pumping rates, well locations and pumping durations. The results are summarized as follows:

- Dewatering Area 1 required 11 dewatering wells, each pumping at 500 gpm for 6.5 to 7 months.
- Dewatering Area 2 required 16 dewatering wells, each pumping at 400 to 500 gpm for 9 to 10 months.
- The Area 2 wells are strategically located to minimize interaction with the Parkview plume and extraction wells.
- Dewatering Area 3 required 6 dewatering wells, each pumping at 400 to 500 gpm
- Dewatering Areas 2 and 3 initially takes approximately 9-10 months.
- The Area 3 wells are located outside the area to minimize the effect on the NSC plume.
- For Areas 1, 2 and 3, water table recovery approaches the critical surface (10-15 feet below ground surface) in 1.5 to 2 months after the dewatering wells are turned off.

5.0 CONVEYANCE AND DISCHARGE OPTIONS

There were several conveyance and discharge options evaluated as part of this analysis. This section provides information how the different options were analyzed, evaluated and selected.

5.1 WELL AND PUMP LOCATIONS

The first part of the conveyance and discharge analysis was to evaluate the well and pump locations. After the modeling analysis was complete and illustrated the optimal well configuration for the system, the well locations were checked in the field to ensure that the site identified for wells and pumps were accessible for installation, maintenance, and repair. Where possible, sites on City property or Right-of-Way were selected. In Appendix B, detailed maps of the proposed well sites are presented. During final design of the system, these locations will be further checked for other issues related to constructability such as underground utilities etc. Minor changes in location (less than 75 feet) will not require rerunning the groundwater model, however, if significant relocation is required due to access or other constructability issues, a model run will be required to verify that the target dewatering depths are achieved.

5.2 CONVEYANCE SYSTEM ROUTING AND DISCHARGE POINTS

As described in the 2000 Dewatering Study Report, there are two discharge options on the south end of the City:

1. Discharge to the Platte River via the Wood River Diversion Channel
2. Discharge to Wood River

For this updated study, a third option was evaluated:

3. Discharge to Moores Creek Drain that flows into Boy Scout Lake

Looking at hydrogeologic factors alone, the best option would be Number 1: discharge to the Platte River via the Wood River Diversion Channel. This is the best option because the discharge water is routed three miles away from the areas of the City with high water table conditions. As will be discussed in more detail in the Opinion of Probable Cost (Section 6), this option is the most costly. Option Number 3: Discharge to Morris Creek Drain would route the discharge water away from the Areas 1, 2 and 3, however, the drainage ditch does not have a large enough capacity to handle the entire discharge stream from the proposed system. Furthermore, the added surface discharge from the entire system may cause problems in the northeast part of the City.

For these reasons, Option Number 2: discharge to the Wood River was evaluated as an option for the updated dewatering system. The designed discharge would be on the order of 25-35 cubic feet per second. This represents only 15% of the flow in the Wood River and the added volume of water should not be a problem for the area. During the study analysis phase, questions arose about the additional flow that would need to be accommodated in the Wood River such as, 'would the additional flow cause the proposed dewatering system to fail along the western margin of the Wood River in Area 2? The groundwater model indicated that the system was able to achieve the target dewatering levels even when flow was increased. For these reasons, the conveyance for the dewatering system was designed with several discharge points along the Wood River and one to the north into Morris Creek drain. Details of the conveyance and discharge layout are as illustrated in Figure 7, Appendix B and described as follows:

- Area 1 has a total of eleven wells and two discharge points.
    - For ten of eleven dewatering wells in Area 1, the discharge will be routed south either directly to the Wood River or to the Platte River via the Wood River Diversion Channel.
    - The difference between the two options is primarily a cost decision because the volume of flow that is scheduled for discharge from the dewatering system is 25-35 cubic feet per second which is only 15% of the maximum capacity in the Wood River. The difference in cost between a direct discharge to the Wood River and a discharge to the Wood River Diversion Channel is nearly $1.7 million. This cost differential is due to extra pipe and to provide a low-flow liner in the bypass channel.

- The final discharge point for Area 1 is for well #11 on north side of Highway 30. The cost of piping the discharge from well #11 south beneath the highway is too high to justify tying the conveyance piping to the rest of the system. For this reason, the system includes one discharge point at the Moores Creek drain which flows to Boy Scout Lake. The cost to line the discharge channel with concrete is included in the cost estimate for the conveyance system.
- For Areas 2 and 3, there are 22 wells that are subdivided into seven different discharge points.
  - Four discharge points are piped directly to the Wood River and three discharge to drainage ditches that flow to the Wood River.
  - The three discharge points that flow into ditches are for wells #15 and #16 in Area 2 and #20 in Area 3. The locations were designed to minimize piping since installation of piping in heavily developed areas is costly.

5.3 DEWATERING SYSTEMS OPERATION OPTIONS

In the 2000 Dewatering System study, there were two options proposed for operating the system, one was manual operation where an operator would both physically monitor water levels and operate the dewater system pumps based on the water table readings. The second included a computer-controlled operating system where a centrally located computer would operate the system based on remotely monitored water level probes. The cost of the computer operated system monitoring systems has dramatically decreased in the last ten years and so whereas in the initial study, the cost of the computer automated monitoring system may have been prohibitive, in the current analysis, the cost for manual water level monitoring and pump operation is actually more. For this reason, the updated dewatering system has been designed with pumps that are controlled remotely. Additionally, a network of eleven new observation wells are proposed along with four existing observation wells with water level transducers to remotely assess water level readings to evaluate dewatering system performance. The locations of the proposed and existing observation wells are illustrated on Figure 7.

6.0 PRELIMINARY OPINION OF PROBABLE COST

An updated Preliminary Opinion of Probable Cost was developed for the Updated Dewatering System. The costs are summarized in Table 3 with additional backup included in Appendix C. Installation of the system could be completed as a whole, or alternatively, the cost has been subdivided into three phases that correspond to Areas 1, 2 and 3. One advantage to phased construction would be to assist with project financing. Section 7.0 provides information on the financing options for the project and by phasing the project, the capital improvement loan could be divided by Phase which make the loan easier to secure as well as repay.

Another reason that the project has been subdivided into three phases is that due to the ongoing groundwater remediation systems in both Areas 2 and 3, coordination will be required when final design and construction begin. With EPA oversight on the Parkview Well Site project and the NDEQ involvement with the NSC remediation, Olsson anticipates that there will be additional Regulatory Agency review and approval required in these two areas which may cause project implementation delays.

The capital improvement loan that is part of the Probably Opinion of Cost is based on a 20 year loan with two different interest rates, 7% and 3%. The 7% value was included so that the overall project costs could be compared easily with the 2000 estimates. Currently interest rates are at record low values and a 3% interest rate is achievable in the near future since, the Federal Reserve Bank plans to keep interest rates low until 2014 (Federal Reserve Update, April 25, 2012, http://www.money-rates.com/fed.htm). Other items of note in Table 3 include the importance of minimizing the transmission line length due to the cost of the materials and construction of underground transmission lines in the City.

Construction rates were based on cost estimates developed by local contractors on projects in the Grand Island area. No escalation rates were applied to the project costs since the Phasing of project construction change the escalation factors and may skew the costs if the timing of the project is modified.

Table 3 Updated Dewatering System - Probable Opinion of Cost

	Area 1	Area 2	Area 3	Combined Total
Transmission Line	$7,085,000	$1,617,000	$477,000	$9,179,000
Utility Conflicts	$31,000	$28,000	$4,000	$63,000
Dewatering Wells	$939,000	$1,347,000	$498,000	$2,784,000
Control System	$118,000	$135,000	$62,000	$315,000
Construction Cost Subtotal	$8,173,000	$3,127,000	$1,041,000	$12,341,000
Contingency	$818,000	$313,000	$105,000	$1,236,000
Overhead, Legal, Fiscal, Engr.	$981,000	$376,000	$125,000	$1,482,000
ROW Acquisition	$1,877,000	$1,496,000	$496,000	$3,869,000
Total Project Costs	$11,849,000	$5,312,000	$1,767,000	$18,928,000
Annual Costs (20 years, 7%, A/P)	$1,118,462	$501,415	$166,792	$1,786,669
Annual Costs (20 years, 3%, A/P)	$796,439	$357,050	$118,770	$1,272,259

Table 4 and 5 include Estimated Annual and Monthly Costs for the updated dewatering system. In each of these tables, the largest expense for system operation is power usage. The rates included in the cost analysis were based on current electrical rates provided to Olsson by the City. Miscellaneous system repairs and maintenance are the second largest annual/monthly expenditure. This line item includes pump, pump control and observation well monitoring equipment replacement.

Table 4 Updated Dewatering System – Estimated Annual Cost

Item	Area 1	Area 2	Area 3	Combined Total
Labor	$18,000	$18,000	$18,000	$54,000
Power	$120,000	$94,000	$30,000	$244,000
Miscellaneous Repairs & Supplies	$47,700	$68,900	$26,500	$143,100
Total O & M (Per Year)	$185,700	$180,900	$74,500	$441,100
TOTAL Annual Cost	$982,139	$537,950	$193,270	$1,713,359

Table 5 Updated Dewatering System – Estimated Monthly Cost

Item	Area 1	Area 2	Area 3	Combined Total
Labor	$1,500	$1,500	$1,500	$4,500
Power	$10,000	$7,833	$2,500	$20,333
Miscellaneous Repairs & Supplies	$3,975	$5,742	$2,208	$11,925
Total O & M (Per Month)	$15,475	$15,075	$6,208	$36,758
TOTAL Monthly Cost	$81,845	$44,829	$16,106	$142,780

7.0 FINANCING OPTIONS

Significant capital improvements like the Updated Dewatering System presented for the City of Grand Island in this report are typically paid for by municipalities using a combination of different financing options. For this project, although the specific financing options have not changed since the 2000 study, more detail is available about each option and as such, each are described in more detail below.

7.1 DEWATERING DISTRICTS

As presented in the 2000 report, drainage districts for surface water drainage improvements have been used successfully across the State; however, dewatering districts are new in Nebraska. Grand Island's City Attorney has reviewed Nebraska State Statues and has determined that dewatering districts are within the City's jurisdiction to implement. Accordingly, Areas 1, 2 and 3 could be designated as dewatering districts by the City with additional assessments applied to the residents that benefit from the project. It is recommended that a multiple level assessment be developed since some residents may receive more benefit from the project than others. A final rate determination would be developed based on additional study including issues such as the need to account for the size and/or location of the assessed property. The City will determine the appropriate level of assessment to meet the needs of the project, yet remain affordable for its residents.

7.2 USER FEES

Another option for revenue generation presented in the 2000 report was to apply additional charges to the City Utilities user fees. This option is not recommended due to the fact that the user fees would be applied at a flat rate across the City with no adjustment afforded to the portions of the City that are not benefiting from the project.

7.3 OUTSIDE FUNDING

Grants and loans are available through a variety of organizations and in the 2000 Study Report; the Nebraska Department of Natural Resources Development Fund was identified as a potential source of outside funding. This option is no longer available since the Fund is no longer viable.

7.4 REVENUE AND VARIOUS PURPOSE BONDS

The City of Grand Island has the authority to secure bonds to complete capital improvements projects. There are two types of bonds available to the City including Revenue Bonds and Various Purpose Bonds. Revenue bonds are retired with revenue such as from the sale of discharge water for beneficial uses. Alternatively, the Various Purpose bonds are retired with revenue generated by the assessments, operating income or a combination of both. As with the initial study, the Various Purpose bonds are proposed as the best option for the City because of the flexibility in repayment opportunities.

7.5 WATER BANKING

Water banking is one way that the project may be able to generate revenue. According to the Central Platte NRD website (http://www.cpnrd.org/Water_Bank.html), the NRD's Water Banking Program began in January 2007 to try to reduce the need to regulate irrigators within the District. As part of the program, the NRD purchases water rights as a solution to balance water that is being used with water that is available. Two major programs required the NRD to find a solution– the Platte River Recover Implementation Program (PRRIP) and Legislative Bill 962. The NRD must stay in compliance with both of these programs. Currently, the majority of the NRD is at its limit for water use, known as fully appropriated. The western most part of the District, above Elm Creek, is designated as over-appropriated; which requires the NRD to bring water back to a fully appropriated status.

As water demands on the Lower Platte River increase, depletions will need to be offset. Using Central Platte NRDs innovative system of water banking, there may be some opportunity for revenue generation through offsets depletions to the Platte River downstream of Grand Island. This type of activity would be administered through the Central Platte NRD in cooperation with the City of Grand Island.

8.0 IMPLEMENTATION RECOMMENDATIONS

Implementation of a project of this magnitude is impossible without the support of the public. The City continues to experience high groundwater levels based on the frequency and amounts of the rainfall events. Since the 2000 report was completed, the City has experienced periods of wet and dry cycles and that will continue. The following recommendations will provide guidance in the implementation of a comprehensive dewatering system for the City of Grand Island.

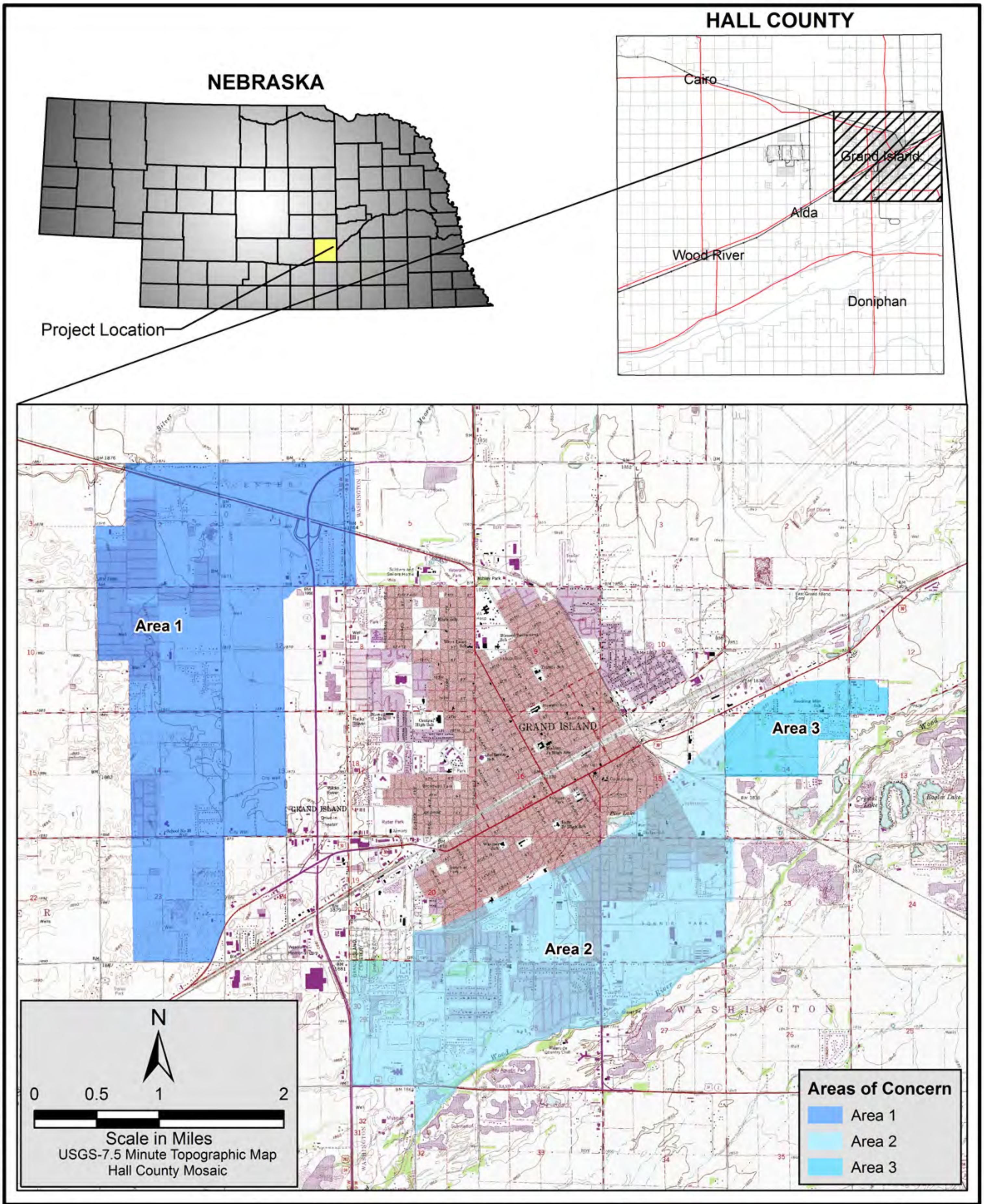
- Conduct neighborhood meetings within the three project areas. Determine the local support and discuss with the residents the Dewatering District concept, and how it would be used to fund the proposed capital improvements and OM&R.
- The City should work closely with the Central Platte Natural Resources District (CPNRD) to review the possible local, state and federal funding sources. Determine if any outside funds could be made available to help finance the projects. Use the CPNRD resources to create a water banking program for future income options.
- The City should proceed to create the Dewatering Districts. Three Districts should be created.
- Determine the financing and revenue sources and schedule, to meet the necessary obligations. Obtain temporary financing to begin the development of the Projects.
- Authorize Olsson Associates to complete project plans and specifications and obtain all necessary permits and right-of-way and /or easements in cooperation with the City, as necessary, for the project construction, operations and maintenance.
- Solicit construction bids for the project from local contractors.
- Construct projects per plans and specifications
- Determine Project Costs and Assessments. Conduct hearing for to make the final assessments to each resident within the benefited area.
- Finalize financing based on the amount of bonds required to pay off the temporary financing and pay for the project over a 20 year period.

The City of Grand Island and the CPNRD should continue to develop this project. The plan proposed in this study consists of a series of vertical wells placed throughout the project area. The wells will be connected together with a system of pipes to transfer the water to the Platte River. The City will be able to monitor and control the system with a centrally located control system.

The groundwater levels continue to plague local residents, not knowing the next time the groundwater will enter into their basements. The property values in the affected would return to the current market values of the City when the projects are completed.

The findings of the updated study are similar to that found in the 2000 report, but the modeling is more refined to better understand the groundwater movement. Construction costs have increased but the interest rates have decreased, resulting in only minor annual cost increases from the 2000 study. The project remains affordable to the City of Grand Island.

FIGURES



PROJECT: 011-2231	STUDY AREA LOCATION MAP City of Grand Island and CPNRD Dewatering Study Update Grand Island, Nebraska	OLSSON ASSOCIATES 1111 Lincoln Mall, Suite 111 P.O. Box 84608 Lincoln, NE 68501-4608 TEL 402.474.6311 FAX 402.474.5160 www.oaconsulting.com	FIGURE
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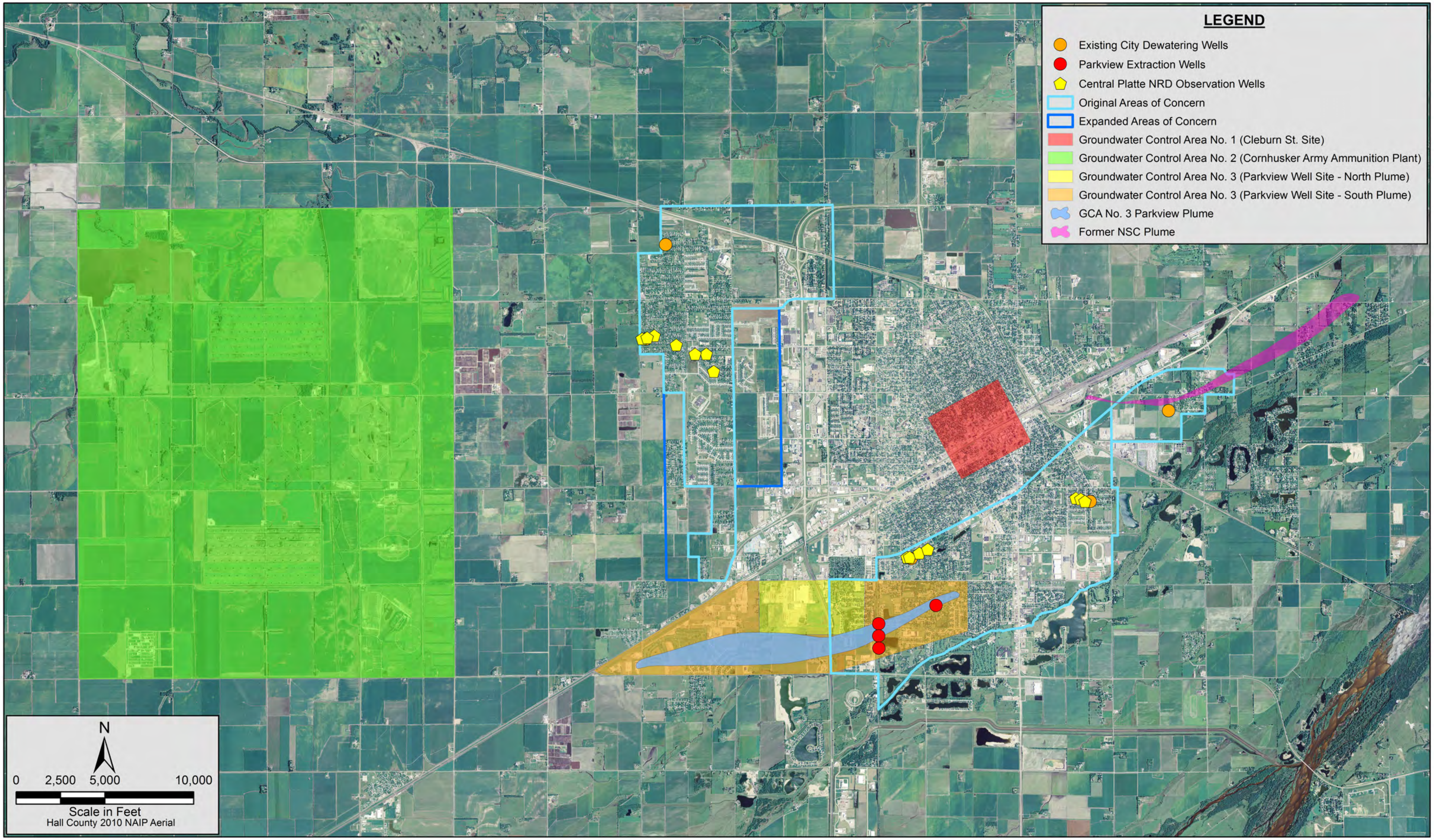

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PROJECT: 011-2231	GROUNDWATER CONTAMINATION PLUME MAP City of Grand Island and CPNRD Dewatering Study Update Grand Island, Nebraska	OLSSON ASSOCIATES 1111 Lincoln Mall, Suite 111 P.O. Box 84608 Lincoln, NE 68501-4608 TEL 402.474.6311 FAX 402.474.5160 www.oaconsulting.com	FIGURE
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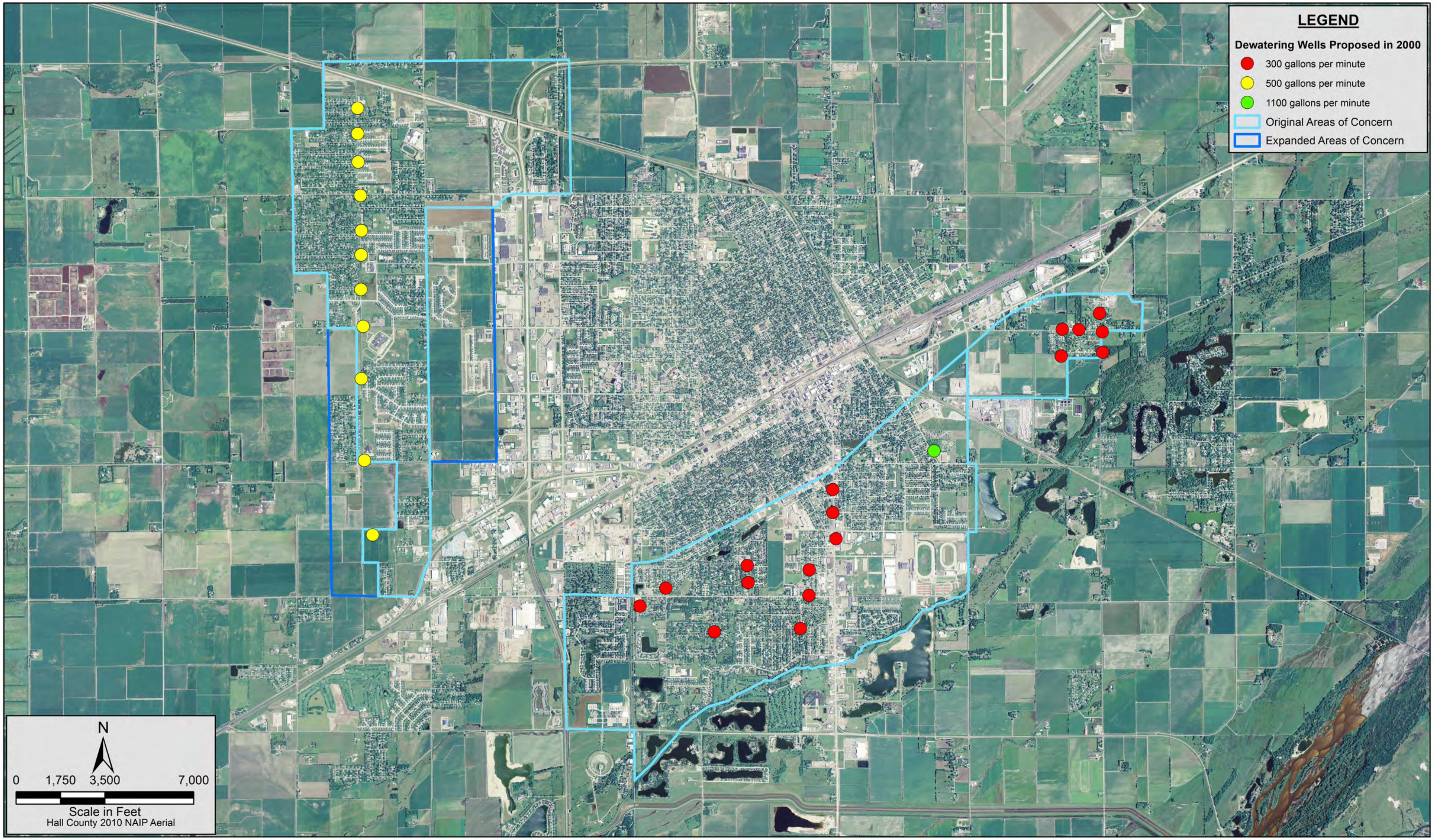
DATE: May 07, 2012			



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OLSSON ASSOCIATES

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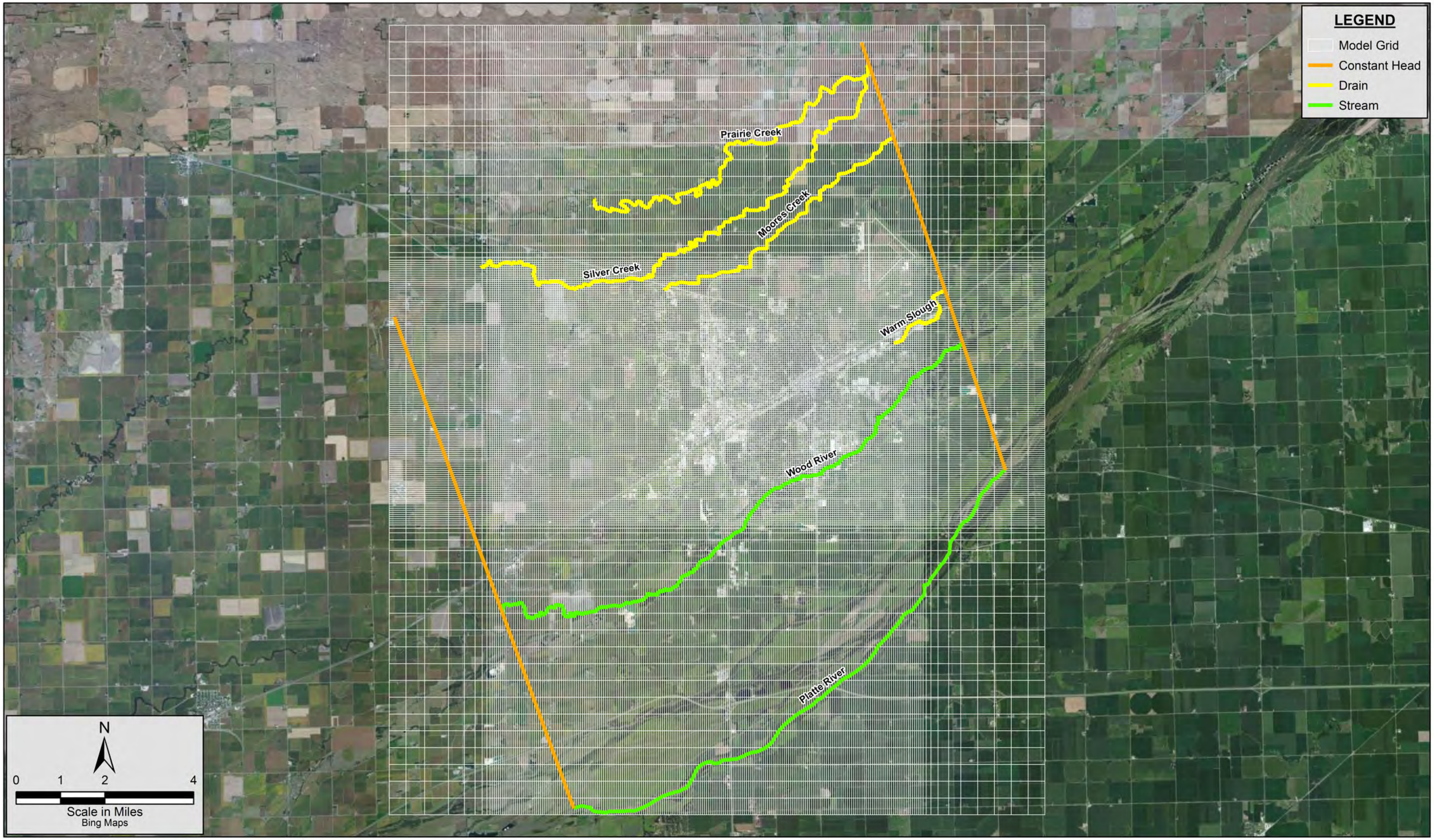

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PROJECT: 011-2231

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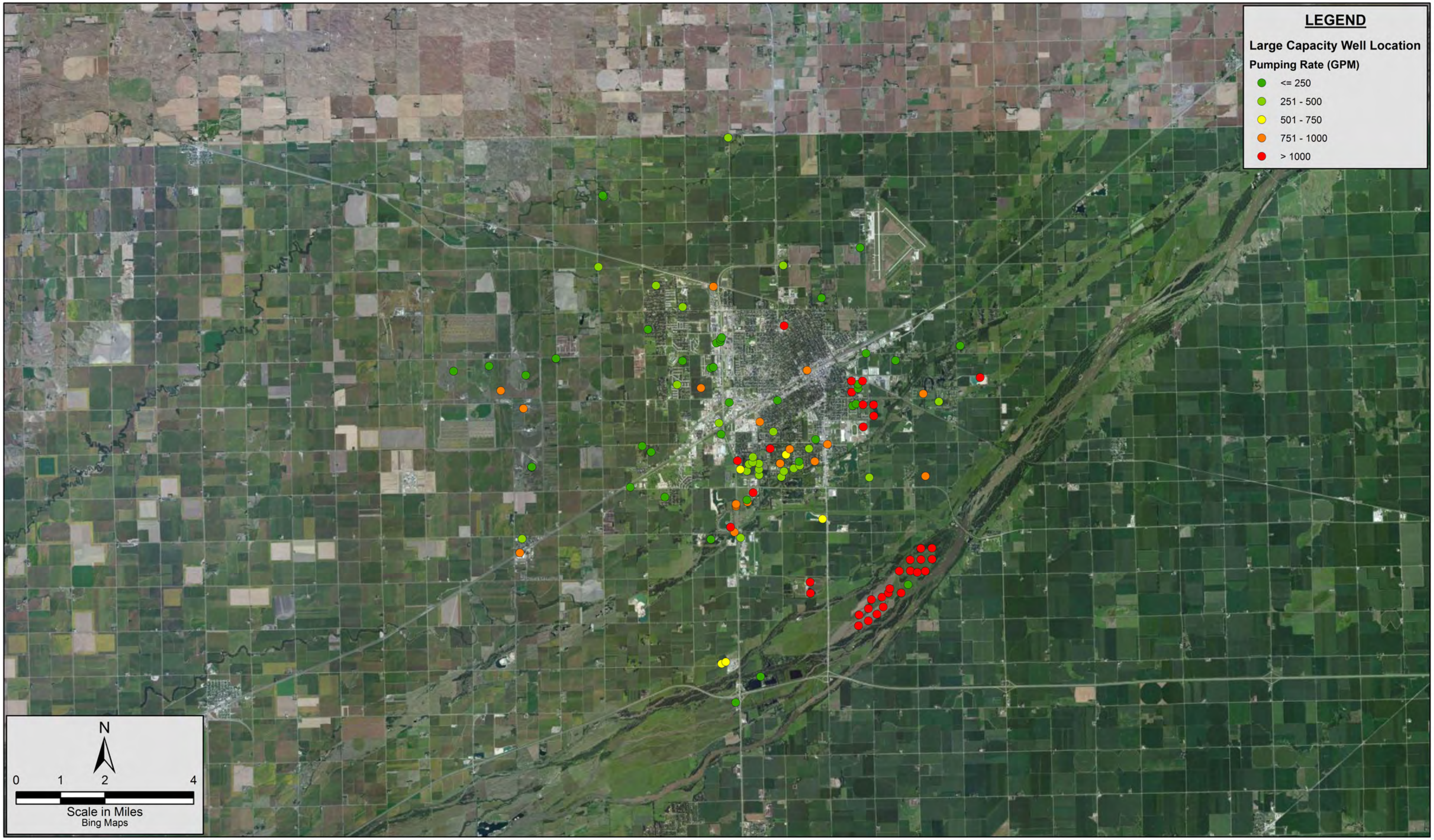
DATE: May 07, 2012

2012 GROUNDWATER MODEL GRID AND BOUNDARY CONDITIONS
City of Grand Island and CPNRD Dewatering Study Update
Grand Island, Nebraska

OLSSON ASSOCIATES

FIGURE 5



PROJECT: 011-2231	**2012 GROUNDWATER MODEL LARGE CAPACITY WELL LOCATIONS** City of Grand Island and CPNRD Dewatering Study Update Grand Island, Nebraska	FIGURE
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DATE: May 07, 2012		

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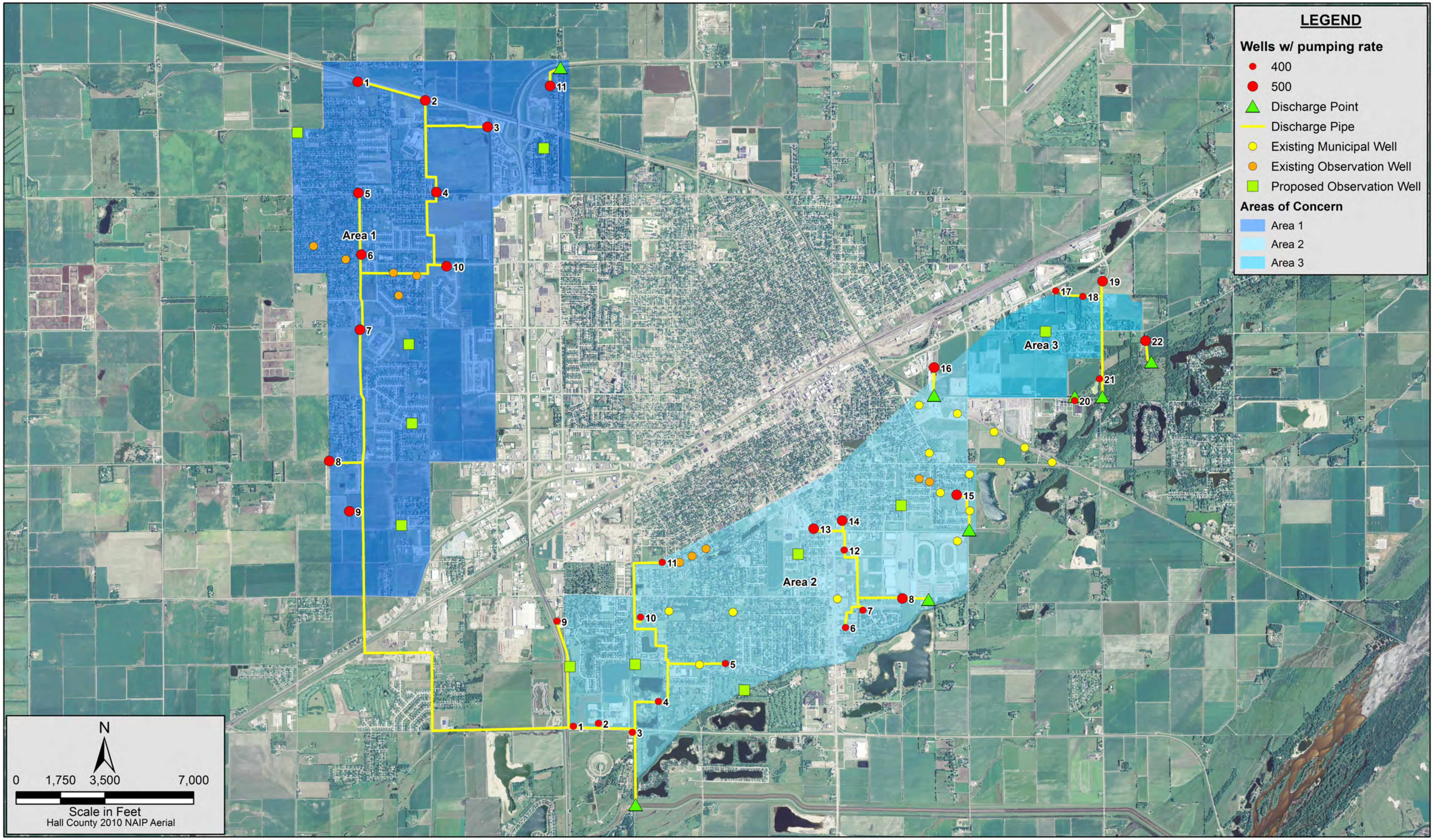

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PROJECT: 011-2231	UPDATED DEWATERING SYSTEM LAYOUT City of Grand Island and CPNRD Dewatering Study Update Grand Island, Nebraska	OLSSON ASSOCIATES	FIGURE
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DATE: May 07, 2012			

APPENDIX A

Groundwater Model Report

DRAFT Groundwater Model Report
for the Grand Island 2012 Dewatering Study Update

Prepared by Olsson Associates for the City of Grand Island
and Central Platte Natural Resources District

Contents

APPENDIX A - Additional calibration hydrographs for the transient model.

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

In 2000, as part of a comprehensive study of dewatering high water table conditions for the City of Grand Island, Nebraska (City) and the Central Platte Natural Resources District (CPNRD), Olsson Associates (Olsson) constructed a numerical groundwater model of the aquifer underlying Grand Island to use as a tool for designing a system of dewatering wells to reduce high water table conditions impacting several areas of the city. At the time of this study, funding options were not available for the City to follow through with designs presented in the findings of the study. In 2011, Olsson again partnered with the City to update the original study conducted a decade earlier. This report presents the development process, results, and recommendations of an updated groundwater model for the Grand Island area.

The groundwater model (model) presented in this report has many general similarities to the model developed in 2000, including the areal extent, two-dimensional (single layer) aquifer representation, and analogous model boundaries. However, with the luxury of nearly a decade of elapsed time, availability of observation data and advancements in model technology and an overall understanding of accepted groundwater modeling practices, Olsson is afforded the ability to improve on the 2000 model version, an enhancement that will provide greater confidence in engineering designs for dewatering high water table conditions in Grand Island. The most significant difference with the updated model is the application of inverse model calibration methods, specifically using the PEST code (Doherty, 1994) with application of the pilot point method. Implementation of pilot points automates the estimation of input parameters such hydraulic conductivity, and the resulting parameter fields are smoothed over the model domain instead of using zones with discreet boundaries between input values. The second major difference in the two models is the more rigorous calibration of both the steady state and transient conditions, especially in regards to the ability of the model to simulate observed water level changes that occurred during the drought of Table 1 presents a comparison of the 2000 and 2012 models prepared by Olsson.

Groundwater model development and application for dewatering analysis was completed in the following sequence of tasks- 1) Data gathering and site conceptualization, 2) Development and calibration of a steady state model to conditions observed in the late 1990s, 3) Development of a transient model that simulates the period from 1999 to 2011, 4) Test and evaluate dewatering well configurations in the Northwest area of concern, 5) Test and evaluate dewatering well configurations in the South area of concern, 6) assess the time of recovery of the water table when wellfield is idle following initial dewatering 7) Develop capture zones for new dewatering wells over a 10 year period, and 8) Assess influence of dewatering well pumping on simulated contaminant plumes on the west and east sides of the South area of concern where known plumes have been mapped and are the subject of federal and state mitigation programs and operations.

2.0 SITE CHARACTERIZATION/CONCEPTUAL MODEL

2.1 Topography and Land Use

The City of Grand Island is situated in the Valleys topographic region of Nebraska as designated by the University of Nebraska-Lincoln's Conservation and Survey Division (UNL-CSD, 1973). Valley regions are defined as flat-lying areas along major streams that are underlain by mixes of stream-deposited silt, clay, sand and gravel. Specifically, Grand Island is situated within the broad Platte River valley, which extends over 14 miles in width near the City. The landscape within the model study area slopes from a high of approximately 1,903 feet above sea level (asl) in the west-northwest corner of the study area to a low point of about 1,820 feet asl just north of the study area's southeast corner at a slope of 0.0011. In the southern half of Grand Island, a pronounced southwest to northeast trending "ridge" in the surface topography separates the land to the north by as much as 10 feet from the landscape to the south and is clearly evident in area topographic maps.

Land use within the model study area is primarily mixed urban with surrounding agricultural land and riparian areas that are crossed by numerous highways and secondary roads. Grand Island's population in 2010 is 48,520 (U.S. Census, 2010) and covers a large portion of the study area. Within the city proper, various land uses exist, including residential, industrial, retail, open spaces/parks, roadways, parking lots, schools, and undeveloped land.

2.2 Climate Conditions

Annual precipitation at Grand Island averages about 25.5 in/yr, with approximately 80 percent of this amount received during the growing season months of May through September. During the transient period simulated in this study, precipitation extremes range from a high of nearly 39 inches in 2007 to a low of just over 17 inches in 2002. The period from 2000 to 2010 witnessed one of the driest periods on record in east-central Nebraska, with below average precipitation in each of the first five years of the decade, followed by four of the last six years of the decade with above average precipitation. Temperatures average 49.8° F in the model study area, with an average annual high of 61.1° F and an average low of 38.6° F (www.usclimatedata.com). Pan evaporation is approximately 65 inches annually in the Grand Island area, which with a cited conversion to lake evaporation rate for larger bodies of water of 0.7, equates to about 46 inches annually of lake evaporation (Penman, 1948, Gutentag and others, 1984).

2.3 Geology/Hydrostratigraphy

Movement of groundwater occurs through the pore spaces between unconsolidated (non-cemented) grains of clay, silt, sand and gravel. Grand Island is underlain by Quaternary-age alluvial sands and gravels deposited by the braided Platte River as it shifted across what is now Hall County during recent geologic time. The Quaternary deposits range in thickness from 70-175 feet across the study area, with a saturated thickness ranging from 50 to 150 feet in thickness depending on location. The Quaternary sand and gravel deposits that form the primary aquifer under the Grand Island area overlie the Pleistocene-age Seward Formation.

This formation, composed primarily of fine-grained silts and clays, forms the lower base of the aquifer, as its permeability is several orders of magnitude lower than the overlying sand and gravel deposits (UNL-CSD, 1998). A 10 to 20 ft thick silty clay bed is found under the northwest part of Grand Island at a depth of around 60 ft bgl (Dreezen, 1999).

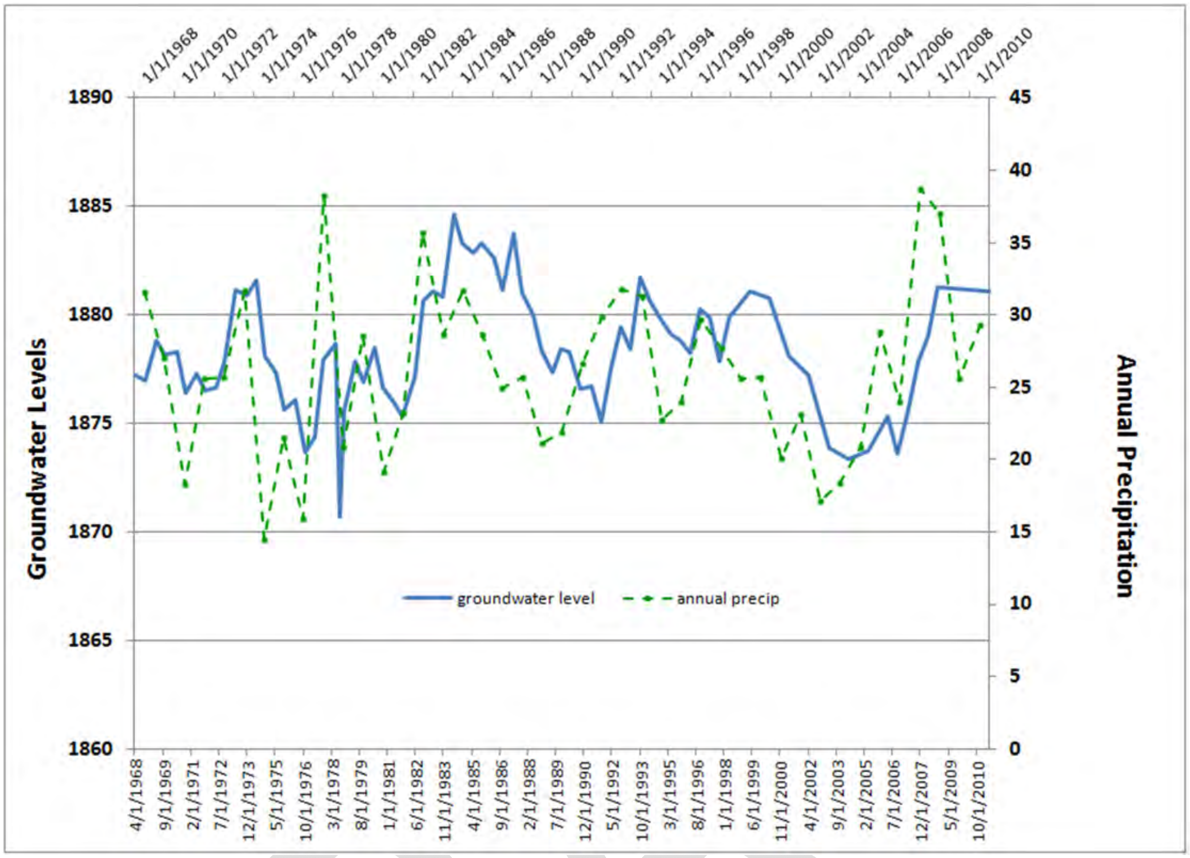
The Ogallala Group is not mapped under Grand Island and most of northeast Hall County, although the formation is encountered both west and north of the study area boundary (Cannia and others, 2006).

Hydraulic conductivity, a numeric representation of aquifer permeability, varies across the site but generally is high enough to allow for development of highly productive wells for irrigation, municipal and industrial use. Various data sources from studies conducted over the last several decades provide a basis of magnitude and spatial variability of the Quaternary deposits in east-central Nebraska's Platte Valley. A U.S. Bureau of Reclamation study of the Platte River study (Johnson, 1970) has mapped hydraulic conductivities (converted from transmissivity) ranging from 174 to 468 feet per day (ft/d) across the study area. The Platte River Cooperative Hydrology Study indicates a range of hydraulic conductivity from 50 to greater than 125 ft/d for that study's "Hydrodstratigraphic Unit 2" which is comprised of Quaternary sand and gravel (Cannia and others, 2006). The Central Platte Natural Resources District (CPNRD) provided Olsson a summary of aquifer (pump) tests conducted by various federal, state, and local entities within its boundaries over the last two decades. Within the vicinity of Grand Island, aquifer tests yield hydraulic conductivities ranging from 133 to 373 ft/d. These results provide a basis for suitable and acceptable ranges of simulated hydraulic conductivities in the groundwater model.

The specific yield of an aquifer is an expression of the volume of drainable water per volume of total aquifer material. Data for this aquifer characteristic is more limited than for hydraulic conductivity. Near Grand Island, specific yield ranges from 0.049 to 0.19. The test location closest to the city was calculated at 0.165. Further downstream in the Platte Valley, an aquifer test yielded a specific yield of 0.11. These are all typical values for alluvial aquifer material (Fetter, 1994), and overall the parameter will show a much smaller range of possible values than that of hydraulic conductivity. The range of 0.05 to 0.2 was used as a basis for values to utilize in calibration of the transient groundwater model.

The water table depth ranges from less than 10 feet to greater than 30 feet depending on location within the study area and the prevailing climate conditions. Because of the relatively shallow water table conditions combined with the coarse deposits in the shallow subsurface, groundwater levels respond closely to precipitation patterns. Figure 2.3-1 shows a comparison of the water table at USGS monitoring well no. 405318098252202 (near the western city limits of Grand Island) and annual precipitation. As shown in this hydrograph, the water table trends in an analogous pattern to precipitation. Deviations between the two plots are most likely attributed to operations of nearby wells. The changes in water levels observed in monitoring wells around the site provided guidance in assigning recharge rates in the model during the transient simulation.

Figure 2.3-1. - Groundwater levels vs. annual precipitation at USGS monitoring well 405318098252202.



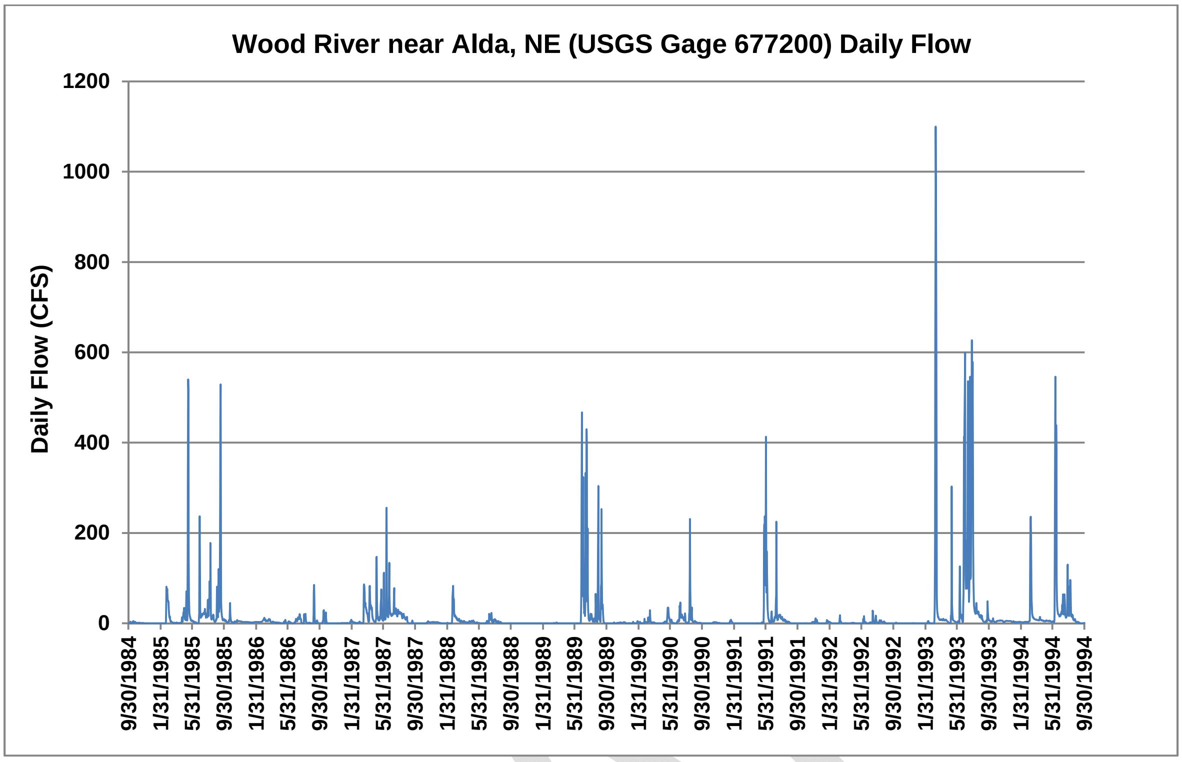
2.4 Hydrology

2.4.1 Surface

The Platte River flows in a northeasterly direction within the model study area. The braided river system contains at least four channels, with shorter interconnecting braids between the main channels in the Grand Island area. Average daily flow at the USGS streamflow gage (06770500) near the US Highway 34 bridge southeast of the city is 1,877 cubic feet per second (cfs). The river has been considered to be either neutral in exchange of water with the aquifer or losing to the aquifer near the City. The river/aquifer relationship can vary seasonally depending on climatic conditions both locally and within the overall river basin.

Wood River parallels the Platte River and crosses the model study area on the south side of the City. Wood River is not considered perennial in the Grand Island area, although it does carry discharge from the Grand Island municipal wastewater system on the east side of the city. However, hydrographs from streamflow gage observations near the west boundary of the model study area do not indicate perennial flow historically, although runoff from summer precipitation events and irrigation runoff are quite common. Figure 2.4-1 shows a sample of these flow patterns for a 10-year period from 1984-1994.

Figure 2.4-1 - Flow rates in Wood River near west boundary of the model domain from 1984-1994.



North of the city, three drainages, Silver Creek, Prairie Creek and Moores Creek cross the model study area. USGS topographic maps indicate ephemeral conditions in these creeks, but conditions do occur during wet periods when these streams will carry perennial flow (personal comm. with D. Woodward, CPNRD, Nov. 13, 2011). East of Grand Island, the ephemeral flowing Warm Slough drains eastward and carries excess runoff from the city.

Over 50 small ponds and lakes, many of which were former gravel pits, are found across the southern half of the city. Water levels in these lakes are a surface expression of the water table and are in direct connection with the shallow aquifer underlying Grand Island. Although the largest lake is approximately 75 acres in size and not a significant body of water, collectively, these surface water bodies cover over 600 acres are capable of evaporating upwards of 2,360 acre-feet annually at an annual lake evaporation rate of 46 in/yr.

2.4.2 Groundwater Flow System

Groundwater movement in the alluvial aquifer underlying the Grand Island area is generally southwest to northeast at a gradient of 0.0013 and roughly parallels the Platte River. The aquifer is considered unconfined, in that the water table freely interacts with and responds atmospheric pressure. Although the presence of a fine-grained silt/clay interval is present within the aquifer under the northwest part of Grand Island, its influence on creating local semi-confined or confined conditions is unknown. Figure 2.4-2 shows the UNL-CSD 1995 regional water table contours across the model study area. This map shows the general orientation of the flow regime, however, it does not highlight local gradients that arise from pumping of the

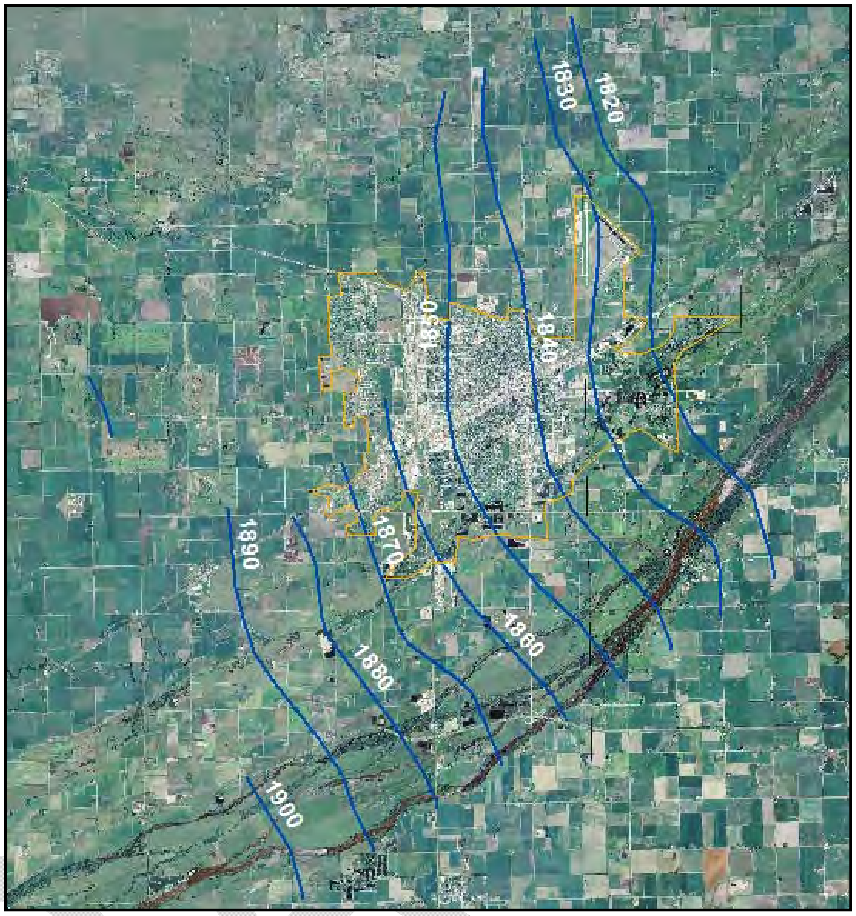


Figure 2.4-2 - UNL-CSD 1995 regional water table contours across the model study area. Orange outline delineates the city of Grand Island. Contour interval = 10 ft.

municipal well fields in Grand Island. Historically, depressions in the local water table have occurred as early as the 1930s, when Wenzel (1940) mapped over 20 feet of drawdown near municipal well locations. With the re-location of a majority of Grand Island's municipal wells south of the city, and the typically rapid flux of recharge to the water table, these depressed conditions have since recovered.

Recharge to the aquifer is primarily derived from precipitation, seepage from surface water sources, and irrigation (both agricultural and urban) return flow. As displayed in figure 2, the typically shallow water table responds rapidly to precipitation events, which would be expected considering the coarse surface soils and materials found in the unsaturated zones. The quantity of recharge reaching the water table varies by land use and other topographic characteristics. Because of the relatively flat landscape in the Grand Island area, land use is the dominant factor that dictates the downward movement of water beyond the root zone. Szilyagi and others (2005) estimated a "long term mean annual recharge" range in eastern Hall County of 1.9 to 2.4 inches per year based on a statewide water balance model. These rates equate to 7.5 to 9.4 percent of annual precipitation in the area. Dugan and Zelt (2000) applied a soil moisture balance model of the Northern High Plains and estimated recharge rates on irrigated lands in east-central Nebraska. Depending on soil type, recharge potential on irrigated land for row crops ranged from 4 to 5 inches annually in eastern Hall County, which equates to 15.7 to 20 percent of annual precipitation at Grand Island. The Platte River Cooperative Hydrology Study recharge application for a simulation period of 1950-1997 averaged a recharge rate on irrigated land of over 6.5 inches or 27 percent of annual precipitation (Peterson, 2007). Recharge rates from these models assume irrigated areas are subject to excess application of water that eventually moves beyond the root zone (deep percolation), or runoff of excess applications accumulate in low areas (ditches) and provide concentrated areas of aquifer recharge.

Seepage from the Platte River and to a lesser extent Wood River can also contribute water to the subsurface. Historical maps of the water table from the 1940s through the 1970s indicated the Platte River being in a neutral state with the aquifer with the river neither gaining or losing. However, the 1979 water table map produced by the UNL CSD (1980) indicates potential losing conditions in the river at that time. This is further substantiated by a USGS investigation (Stanton, 1999) that indicates potential losing conditions southwest of Grand Island and near neutral conditions just south of the city.

3.0 MODEL DEVELOPMENT

3.1 Model Code and Applications

Olsson utilized an array of groundwater model codes and tools to evaluate appropriate wellfield designs with the goal of reducing high water table conditions in Grand Island's two areas of concern. The groundwater model was constructed using MODFLOW_2000 (MF2K), one of the industry's most commonly used groundwater modeling codes. MF2K simulates groundwater flow by dividing the flow regime into a grid where water levels and flows are computed for each individual block within the grid. Groundwater Vistas (version 6.14, Build 20) was used as a graphical interface platform to construct the model grid, enter and modify data inputs, execute and evaluate calibration, and run predictive analyses.

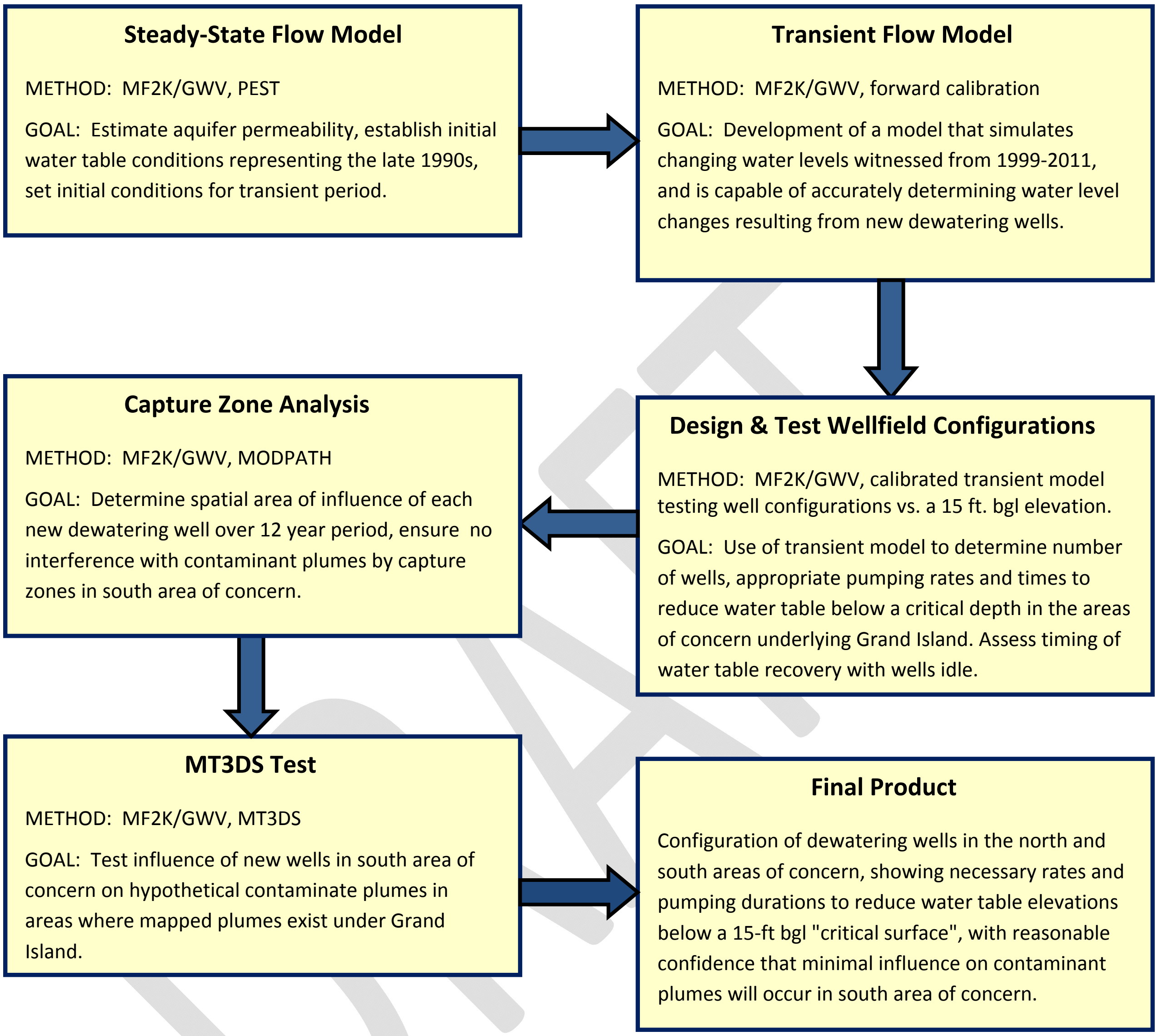
Steady state model calibration was performed using the inverse parameter estimation code PEST (Doherty, 1994). Calibration of the transient model was done by forward (trial-and-error) adjustment of the specific yield, recharge, and estimated pumping rates for area wells (domestic, industrial, municipal) in the model area.

The capture zone analysis, which determines the spatial area of the aquifer contributing flow to a pumping well over specified periods of time, was completed using the particle tracking code MODPATH (Pollock, 1989). This analysis was performed to determine the areas of contribution to each new dewatering well and to assess potential impacts of pumping on established contaminant plumes in the south area of concern. Finally, the three-dimensional solute transport code MT3DS (Zheng and Wang, 1999) was utilized to assess the potential impacts of south area dewatering wells on two hypothetical plumes in locations where two mapped contaminant plumes persist in the aquifer. Figure 3.1-1 shows a flowchart that summarizes the modeling process step-by-step with each individual method and goal.

3.2 Model Structure

The overall rectangular model grid used in this study covers over 263 square miles (Figure 3.2-2). However, the active model area covers approximately 169 square miles. The discrepancy

Figure 3.1-1 - Flowchart of model development and application process.



between these two sizes is a result of applying a traditional grid that is rectangular to an active model domain that is oriented along the axes of the Platte River valley, which angles from the southwest to the northeast in the Grand Island area. This approach was determined the most useful in terms of project time as well as representing the flow regime in relation to the layout of the city and demonstrating results in maps. The active grid area has lengths of 13 miles in both the southwest- northeast and southeast-northwest directions, originating at UTM coordinates x = 2048503.9 and y = 350533.32. The grid is discretized into 162 rows and 224 columns with a cell-size arrangement that telescopes inward toward the city proper where model cells are 250 x 250 ft (1.4 acres). The grid contains a single layer, which represents the entire thickness of Quaternary materials from the land surface downward to the top of the silt/clay deposits of the

Figure 3.2-2. - Study area model grid for the Grand Island area. The developed area for the city of Grand Island in relation to the telescoped grid is delineated by the yellow outline

Seward Formation. Although some vertical heterogeneity likely exists in the model domain, particularly near the clay/silt lens in the northwest part of the city, it was decided that not enough laterally continuous deposits of fine materials are present to warrant multiple grid layers. It was assumed that lateral heterogeneity in the flow regime would dominate in regards to the groundwater flow regime, and that calibration of horizontal hydraulic conductivity would suffice in the reaching a solution of simulated flow conditions.

The top of the grid, which represents the land surface, is based on interpolation to the grid of a highly dense set of GIS points with elevation data from recent LIDAR (Light Detection and Ranging) studies of land surface elevations in eastern Nebraska. LiDAR land surface data is highly detailed and refined, which is advantageous for this study when considering the margin of error needed when modeling water table elevations in relation to basement depths. The bottom grid surface elevations were set by interpolating GIS point elevation data that represent the base of the coarse and gravel deposits in the area depicted in the UNL-CSD borehole data (Dreezen, 1999).

3.3 Boundary Conditions

The governing equations that represent groundwater flow require definition of boundaries for the calculation of water levels to reach a sound solution. Typically in groundwater models, surface water features (if in connection with the aquifer) or geologic structures/materials that inhibit groundwater flow in the flow regime are used as boundaries. When such conditions are not available, water table elevations and/or groundwater fluxes can be used to set fix water level ("head") or flow ("flux") boundaries. Within the model study area, the west and east boundaries are set as fixed (constant) head boundaries. Water level conditions at these boundaries remain constant during the entirety of stress periods, but change between stress periods. This feature of allowing these boundaries to change between stress periods, or "transient constant heads," is an important feature when simulating the drought that persisted from 2001-2004, a time when water levels declined by several feet in the Grand Island area.

The north model border is specified as a "no-flow" boundary, a condition where no flow crosses perpendicular to an assumed flow line that is oriented in the aquifer from west to east. Since the aquifer is unconfined, the water table is considered the upper model boundary, and the base of the Quaternary alluvium is the base of the model domain.

The south model boundary is set as the Platte River with the MF2K Streamflow Routing (SFR) Package. This package allows for explicit simulation of flows observed in the river, and routes the volume of water through cells that are defined as streams. Movement of water across the streambed is determined by the difference in river stage and the adjacent water levels in the aquifer. This exchange of water is controlled by the permeability of the streambed. Streambed permeability was assigned to the SFR package as a vertical hydraulic conductivity, with a value of 139 ft/d based on the average of 21 streambed permeameter tests conducted by UNL researchers (Chen, 2005). Wood River, an ephemeral stream as it enters the model area, was also simulated with the SFR package. Flows do occur at times in the river, and the SFR package simulates each of these flows as they happen in time and space in the model domain. Four north-northeast oriented drainages that typically have ephemeral flow were simulated with the MF2K Drain Package. The Drain Package allows for remove of water from the aquifer when the water table elevation exceeds the base of the drain elevation in each cell. Water is not added to the aquifer from this boundary, however. Figure 3.3.1 shows the active model grid domain with all of the major boundary conditions and the outline of the developed portion of the Grand Island for location reference. Note that significant portions of the model grid are inactive (black) as a method to fix the orientation of the flow regime without tilting the grid.

Inflow to the aquifer from precipitation is represented through MF2K's Recharge Package. Rates were assigned initially as the average rates for four land use types, including irrigated and non-irrigated agricultural land, urban area (mixed paved/lawn), and open urban areas (parks, large fields). Initial values were assigned based on recharge rates estimated by Szilagyi (2005). It was assumed that irrigated agricultural lands would receive the most recharge, and would be at the high end of the range or potential recharge from this study, 2.4 inches, which is about 9 percent of annual precipitation in the Grand Island area. The other three land uses applied slightly less initial recharge, as it was assumed areas with more chance for runoff (paved areas, high roof density) would receive less recharge to the aquifer. Urban open areas and non-irrigated agricultural land were each assigned initial (pre-calibration) rates of 7.5 and 6 percent of annual precipitation respectively, with urban areas receiving 3 percent of annual precipitation at the onset of model calibration. These initial base rates were assumed to deviate during the transient model period of 1999 to 2011, since precipitation patterns fluctuated greatly with some of the lowest annual precipitation rates ever recorded as well as near-record years in precipitation. Adjustment of precipitation-based recharge on a stress-period by stress-period played a key role in calibrating the transient model, especially in the early 2000s during the historic drought period afflicting the region.

Evapotranspiration (ET) from the saturated zone via deep plant roots was simulated via through the MF2K Evapotranspiration Package. Land types that were delineated into unique zones for ET include agricultural land, urban land, riparian zones and open water. Rates applied for each of these zones were based on the 1950-98 COHYST crop irrigation requirement (CIR) rates. Extinction depths, which are the level above which water is extracted through plant roots from the water table, were set at 3 and 7 feet in non-riparian and riparian zones respectively. It was assumed that higher water table conditions in the riparian zones near the Platte River likely have higher water table with vegetative roots that reach deeper than the other land use areas in the model domain. The numerous small gravel pit lakes/ponds were also simulated with the Evapotranspiration Package. Model cells containing these lakes were assigned as ET cells with a deeper extinction depth (10 ft) and an evapotranspiration rate equivalent to a daily rate required to evaporate the annual mapped lake evaporation rate for the Grand Island area (46 in/yr).

Figure 3.3-1. - Configuration of active grid area and major model boundaries. Names of individual surface water features shown near location within grid. The outline of the developed portion of Grand Island is shown in black for location reference

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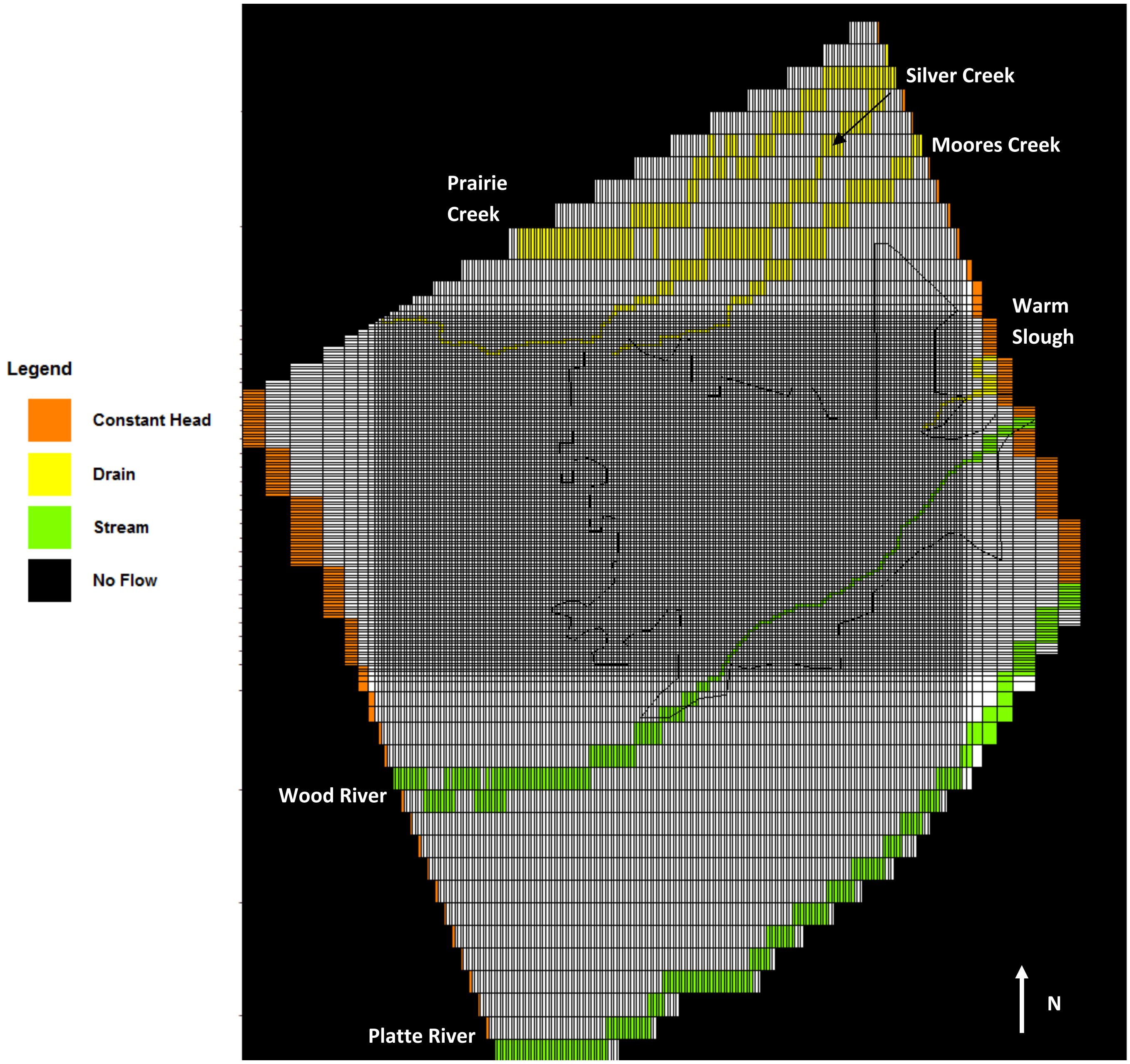


4.0 STEADY-STATE MODEL

The groundwater flow system underlying Grand Island was first simulated to steady-state conditions to establish simulated water levels that represent development period conditions witnessed in the 1990s after development and long-term utilization of both irrigation wells and municipal wells had occurred. The last half of the 1990s showed groundwater levels around 5 to 10 feet above minimum levels observed in the 2000-2005 timeframe. Although the higher water levels in the 1990s are likely attributed to several years of above average precipitation (especially 1993 and 1998) and a corresponding decrease in irrigation and municipal pumpage, the resulting water levels likely represented conditions closer to pre-development conditions and are thus considered appropriate for target levels in the steady state calibration. This process involved calibration of hydraulic conductivity using the parameter estimation code PEST (Dougherty, 1994), with the goal of attaining simulated water levels for use as initial conditions in the transient model. Groundwater levels were calibration to 37 individual observation well locations across the model domain that represent water table conditions in the late 1990s when water levels were considered to be at levels close to predevelopment conditions. These wells include those maintained and monitored by the City of Grand Island, the US Geological Survey, and the Central Platte NRD. Locations (and observed vs. simulated residuals) of the observation locations are shown in figure 4.2-2.

4.1 Parameter Inputs

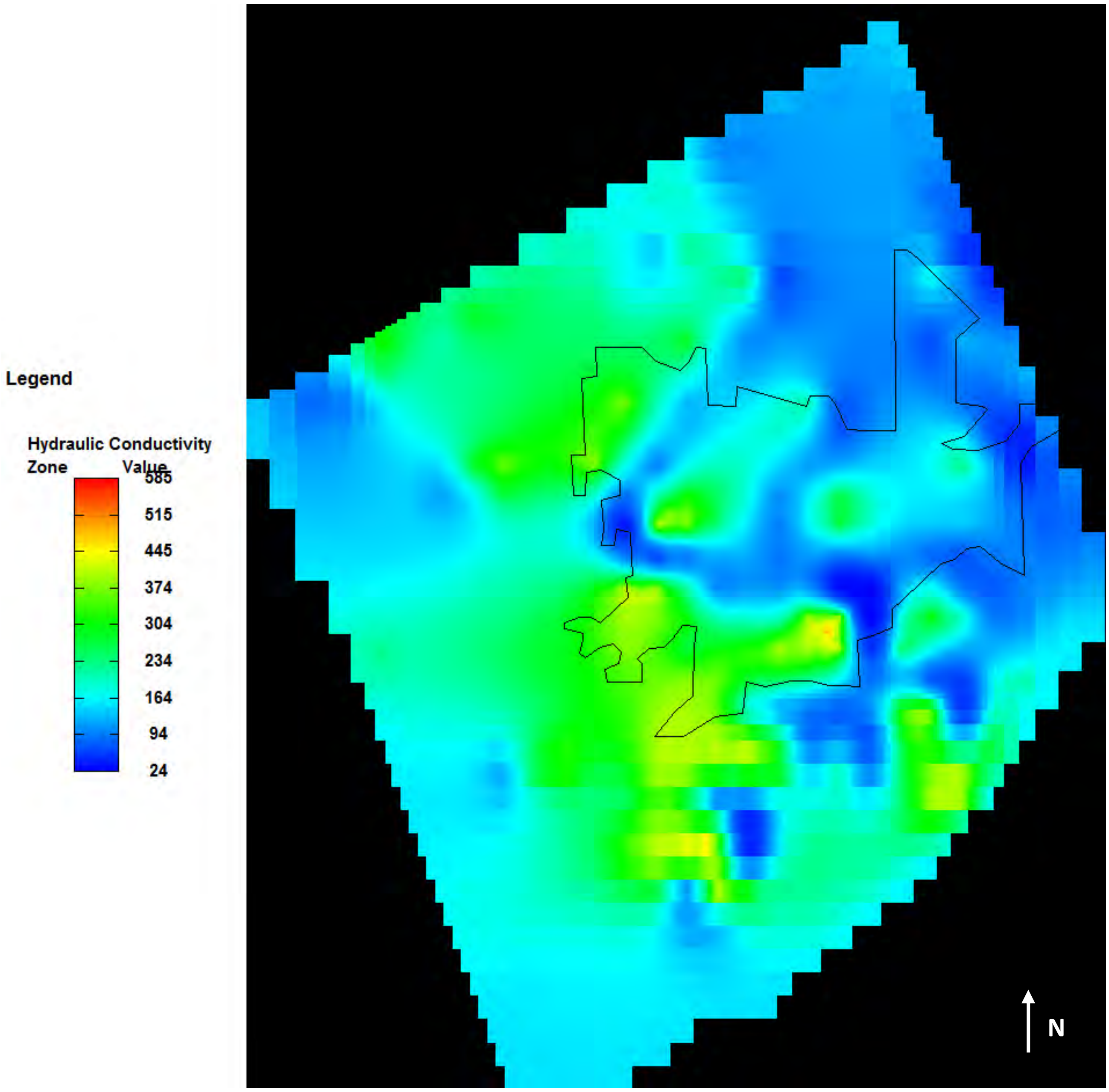
Both fixed and adjusted parameters were applied in the steady-state model. Hydraulic conductivity, a characteristic of the aquifer materials that dictates both rate of travel in the aquifer and groundwater levels, was initially set (but allowed to change during calibration) at an initial value of 200 ft/d across the model grid. This value was set as it represented typical average hydraulic conductivities observed in Platte River alluvium. Calibration of this parameter is discussed in the following section. Recharge was fixed at the base rates for the four recharge zones described in section 3.3. Vertical hydraulic conductivity terms in the MF2K Stream Package was fixed at 140 and 10 ft/d, respectively, for the Platte River and Wood River. The model interface automatically calculated conductance terms from these values with incorporation of each model cell's length and width. Drain conductance in the MF2K Drain Package for the four northeast sloping ephemeral drainages in the model domain was set at 1 ft/d. These values were assigned with the assumption that fine-grained material and decomposed organics are likely present in the beds of these drainages. Elevations for each of these surface features were obtained from contours in the USGS Grand Island 7.5' quadrangle map as well as elevations from Google Earth. Groundwater pumpage was simulated during the steady state period by using well pumping capacity records from the NDNR and associated crop irrigation requirements (for irrigation wells) as well as City of Grand Island pumping records for municipal wells.

4.2 Calibration

The steady state model was calibrated by using a fixed recharge input value and estimating hydraulic conductivity with the automated PEST code as described previously in this report. PEST estimates parameters by continual adjustment through a series of model runs with the goal of minimizing an objective function. The objective function expresses the amount of error between all of the observations from the real world system and their simulated equivalents in the model. A unique feature within PEST is the option to use "pilot points" in estimating parameters. Pilot points are essentially "anchors" where the parameter value is defined in the model domain and allowed to adjust during the calibration process. During each calibration iteration, the parameter values at each pilot point are re-interpolated to the grid before the next run. This approach essentially makes each pilot point a parameter. The advantage this approach has over traditional application of assumed geologic zones is that smoother transitions of geologic characteristics, and thus parameter values, are more easily obtained if an appropriate number and placement of pilot points are applied. This approach is considered superior to arbitrarily drawn zone boundaries, since changes in fluvial-deposited settings are more likely gradational than abrupt, and it is often difficult to defend the placement of a zone boundary where little is known of the subsurface characteristics. The process of parameter adjustment adheres to what the observations indicate the parameters should be instead of pre-defined estimates by modelers of what the parameters should be (Environmental Systems, Inc, 2007). For calibration of hydraulic conductivity, 589 pilot points were used in the steady-state calibration process, and were evenly spaced every 0.5 miles using an automated feature in the graphical user interface. This density of points was considered appropriate for the model area to capture variations in hydraulic conductivity yet allow model computation to complete in a reasonable timeframe.

It should be noted that in groundwater systems without flow observations, high correlation can exist between hydraulic conductivity and recharge parameters. Highly correlated parameters can lead to non-unique models, i.e. models where more than one set of parameter value combinations could lead to the same model solution. Because of a lack of consistent data between two gages on the Platte River not allowing for a true estimate of actual river flow gains/losses, a reliable flow observation was not attainable in the model domain. The same conditions exist for Wood River as well since non-perennial conditions typically persist in that drainage. To bypass this potential problem, OA decided to fix the steady state recharge rate and focus exclusively on the hydraulic conductivity. This was decided after initial trial-and-error tests of these parameters indicated that the hydraulic conductivity values and distribution have much greater influence on the distribution of heads than the range of recharge values expected for pre-irrigation conditions in east-central Nebraska. It should be noted however, that the COHYST project completed an estimate of pre-development gains and losses in numerous Nebraska rivers and tributary streams in the Platte and Republican River basins, including the reach from Odessa, NE to Grand Island (Peterson and Carney, 2002). This report included a normalized mean estimate of exchange of water between the aquifer and the Platte River of -3 cfs per mile (losing conditions). Across 13 miles of river simulated in the model, this equates to a loss of about 40 cfs from the Platte River. With the fixed conductance and flow terms assigned to the SFR Package, the model simulated a loss along the Platte River in the model of about 41 cfs. Although not formally used as an observation, this resulting seepage provides greater confidence in the steady state simulation results even with fixed input parameters and observed flows in the Platte River. The resulting hydraulic conductivity field from the final, calibrated steady state model, along with the pilot point locations used in the calibration process, are shown in figure 4.2-1.

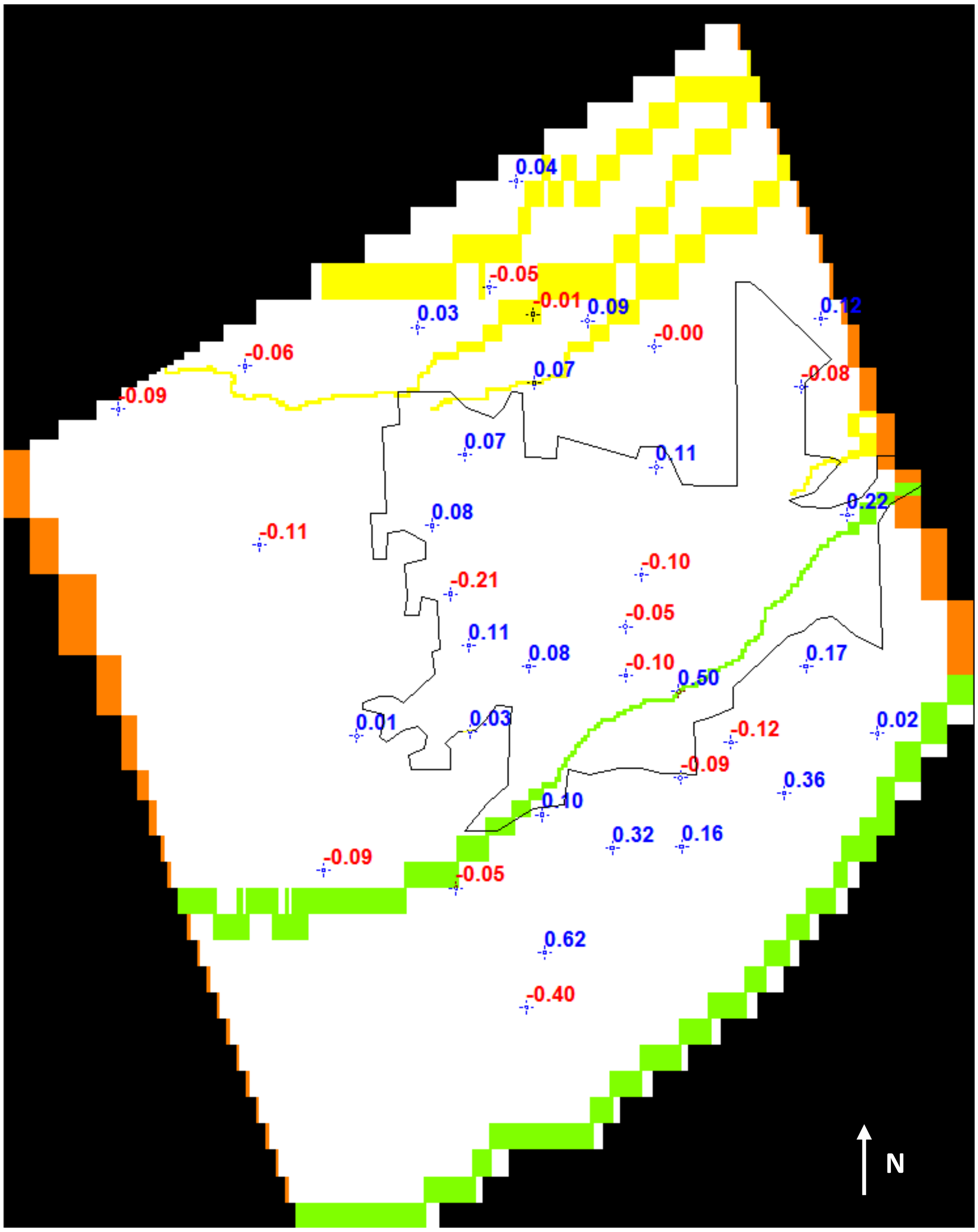
Figure 4.2-1. - Hydraulic conductivity field in steady state model resulting from calibration with PEST (pilot point locations show by cross marks) .



4.3 Steady State Results

Figure 4.3-1 shows final simulated water level contours and the differences (residuals) of simulated and observed heads in the calibrated steady state model. The residuals tend to have no dominant pattern, although the simulated water levels trend slightly above observed levels near the center of Grand Island, and an east to west line of points north of the Platte River show the largest absolute value of residuals (0.16 to 0.62 ft) over the entire grid area. A plot of simulated residuals versus observations (fig. 4.3-2) allows for a graphical demonstration of the

Figure 4.3-1. - Observed vs. simulated groundwater level residuals for calibrated the steady state model at 37 observation locations. Negative values (red) indicate simulated water levels greater than observed, positive values (blue) indicate simulated water levels below observed levels.



randomness or residuals across the model domain, which indicates if bias is present in the simulation. Ideally, an even distribution of points around the zero line should exist. Values in this plot show a reasonable scatter of points, indicating that little to no bias in the hydraulic conductivity parameter field exists and that an evenly distributed, constant recharge rate is acceptable for the steady state simulation. Another demonstration of the quality of calibration is a basic simulated vs. observed water level plot (fig. 4.3-3). Ideally, a perfect model would place all the points on the 1:1 line. The points plot relatively evenly on the 1:1 line and does not reflect bias in any direction.

Figure 4.3-2 - Observed water levels vs. simulated vs. observed water level residuals.

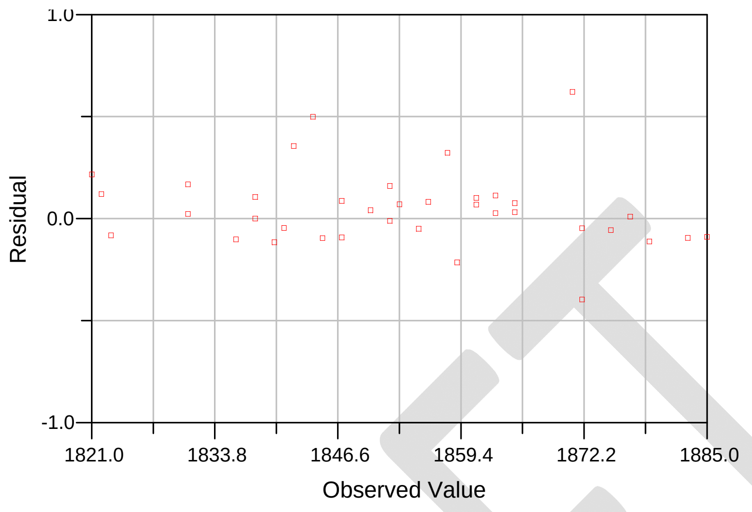


Figure 4.3-3 - Simulated vs. observed water levels for the calibrated steady state model.

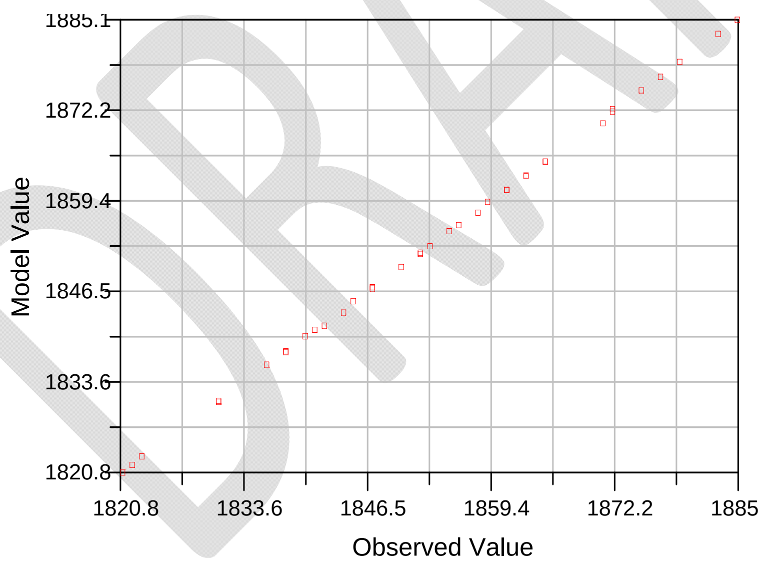
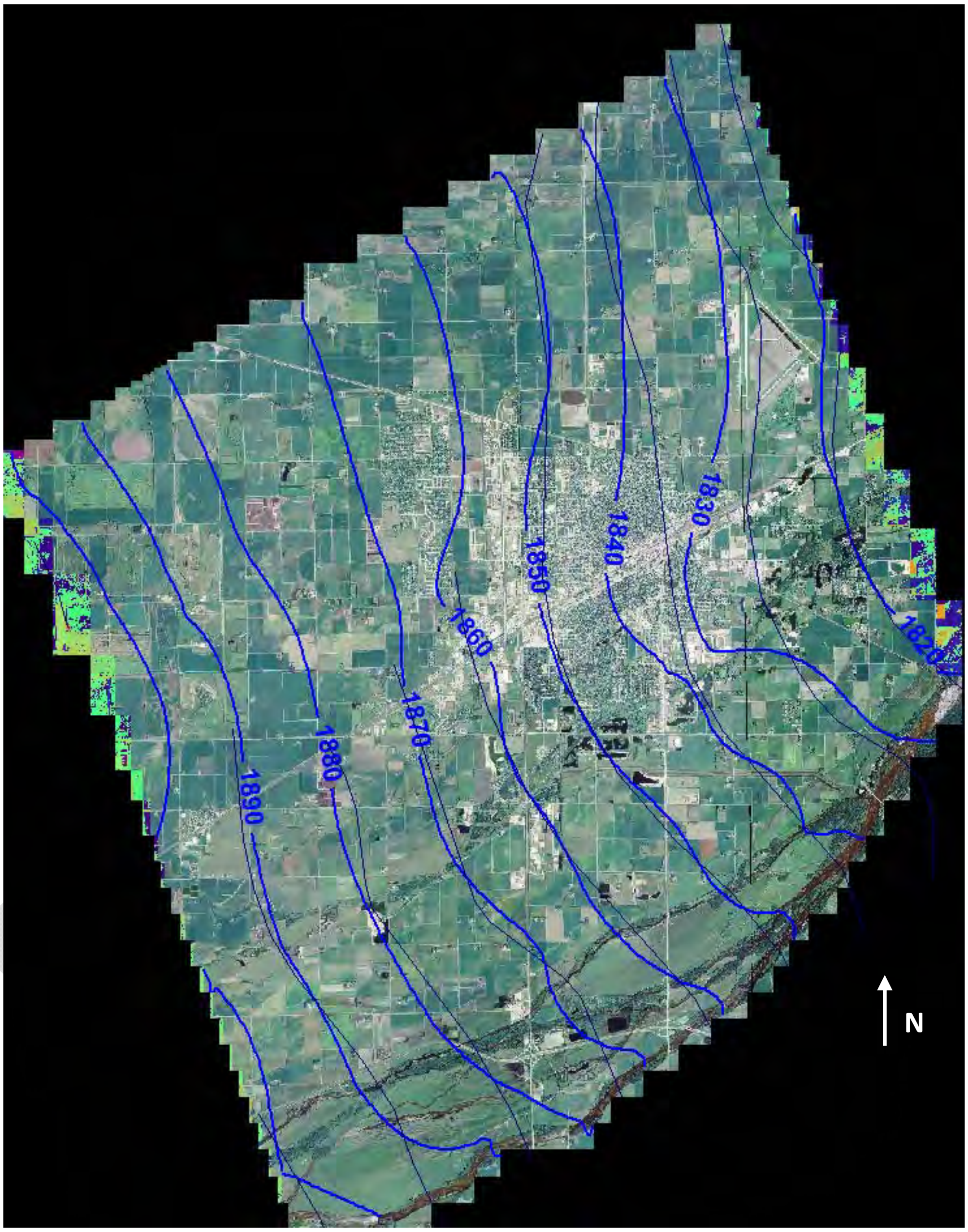


Figure 4.3.4 shows the simulated water table contours plotted across the model domain at a 10 ft interval. Accompanying the simulated contours are the 1995 regional water table contours as defined by UNL-CSD. The contours match closely with the exception of the east part of Grand Island, where the simulated 1830 and 1840 ft contours show a depressed water table in the vicinity of where most of the city's public water supply extraction wells operated prior to the 2000s. Another area of deviation between the two contour sets is at the Platte River where the simulated contours deviate eastward are controlled by the simulated river stage and adjacent

Figure 4.3-4. - Simulated steady state water table contours (blue contours). The 1995 mapped regional water table contours from UNL-CSD are shown in thin black lines.



groundwater levels. The resolution of the 1995 regional water table contours, which cross the entire state, was not detailed enough to capture this local area of water table depression or conditions immediately adjacent to the Platte River.

Table 4.3-1 shows the final model calibration statistics, including the objective function produced from PEST. The total for the objective function, which is the sum of the squared residuals at each of the observation points, is 1.34. The steady state model budget is shown in Table 4.3-2.

Table 4.3-1 Final Statistics from Calibrated Steady State Groundwater Model.

Steady State Calibration Statistics	
Residual Mean	0.05
Residual Standard Deviation	0.18
Absolute Residual Mean	0.13
Residual Sum of Squares	1.34
Minimum Residual	-0.4
Maximum Residual	0.62
Range of Residuals	1.02

Table 4.3-2 Calibrated Steady-State Groundwater Model MF2K Budget.

In ($ft^{3/}d$)		**Percent of In/Out Component Total**
Storage	0	0
Constant Head	1,400,393	18.8
Wells	0	0
Drains	0	0
Evapotranspiration	0	0
Stream Leakage	4,964,665	14.4
Recharge	1,074,494	66.7
TOTAL IN	7,439,521	
Out ($ft^{3/}d$)		
Storage	0	0
Constant Head	1,235,975	16.6
Wells	1,779,309	23.9
Drains	20,718	0.29
Evapotranspiration	2,921,597	39.3
Stream Leakage	1,481,551	0
Recharge	0	19.9
TOTAL OUT	7,439,151	
IN - OUT	**370.5**	
PERCENT DISCREPANCY	**0.0**	

5.0 TRANSIENT MODEL

5.1 Transient Model Development

The transient model for the study area utilized the steady state model as a base framework for modification of inputs needed to simulate conditions that change over time. The transient version of the model was developed to 1) calibrate the specific yield and recharge to demonstrate that the model can reasonably track changes in the water table conditions which change over time and 2) conduct the various engineering assessments for designing wellfields intended to provide long-term reduction of the high water table conditions under the City. The transient model simulates the period from May 1999 through to April 30, 2011, a period that includes one of the most severe High Plains droughts on record from 2001-2004, as well as several years with above average precipitation. This period was considered ideal because of its relatively recent occurrence and the abundance of associated groundwater level records available. As is the case in many locations worldwide, data gaps and lack of records become more prevalent the further back in time information is sought from. Along with the abundance of observation data, the time period was considered idea to calibrate transient recharge conditions and specific yield. The 12 year transient period was simulated with 72 stress periods and based approximately on an agricultural irrigation schedule that is common for central Nebraska. Individual stress periods were defined for the months of May through September each year, and one stress period representing the non-irrigation season period from October through April (212 days). Pumping rates for municipal, industrial and irrigation wells were adjusted during this timeframe.

5.1.1 Calibration Targets

The transient model was calibrated to water level changes observed at select wells that contained records over the entire transient period. Calibrating to water level changes in transient models is common industry practice, as one of the primary goals of transient calibration is to ensure that the model is capable of replicating temporal changes in the water table elevation. This approach reduces the focus on the tedious calibration to water levels measured at a point in time, which in many cases will have an offset between simulated and observed conditions. For each observation location, a base water level datum was selected in the spring of 1999, and water level changes were calculated from the elevation observed at this point in time. Considering the precipitation patterns and water table elevations in the late 1990s, this point in time was considered an appropriate datum to base water level changes on as the water table in the Grand Island was relatively high in comparison to its overall period of record.

5.1.2 Specific Yield

Specific yield is the ratio of the volume of water drainable by the influence of gravity to the entire volume of saturated material. This parameter is applied to transient models only, since water levels do not change over time in steady state simulations. Specific yield plays an important role in calibration of water levels that change over time, and in particular dictates the rate of drawdown and recovery (as demonstrated by the slope in drawdown/recovery curves). During calibration of the transient model, a specific yield value of 0.13 was ultimately arrived at through a trial-and-error process and was fixed for the engineering analyses performed with the model. The High Plains Regional Aquifer Systems Analysis (Gutentag, 1984) reported an average specific yield in the High Plains aquifer of 0.15, with the area near Grand Island in the 0.10-0.20 range. As previously mentioned, aquifer tests reported by the CPNRD in the Grand Island area also reported specific yield values of 0.11 to 0.16.

5.1.3 Pumpage and Recharge Inputs

Along with adjustment of recharge and specific yield, groundwater pumpage from industrial, municipal and irrigation wells was also modified during the transient timeframe. Because the city covers a substantial portion of the model study area, particular focus was placed on adjusting pumping rates within the city limits where many of the transient water level observations are located. The city of Grand Island provided Olsson pumpage volumes by month for many of its municipal wells. For these wells with available data, total volumes were converted to daily rates and only required slight modification during calibration. For other municipal wells and industrial wells without records, initial rates were set at the registered pumping rate listed in the NDNR registered well database for each well, pumping for full 24 hour periods. Initial transient runs indicated excessive simulated drawdown, and subsequent runs incorporated pumping rates calculated with a shorter assumed pumping duration each day (6-12 hours). The city of Grand Island also provided pumping volumes for dewatering wells located in both the northwest and south areas of concern.

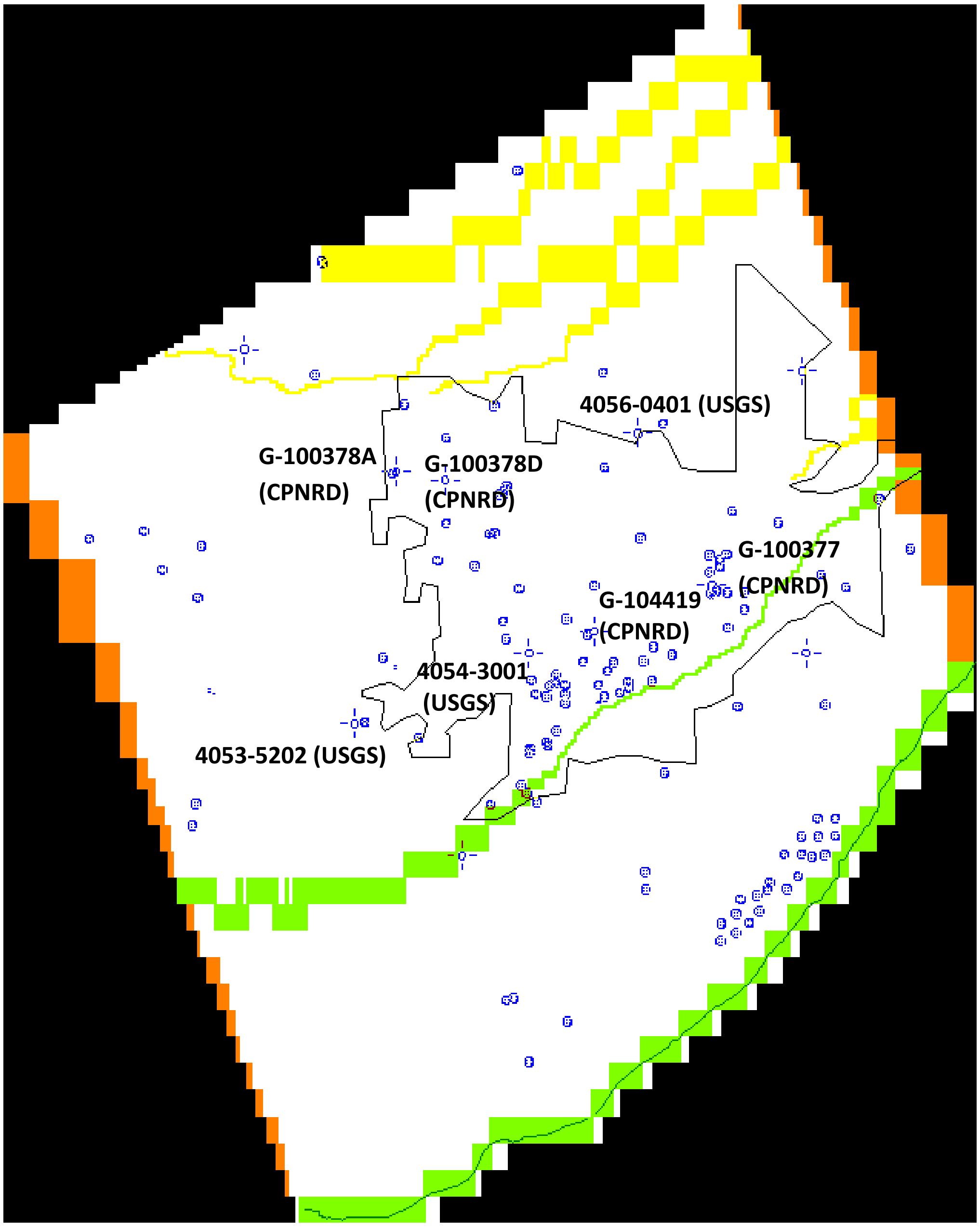
Pumpage rates assigned to area irrigation wells was based on a rate determined by the net irrigation requirement (crop irrigation requirement less growing season precipitation) for corn and soybeans (based on 2005 CALMIT land use map). The volume of water required for each well was simulated with an extraction rate that pumped this total volume over the entire growing season (May-September). This approach approximates the total volume needed over a growing season, but likely deviates from actual pumping patterns as irrigators likely turn wells on and off throughout the growing season depending on crop growth stages and precipitation patterns. Simulated wells and transient water level change observation points are displayed in figure 5.1-1. It should be noted that in creating the transient pumpage dataset, all wells that were registered with the NDNR at a rate of less than 50 gallons per minute (gpm) were excluded from the simulated wells. These wells were typically domestic or low-capacity commercial use wells.

Recharge in the transient simulation was adjusted with multipliers of the base recharge rates applied in the steady state model (Section 3.3). Recharge ranged from 1.5 percent of the base recharge rates during the worst periods of the drought (2002-04) to a multiplier of 3 times the base rate in 2008 when annual precipitation was about 60 percent above the average amount of 25.5 in/yr.

5.2 Transient Model Results

Matches of simulated versus observed water level changes are displayed for four monitoring well locations in Figure 5.2.-1. The wells displayed are from different locations (west, central, and east) across the city of Grand Island to show the spatial variability in how water levels changed across the city during the transient model period. Appendix A displays the remaining hydrographs for the wells in the transient model calibration (all of which have incomplete observation periods of record). Because of the intended use of the model (dewatering), calibration effort focused on ensuring that the simulation is capable of attaining the maximum water level increases or decreases observed at the monitoring wells, and that the simulated changes trends mimic the observed changes. From the hydrographs displayed in figure 5.2.-1, the model is capable of reaching these maximum and minimum water level changes observed across the model area, although at two locations, the simulated changes recover at a slower

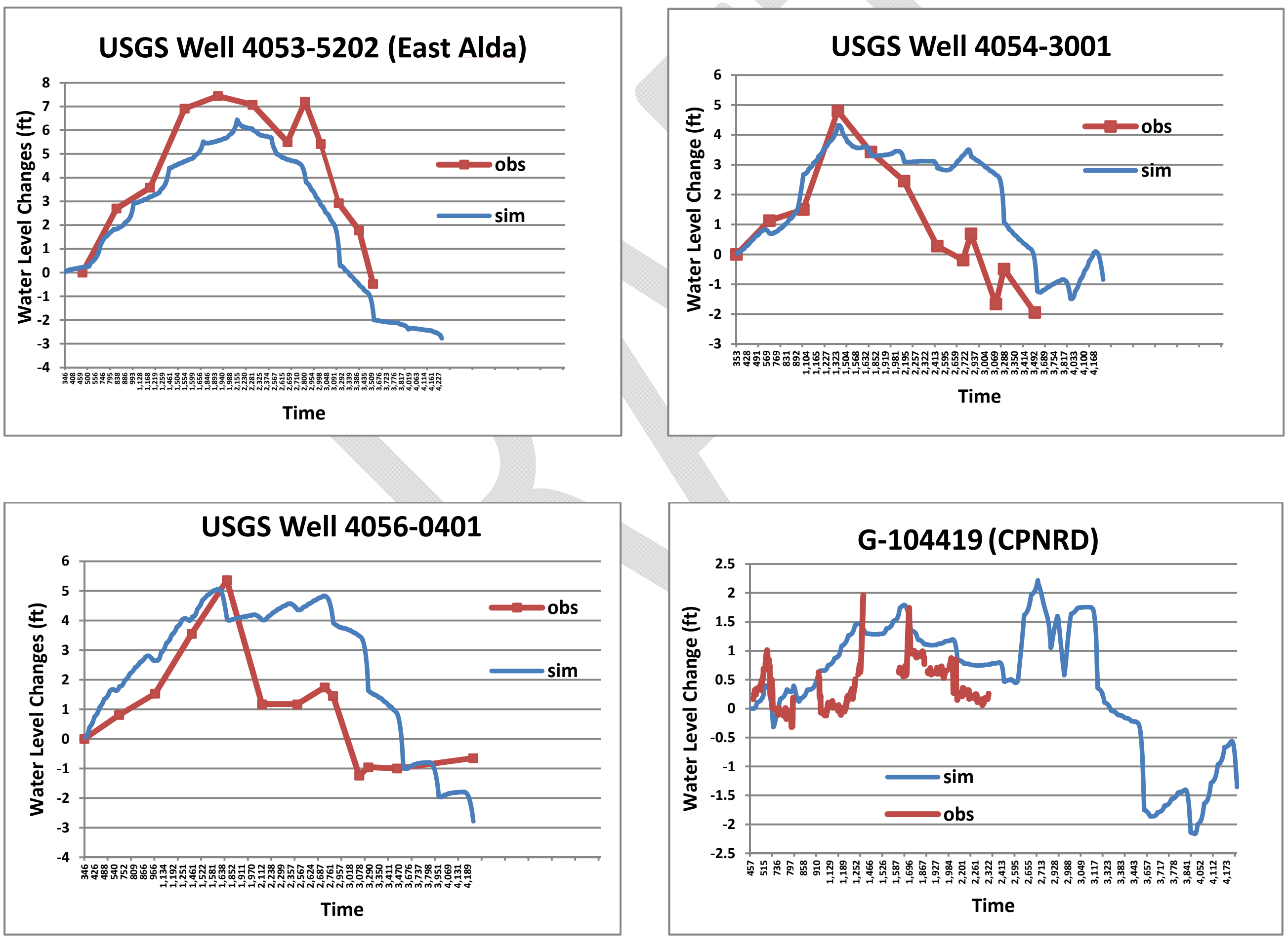
Figure 5.1-1. - Locations of wells (circles) and water-level change observations simulated in the 1999-2011 transient period. Monitoring wells are labeled by USGS or NDNR registration number.



rate than the observed data. This condition could be due to excess simulated pumpage in nearby wells where simulated pumpage rates are estimated using limited information. Further calibration of individual pumping rates could bring these matches closer, but for the intended uses of the model, this was considered unnecessary with the fac that the model appears to satisfactorily mimic the large swing in water levels witnessed between 1999 and 2011.

Tables 5.2-1 and 5.2-2 list the model calibration statistics and final cumulative model budget, respectively. The mean absolute residual between simulated and observed water level changes is 1.8 feet across the monitoring well's 5,163 water level change observations between 1999 and 2011. The mean water level change residual is -0.87, indicating that the simulated water level changes are slight greater than the observed changes on average, by slightly less than one foot. The maximum and minimum residuals were 6.7 and -9.3 feet respectively. These residuals likely originate from monitoring well 100378-D (Appendix A), a well that appears to show rapid responses to pumping that the model could not account for, likely due to either a well near the observation location not simulated or from semi-confined conditions in the area of the wells.

Figure 5.2-1 - Hydrographs of simulated vs. observed water level changes during the transient model period. Note the x-axis lists time in model units.



The cumulative model budget for the transient model reveals that inflow of water from the Platte River is a substantial component of flow in the groundwater system in the Grand Island area, nearly three times the volume of precipitation based recharge. This condition would be expected considering that Grand Island has been pumping from a municipal wellfield adjacent to the river over much of the transient model time period and from this stress induces seepage from the river channels into the aquifer. Pumpage from wells and evapotranspiration account for nearly

75 percent of the outflow component of the model budget. The percent discrepancy between the in- and outflow components is 0.04 percent.

Table 5.2-1 Final transient calibration statistics.

Transient Calibration Statistics	
Number of Observations	5,163
Residual Mean	-0.87
Residual Standard Deviation	2.2
Absolute Residual Mean	1.8
Residual Sum of Squares	28,900
Minimum Residual	-9.3
Maximum Residual	6.7
Range of Residuals	15.4

Table 5.2-2 Transient model budget showing cumulative volumes at the final time step of stress period 72. Note that the units are in acre-feet.

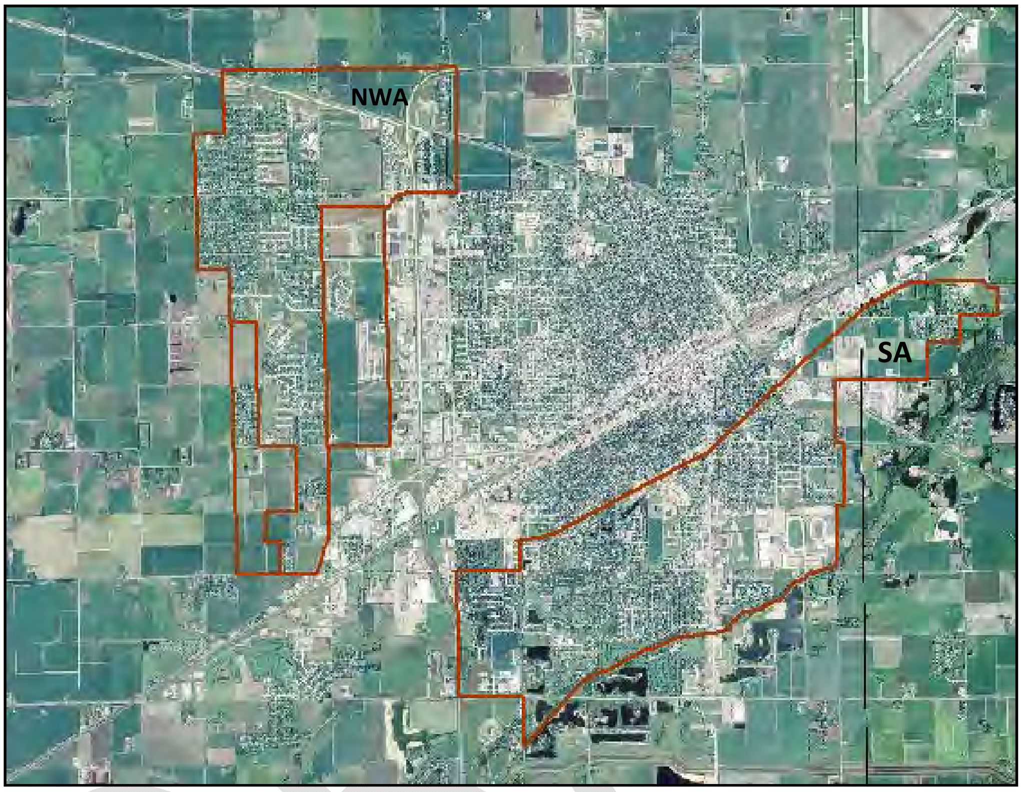
In (acre-feet)		**Percent of In/Out Component Total**
Storage	130,501	15.0
Constant Head	104,084	12.0
Wells	0	0
Drains	0	0
Evapotranspiration	0	0
Stream Leakage	155,690	55.2
Recharge	479,915	17.9
TOTAL IN	870,190	
Out (acre-feet)		
Storage	116,774	13.4
Constant Head	80,055	9.2
Wells	397,112	45.7
Drains	4,714	0.5
Evapotranspiration	243,902	28.0
Stream Leakage	27,285	3.1
Recharge	0	0
TOTAL OUT	869,842	
IN - OUT	**348**	
PERCENT DISCREPANCY	**0.04**	

6.0 DEWATERING ANALYSIS

The calibrated transient model was utilized to test various configurations of dewatering wells in both the northwest area of concern (NWA) and the south area of concern (SA) in Grand Island

(Fig. 6.0-1). The objective of this phase of the study focused on optimizing a configuration of wells in each area that could achieve long-term reduction of the water table to levels consistently below residential basements.

Figure 6.0-1: Northwest (NWA) and South (SA) areas of concern for high water table conditions in Grand Island.



6.1 Methodology

The calibrated transient model was applied to this exercise without altering the time structure of the model. In many situations where models are used to assess future conditions or test modifications in system stresses, additional time is added to a model to represent some future scenario. However, in this case, it was decided that this time-consuming process would be unnecessary since the anticipated timeframe of dewatering would be on the order of months instead of years. With this approach, it was determined that in the last two years of the transient simulation period (May 2009 - May 2011), average to above average precipitation patterns helped return water table conditions to pre-drought conditions at levels that would be considered "normal" or possibly "wet" as some hydrographs (fig. 5.2-1) showed water levels recovering above 1999 observations. The two year window of normal to wet conditions during the transient period was considered opportune for testing well configurations in areas of concern.

Dewatering the water table to at least 15 feet below the land surface was defined as the target goal for the wellfield simulations in the NWA and SA. This criteria was set with the assumption that the base of a typical residential basement in Grand Island did not exceed 10 feet below the land surface. An additional five feet was added to this depth as a conservative precaution with model error and deviations/inaccuracies in the land surface elevations taken into consideration. This surface was created by subtracting 15 feet from the LiDAR-based land surface dataset (top of the model grid) at each model cell. This new "critical level" at 15 feet below the interpolated land surface in the model was then used to compare water levels simulated with the various wellfield test designs.

Wellfield design testing involved placement of new wells in the NWA and SA one well at a time to determine the effectiveness one new could induce on the water table and to determine what areas required higher density of extraction wells. The simulated water table was compared to the critical surface after each model run to assess where placement of the next well would be most effective. This process was repeated until the water table was below the critical surface across each of the areas of concern. Well capacities were tested with rates of 300 to 500 gallons per minute (gpm).

6.2 Dewatering Results

6.2.1 Northwest Area

A total of 11 wells across the NWA, each pumping at 500 gpm, reduced the water table below the critical level after about 9 months of pumping (Fig. 6.2-1). It should be noted that a majority of the area becomes dewatered below the critical level in the 6-7 month timeframe, but a few isolated zones on the fringes of the NWA remain until the 9 month mark. Figure 6.2-2 shows the locations of the NWA dewatering wells. It should be noted that many of the lingering areas above the critical level are very small in magnitude, typically on the order of less than one foot and with this condition, dewatering below most basements should have already occurred well before nine months have elapsed.

6.2.2 South Area

The SA required 22 wells to lower the water table below the critical level (Fig. 6.2-3). The timeframe for dewatering to occur across most of the SA takes about 2-3 months longer than for the NWA, as initial water table conditions in the SA are typically higher in relation to the critical level than areas in northwest Grand Island. By the 10 month point, the water table was below the critical level across the entire SA with the exception of small areas in the northeast corner. Two contaminant plumes are present in the west (Parkview - managed by US EPA) and northeast (VCP - managed by the Nebraska Dept. of Environmental Qualtiy) areas of the SA. Wells that are closer to the plumes (within 0.5 miles) are simulated at 400 gpm and wells beyond this distance from the plumes are simulated at 500 gpm. The dewatering wells in the northeast corner of the SA, some of which are situated beyond the SA boundary, were not placed in the area of remaining critical level exceedence in order to not disturb the contaminant plume crossing the area. When cleanup operations are complete and contaminant levels fall below the NDEQ requirements, additional wells could be placed in this area to further lower the water table below the critical level. The progress of dewatering over time is displayed in Figure 6.2-4.

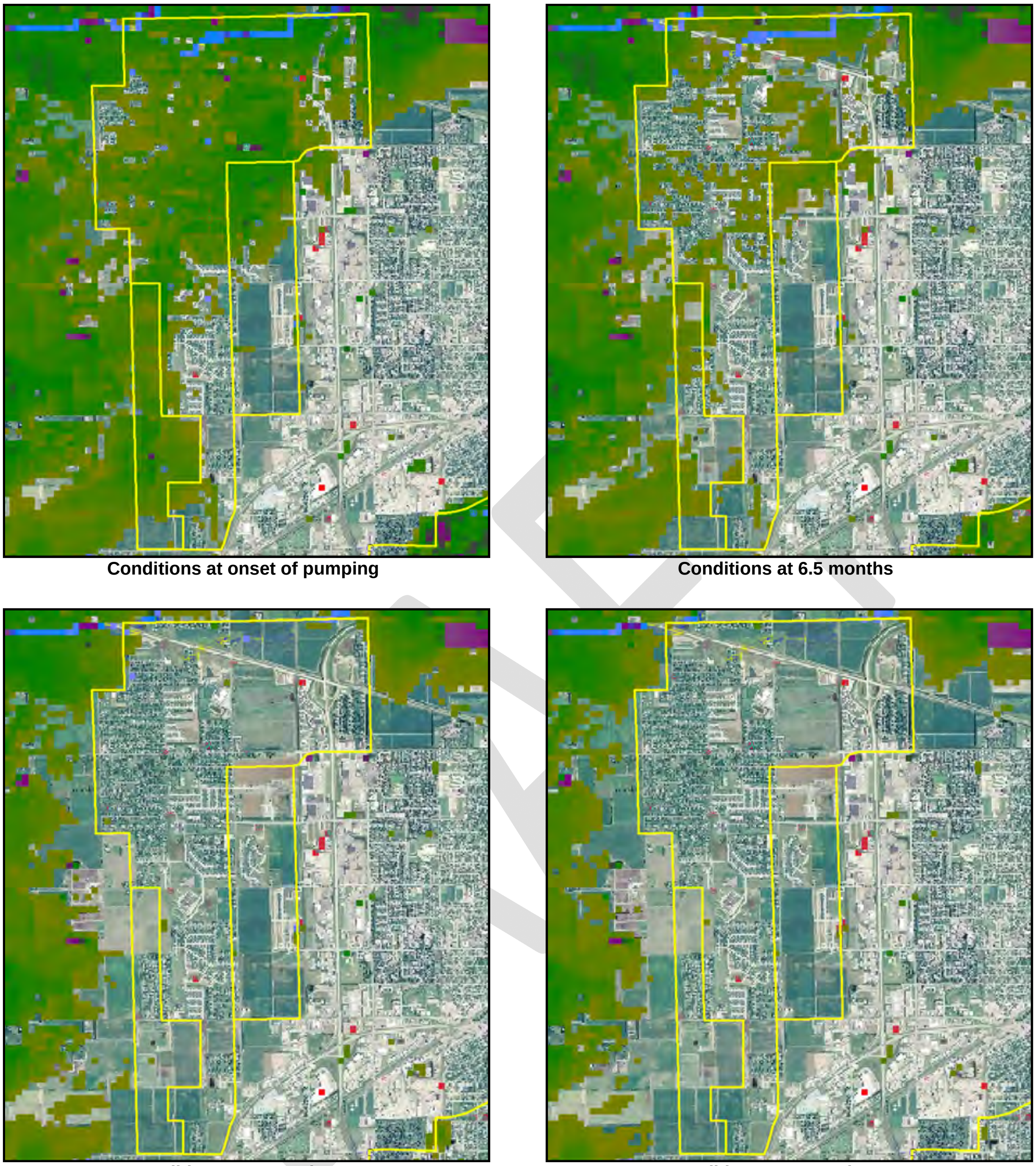


Figure 6.2-1- NWA dewatering over 9 month period. Negative values on scale (right) indicate level (in feet) water table exceeds critical level (value is negative due to calculation of critical surface - water table).

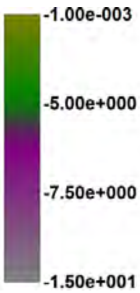


Figure 6.2-2 - Locations of 11 dewatering wells in the NWA.

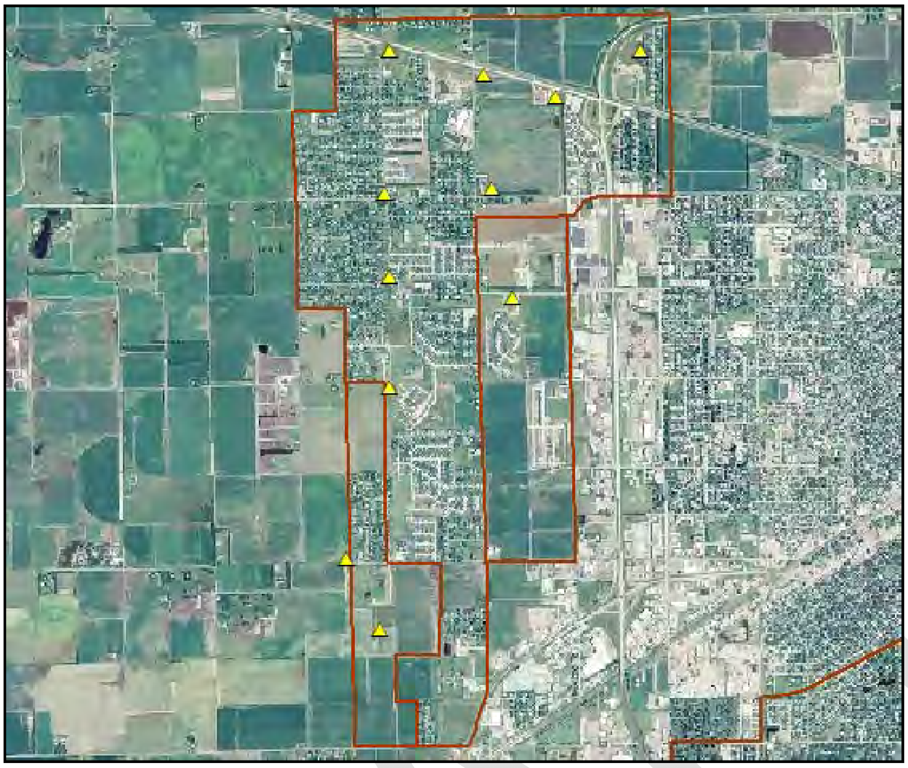


Figure 6.2-3 - Locations of 22 dewatering wells in the SA.

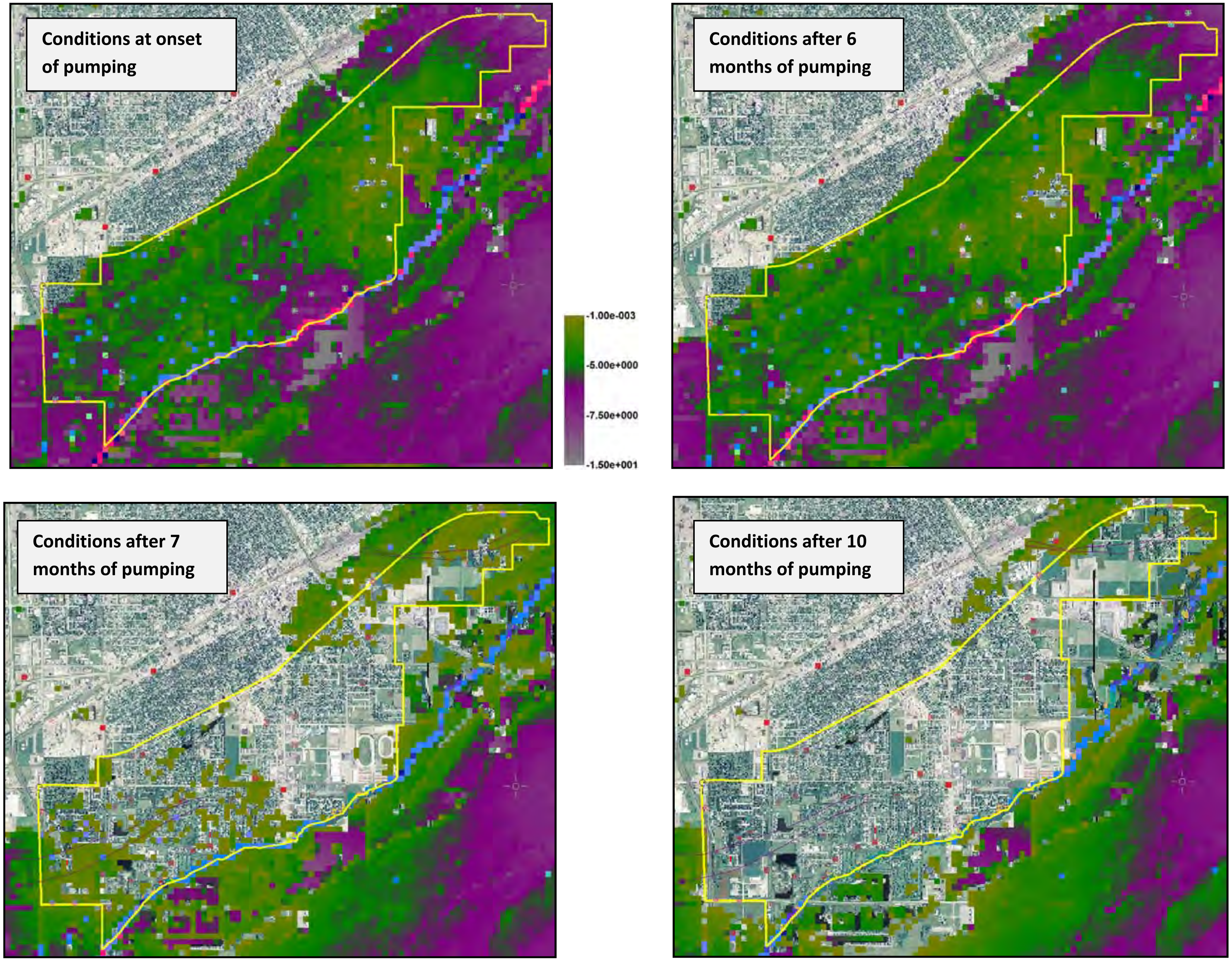


Figure 6.2-4 - SA dewatering over a 10 month period. Negative values on scale (above) indicate level (in feet) water table exceeds critical level (value is negative due t[redacted]

6.3 Capture Zone Analysis

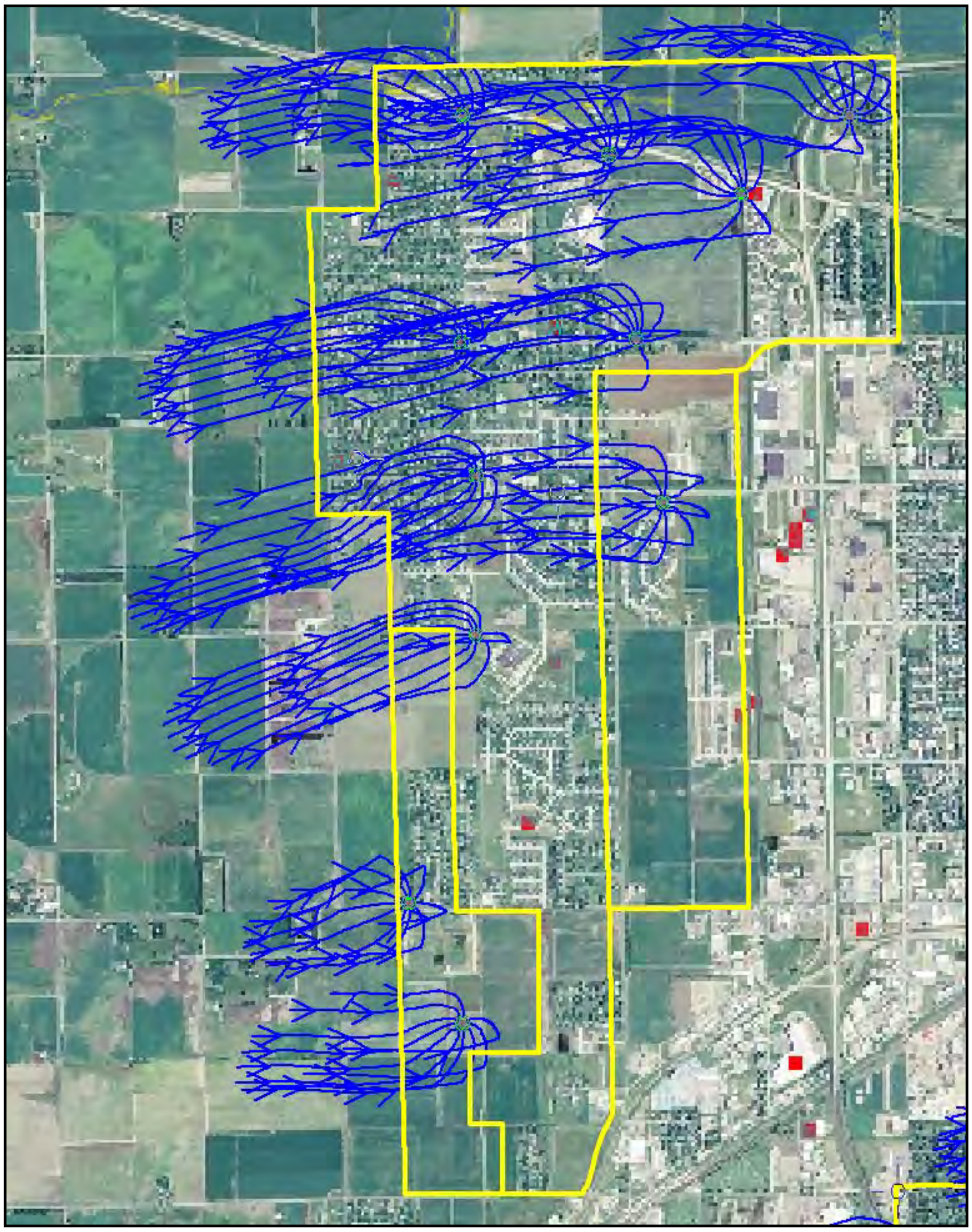
Following completion of a preliminary layout of wells across the NWA and SA, a capture zone analysis was performed to assess the areas of influence (capture zone) for each new dewatering well over the transient model period. The overall impetus for this test was to ensure that no capture zones for dewatering wells contain or have particles that terminate in either of the Parkview or VCP plumes that are within the SA. Additionally, capture zone analysis for the NWA wells provides an exploration of how hydraulic conductivity heterogeneities vary across the model domain and influence the characteristics of each well's capture zone and to ensure no boundary influences occur on the wells in the NWA.

MODPATH (Pollock, 1989) was utilized to perform the capture zone analysis. MODPATH is a particle tracking code that couples with MF2K and communication between the two model applications is enhanced by the GUI (Groundwater Vistas) used in this project. The code assigns "particles" in the model grid cell containing the pumping well and tracks them backwards in time for the length of the transient period in the model, until the particles either encounter a model boundary or cease at the end of the 12 year time period. The code uses velocities and gradients from the transient MF2K output to direct the track of particles over time. Inputs required from the user include porosity and the elevation within the model cell to assign the particles that corresponds to elevation of the well screen. A porosity value of 0.3 was assigned over the entire model domain (Fetter, 1994), and the particles were assigned to an elevation corresponding to the bottom 40 feet of the cell containing each pumping well.

The 12-year capture zones for the NWA wells vary in length from 0.75 miles to about 1.5 miles in length and are oriented slightly northeastward (fig. 6.3-1). Capture zones tend to extend further westward in the northern half of the NWA, which indicative of the higher hydraulic conductivities in this area of the model domain. No capture zone particles approach the west model boundary, which indicates that no little to no boundary influences occur during the dewatering analysis. Particle tracks overlap for several of the wells in the northern half of the NWA, which indicates that well-to-well interference, which indicates potential for further drawdown than what a single well can induce can occur.

Figure 6.3-2 displays the capture zones for each of the dewatering wells in the SA. Although the well locations in this figure display the final locations for dewatering wells in this area, it should be noted that determination of the final well locations in the SA was a trial-and-error process that involved testing well capture zones locations in relation to the contaminant plumes in the SA along with guidance from the OA engineering design group regarding the most appropriate locations for sighting wells, pipeline, and discharge areas within the city. Development of the final well configuration in the SA required significantly more time and testing than the final well field design in the NWA. Like the NWA, the capture zone sizes vary across the well field area, but unlike the NWA, the orientations of the SA capture zones show more variability in shape, length and orientation. In addition to variability in aquifer permeability within the SA boundary, a significantly greater number of high capacity wells operate in this area and induce local changes in gradient which could influence the orientation of the dewatering well

Figure 6.3-1 - 12-year capture zones for dewatering wells in the NWA.

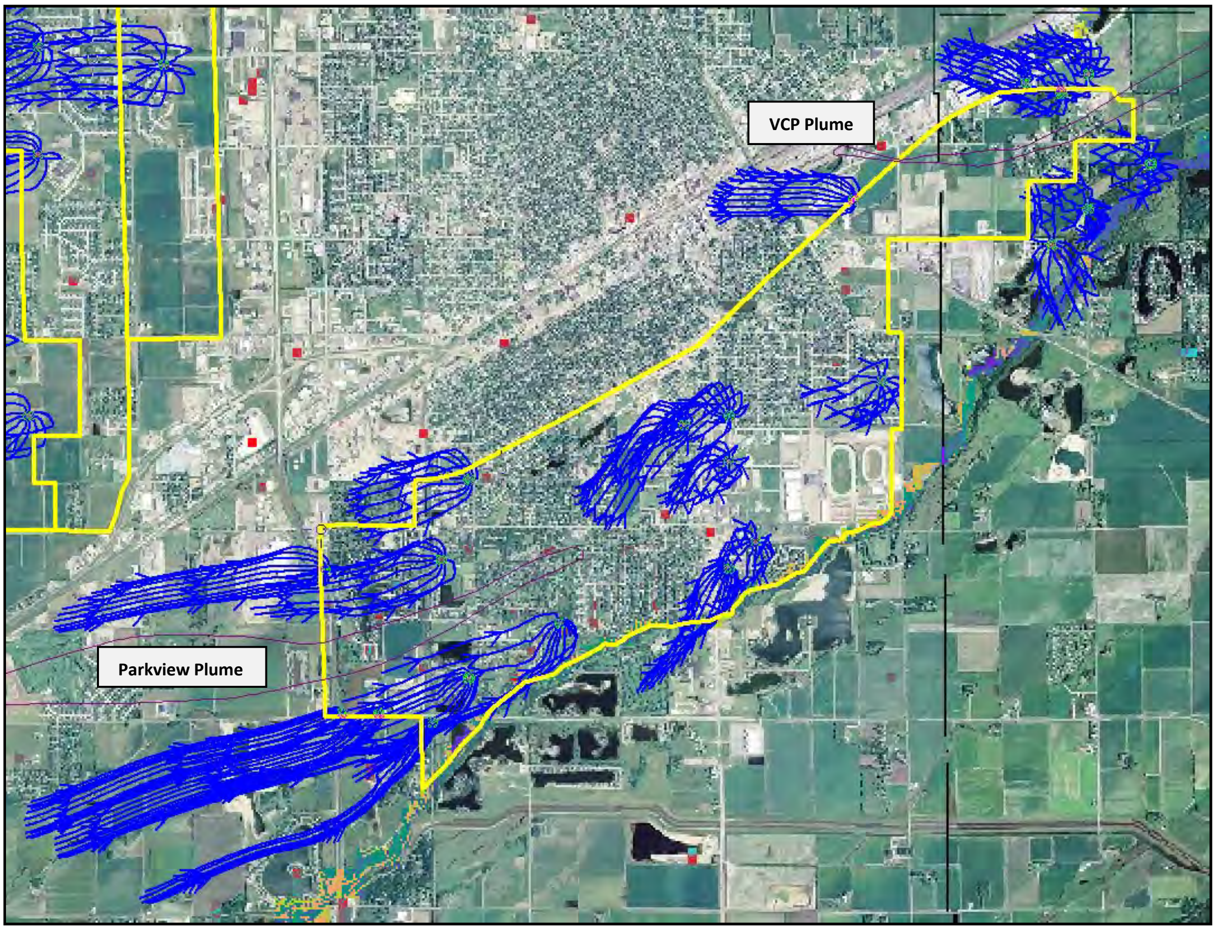


capture zones. In general, capture zones for wells on the west side of the SA are longer than the rest of the well field, with some particle traces approaching 2 miles. The greatest amount of particle overlap occurs in the area of the west wells, thus indicating that greater potential for well-to-well interference (and more drawdown) could occur. Capture zones for wells in the center and northeast side of the SA tend to be shorter in length and have more variability in the orientation of the particle traces.

Figure 6.3-2 displays the mapped boundaries of the Parkview and VCP plumes. No particle traces cross or terminate in these plume areas over a 12 year period. Dewatering wells on the south side of the northeast plume have particles that terminate near the south plume boundary.

It is possible that in the future, contribution of water to these dewatering wells could occur from within the plume area. However, it is assumed that over time, remediation efforts for this plume will reduce contaminant concentrations below regulatory limits and thus reduce and eventually eliminate the question of the capture zone areas for the dewatering wells on the northeast fringes of the SA.

Figure 6.3-2 - 12-year capture zones for dewatering wells in the SA. Note the location of the contaminant plume boundaries (magenta lines) on the west and east sides of the SA.



6.4 Transport Model Assessment

A contaminant transport analysis was conducted as a final test of the potential influence that dewatering wells in the SA could induce on the Parkview and VCP plumes. Although particles from each dewatering wells capture zones do not cross the mapped plume areas, gradients induced from pumping can still influence lateral migration of contaminants. The goal of this test was to use hypothetical plumes located in the areas of the mapped plumes to assess if latitudinal and/or longitudinal changes occur from dewatering well operations over the time period tested for dewatering the water table below the critical level in the SA. This exercise was not an attempt to explicitly recreate the west and east plumes in terms of exact shapes, concentrations, and historic migration pathway. OA utilized the transport code MT3DS (Zheng

& Wang, 1999) to conduct this analysis. This code interfaces with MF2K under the control of Groundwater Vistas.

Like MODPATH, MT3DS uses flow velocities and gradient from MF2K to compute contaminant movement in the saturated portion of the flow system. MT3DS required more parameter inputs than MODPATH, as well as trial-and-error adjustments in input parameters with the goal of approximating the shape of the mapped contaminant plumes with hypothetical simulated plumes. This exercise required adjustment of a parameter called hydrodynamic dispersion, or simply dispersion. This term describes the movement of water at varying velocities through pore spaces and the diffusion of solutes within the groundwater and is expressed by:

$$D_L = a_L * v_x + D^* \qquad \text{Eqn. 1 (Fetter 1994)}$$

D_L = hydrodynamic dispersion (*L indicating longitudinal direction, the same equation applies for both transverse and vertical flow*), unitless

a_L = dynamic dispersivity (ft), from equation 0.0175L1.46, with L = flow path length (Neuman, 1990)

v_x = average linear groundwater velocity (ft/d)

D* = molecular diffusion

Initial concentrations similar to the those measured in a transect of monitoring wells near where the Parkview plume enters the SA were set in grid cells along the west border of the SA instead of the original source of the plume. This approach reduced the amount of calibration time needed to simulate a test plume and was considered valid since information on concentrations were available in close proximity to the SA boundary. The initial calculation of dispersion using estimated inputs for velocity, diffusion, and dispersivity, yielded a value of 6,709. This term was eventually adjusted with an increase by a factor of about 3.5x to obtain a similar shape and area as the west plume. Transverse dispersion was set at 0 through trial and error calibration of this term. Based on the nature of the channel-derived alluvial deposits within the aquifer in this area, it is expected that longitudinal dispersion would dominate the transport and plume characteristics, a condition that is further supported by the capture zones for the dewatering wells in this area of the SA.

The simulated Parkview plume boundary was defined at a concentration of 0.5 µg/l, the same concentration at the boundary of the mapped plumes. Figure 6.4-1 shows the pattern of influence the dewatering wells near the west plume could potentially induce in this area. Three contours are displayed in this figure- the Parkview plume area, the simulated test plume simulated without insertion of new dewatering wells, and potential plume shape following 1.7 years of dewatering well extraction. In this test scenario, the hypothetical plume (at a concentration of 0.05 µg/l) widens by about 600 ft on the north and 875 ft on the south side of the plume. The test plume shortens however, over this time period, which would be expected by further dispersion and diffusion induced from the pumping wells. Note that this scenario assumes a constant concentration at the simulated plume source and does not account for decreasing concentrations from remediation efforts or natural attenuation. It is assumed that over time, remediation actions in this area will negate the concern over lateral migration of contaminants at levels above NDEQ regulatory limits, and that the SA dewatering wells in this area will not create long-term disturbance of the Parkview plume to a point of adversely affecting public water supplies in Grand Island. If lateral dispersion of contaminants persist, interceptor wells could be used on the north and south sides of the Parkview plume to capture any laterally migrating contaminant induced by the dewatering wells.

Figure 6.4-1 - Results of a hypothetical contaminant transport analysis at the Parkview plume in the SA. Displayed are the mapped Parkview plume extent from the EPA (magenta), the simulated test plume pre-dewatering well activity (orange), and plume extent following extraction well pumping (blue). Dewatering well locations shown by yellow triangles.

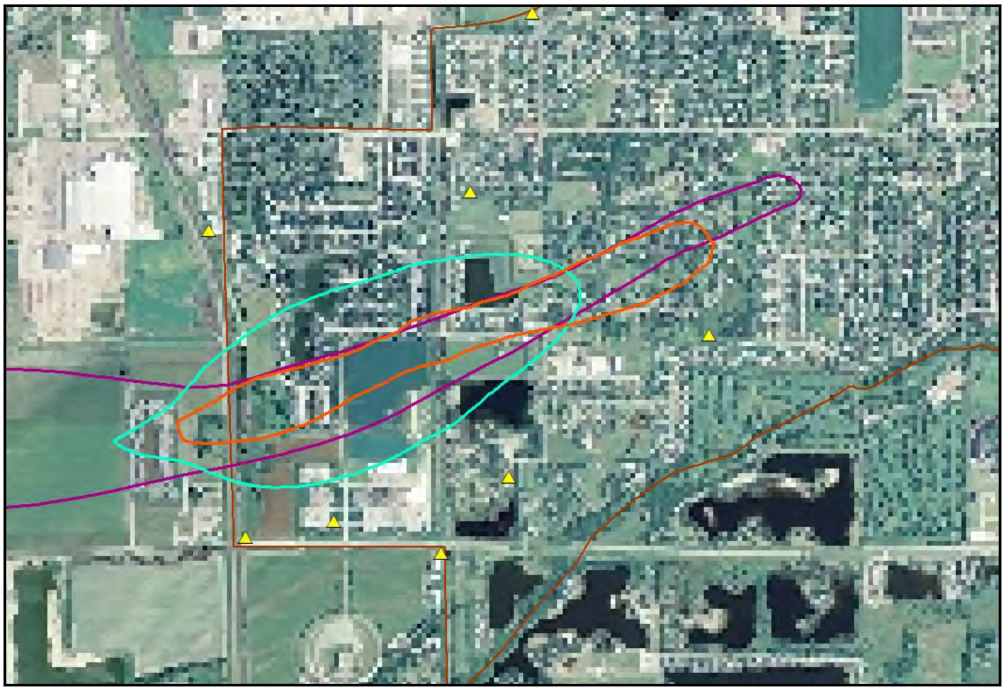
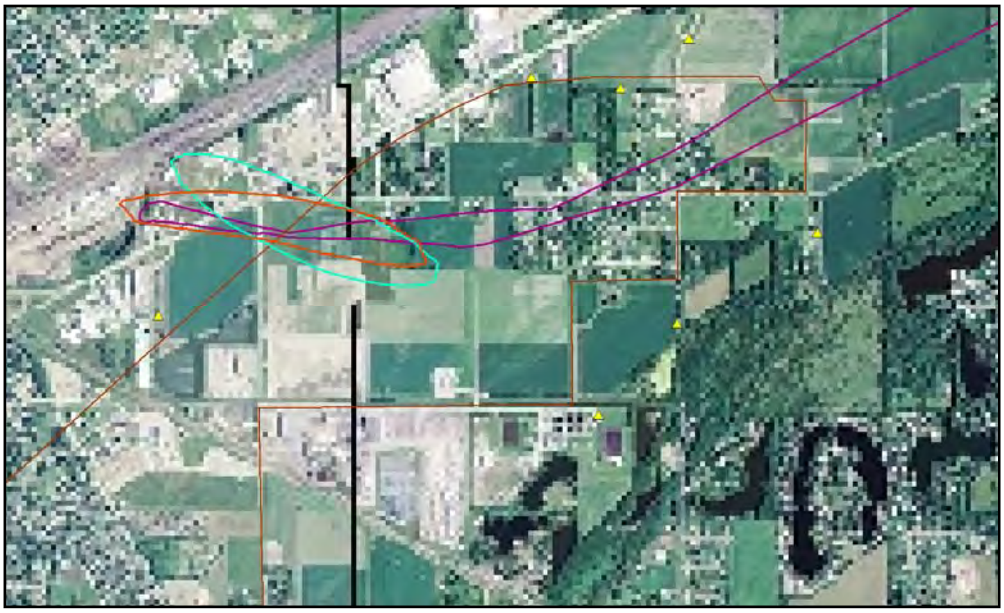


Figure 6.4-2 displays the results of the hypothetical test near the VCP plume. The hypothetical simulation for this area of the model was similar to that at the Parkview plume, but with an initial concentration of 1.0 ug/l defined at the plume source. Due to grid cell resolution and lack of detailed information on locale permeabilities, it was not possible to create a plume shape that resembled the entire length of the VCP plume (the mapped plume width was considerably narrower than the smallest cell in the model grid). Because of this condition, an attempt was made to simulate a hypothetical plume only in the VCP plume source area. The model indicates that over the dewatering period for the SA, the simulated plume (at a concentration contour of 0.5 ug/l) shifts southward by approximately 250 feet, but does not elongate or come within close proximity of public supply wells or other dewatering wells. It should be expected that the simulated plume in this area would be less influenced by pumping in relation to the Parkview area considering the greater spacing and few number of dewatering wells in the northeast extent of the SA. It is possible however that the extremely narrow portion of the plume not simulated in this test could widen from influence of the wells on the north and south sides of the VCP plume. Since the currently mapped plume is extremely narrow, even

Figure 6.4-2 - Results of a hypothetical contaminant transport analysis at the VCP plume in the SA. Displayed are the mapped VCP plume extent from NDEQ (magenta), the simulated test plume pre-dewatering well activity (orange), and plume extent following extraction well pumping (blue). Dewatering well locations shown by yellow triangles.



under the influence of some municipal and industrial wells currently operating in the area, it is unlikely that the addition of the wells that surround this plume exert significant influence on this plume. The capture zone shapes indicate that the wells on the south side of the northeast plume would more likely interfere with the VCP plume in this area as the capture zones for the north side wells are close to perpendicular and away from the VCP plume.

6.5 Water Table Recovery

A final analysis was conducted to assess the recovery trends of the water table across both the NWA and SA if all wells were shut off following water table dewatering below the critical level in each area. This test revealed that between 1.5 to 2 months, the water table begins to approach and exceed the critical level in isolated areas of both the NWA and SA, primarily along the south border and northeast portion of the SA and in the southwest portion of the NWA. Testing revealed that within one month of resuming pumping at all wells, the water table again dropped below the critical level over both areas. These conditions are assuming water table conditions as observed in the 2009-2011 timeframe, which based on area hydrographs was similar to water levels observed in the late 1990s, under normal or above normal climatic conditions. In extended wet periods, the wells will likely need to be run continuously to maintain water table levels below the critical surface. However, during periods of below average precipitation, the wellfields will be able to remain idle over longer periods or run an alternating schedules where not all wells operate simultaneously.

7.0 CONCLUSIONS AND RECOMMENDATIONS

This report covers the development, calibration and utilization of a groundwater model to investigate dewatering options of the Quaternary aquifer underlying Grand Island, NE. The process of this project for model development involved, in order, 1) creation and calibration of a steady state model to water levels observed in the late 1990s, 2) calibration of water level changes and recharge in a transient model that represents the period of 1999 to 2011, 3) design of wellfields for two areas of Grand Island with a criteria of using 15 ft below ground level as a dewatered threshold, including assessment of recovery time 4) evaluation of capture zone areas for each well installed, and 5) test hypothetical contaminant plumes to further assess the influence of the dewatering wellfield designs.

7.1 Key Findings

- Dewatering of the northwest area of concern (NWA) required 11 dewatering wells, each pumping at 500 gpm for a duration of 6.5 to 7 months.

- Dewatering of the south area (SA) of concern requires 22 wells with pumping rates ranging from 400 to 500 gpm. Most of the south area dewaters within at about the 7 month timeframe, but near complete dewatering takes between 9 and 10 months.

- 12-year capture zones for the NWA are typically 0.75 to 1.5 miles in length and are oriented slight southwest to the northeast and do not approach the west model boundary.

- 12-year capture zones in the SA have more variability in shape, length, and orientation. The final design of the wellfield layout was determined after trial and error model runs to ensure no dewatering well capture zones cross the GCA of VCP plume in the SA.

- A hypothetical contaminant transport analysis revealed that the dewatering wells in the SA have the potential influence the contaminant plumes, with greater potential of influence on the GCA plume on the west than the VCP plume in the northeast part of the SA. These conclusions are based however with a simulated constant source contaminant concentration and no natural attenuation. The model demonstrates that to dewater the entire SA, wells will need to be installed in close proximity to the contaminant plumes and gradients induced from pumping these wells could cause lateral migration of contaminants.

- Water table recovery, under normal climatic conditions, approaches the critical surface after 1.5 to 2 months following initial dewatering of both the SA and NWA wellfields. Resumption of pumping following this recovery in both wellfields reduces the water table below the critical level within one month.

7.2 Recommendations

- Because potential exists for disturbance of the GCA and VCP plumes by the dewatering wells in the SA, it is recommended that the city communicates with the EPA and NDEQ regarding the current status of each contaminant plume in the SA and the projected timeframes for cleanup of each plume and discuss possibilities such as interceptor wells between the dewatering wells and the plumes if contaminant is observed to migrate laterally from each plume following initiation of dewatering operations.

- If further analysis is required to assess potential impacts to the GCA and VCP plumes, the contaminant transport models for these areas that were developed intentionally for fate and transport assessment should be used.

- Under normal climatic conditions and achievement of complete dewatering across the NWA and SA, pumping durations across the wellfields will be able to maintain dewatered conditions with a cyclical pattern of pumping, such as 1 month with wells in operation followed by one to two months of no pumping. Changing climatic conditions will dictate modification of this type of schedule with wet periods requiring constant pumping and dry periods requiring less pumping and/or longer recovery periods with wells that remain idle.

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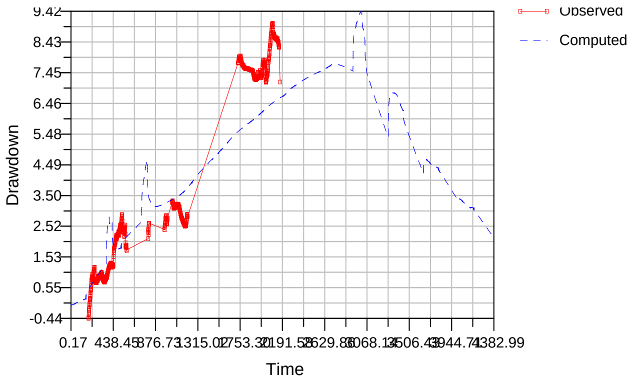
APPENDIX A

Additional calibration hydrographs for the transient model.

G-100378A



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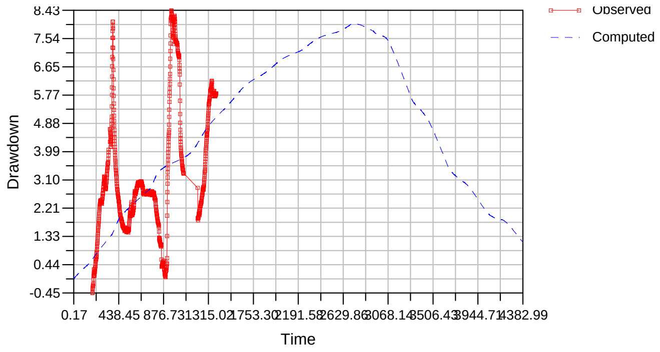

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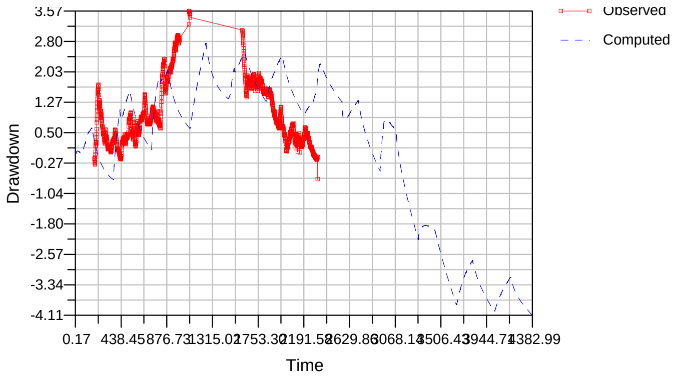

(see figure 5.1-1 for locations in model domain)

APPENDIX B

Detailed Layout of the Updated Dewatering System

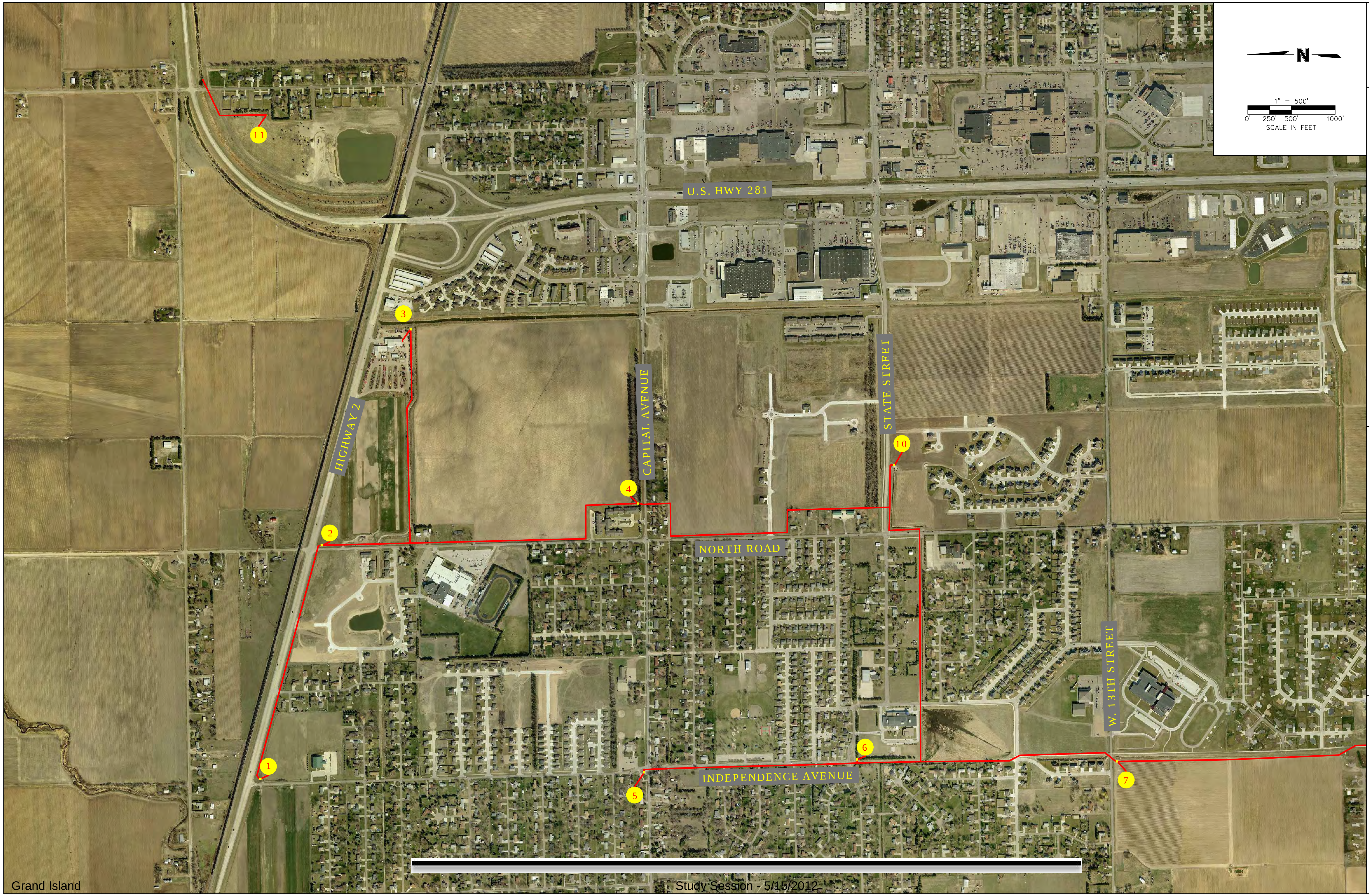

EXHIBIT 1

201 East 2nd Street
P.O. Box 1072
Grand Island, NE 68802-1072
TEL 308.384.8750
FAX 308.384.8752

OLSSON® ASSOCIATES

GRAND ISLAND DEWATERING
COLLECTION SYSTEM LAYOUT

PROJECT NO: 011-2231
DRAWN BY: EJS
DATE: 05/07/12

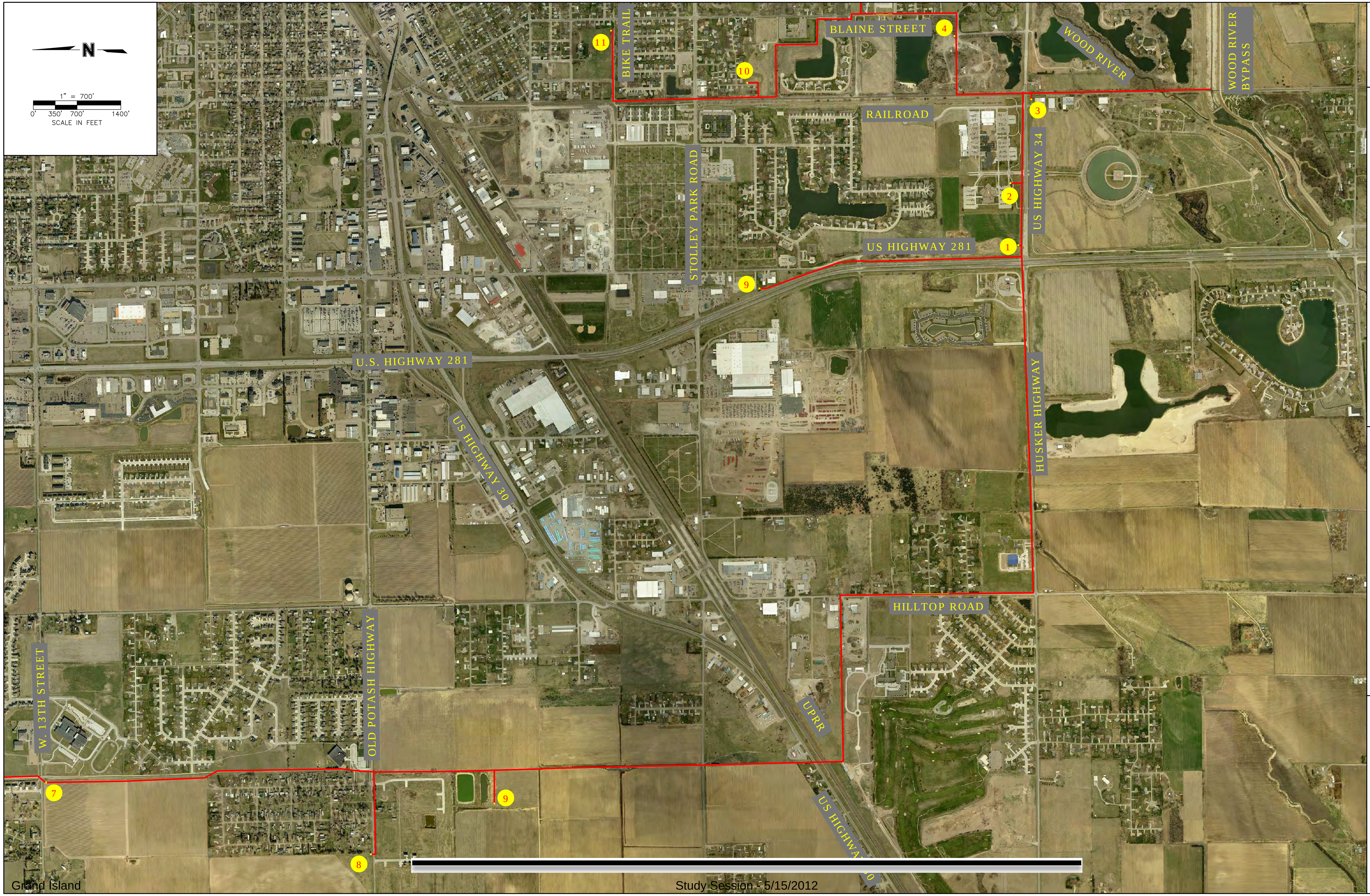

EXHIBIT 2

201 East 2nd Street
P.O. Box 1072
Grand Island, NE 68802-1072
TEL 308.384.8750
FAX 308.384.8752

OLSSON ASSOCIATES

GRAND ISLAND DEWATERING COLLECTION SYSTEM LAYOUT

PROJECT NO: 011-2231
DRAWN BY: EJS
DATE: 05/07/12

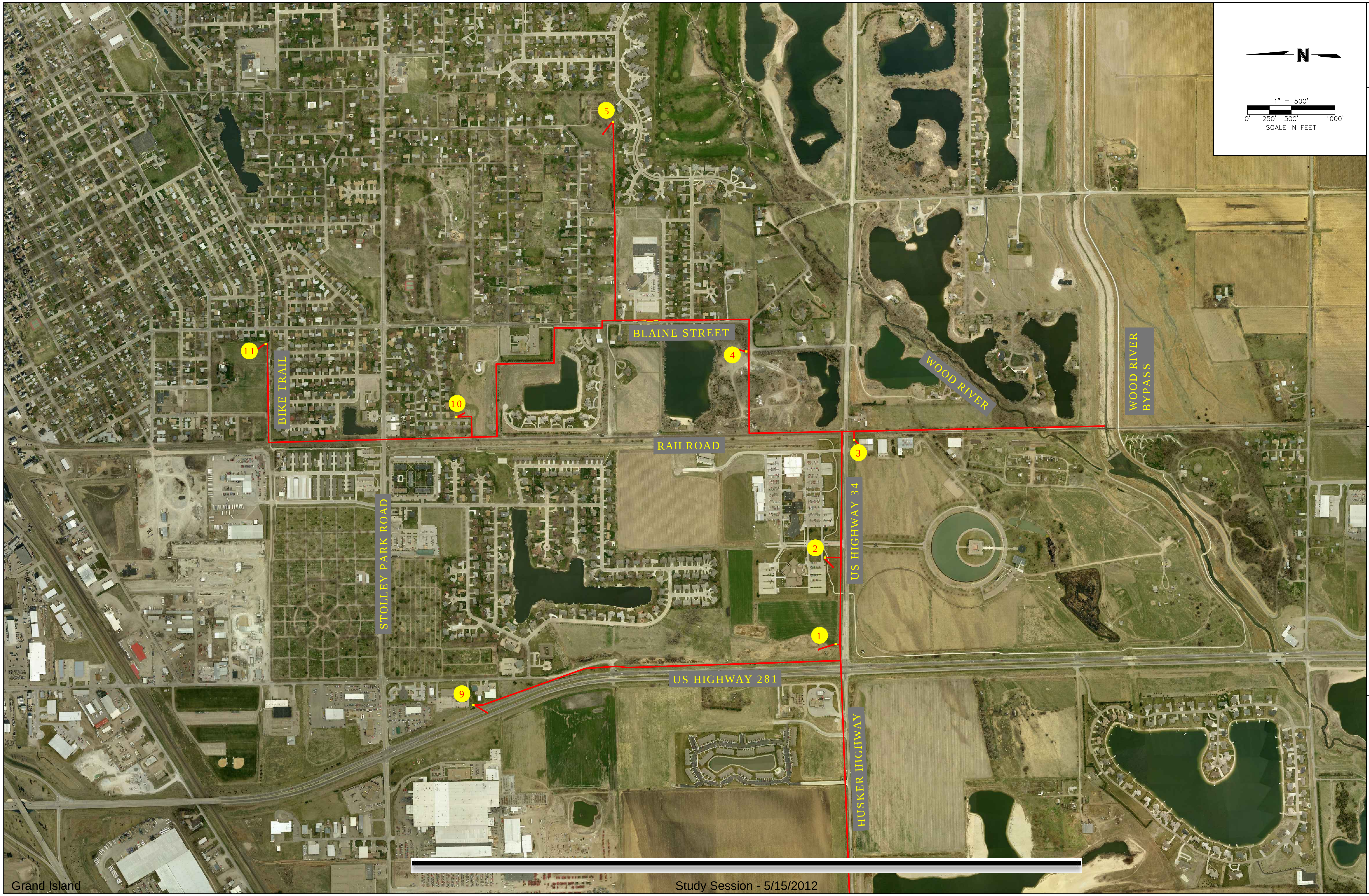

EXHIBIT 3

OLSSON ASSOCIATES

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FAX 308.384.8752

GRAND ISLAND DEWATERING
COLLECTION SYSTEM LAYOUT

PROJECT NO: 011-2231
DRAWN BY: EJS
DATE: 05/07/12

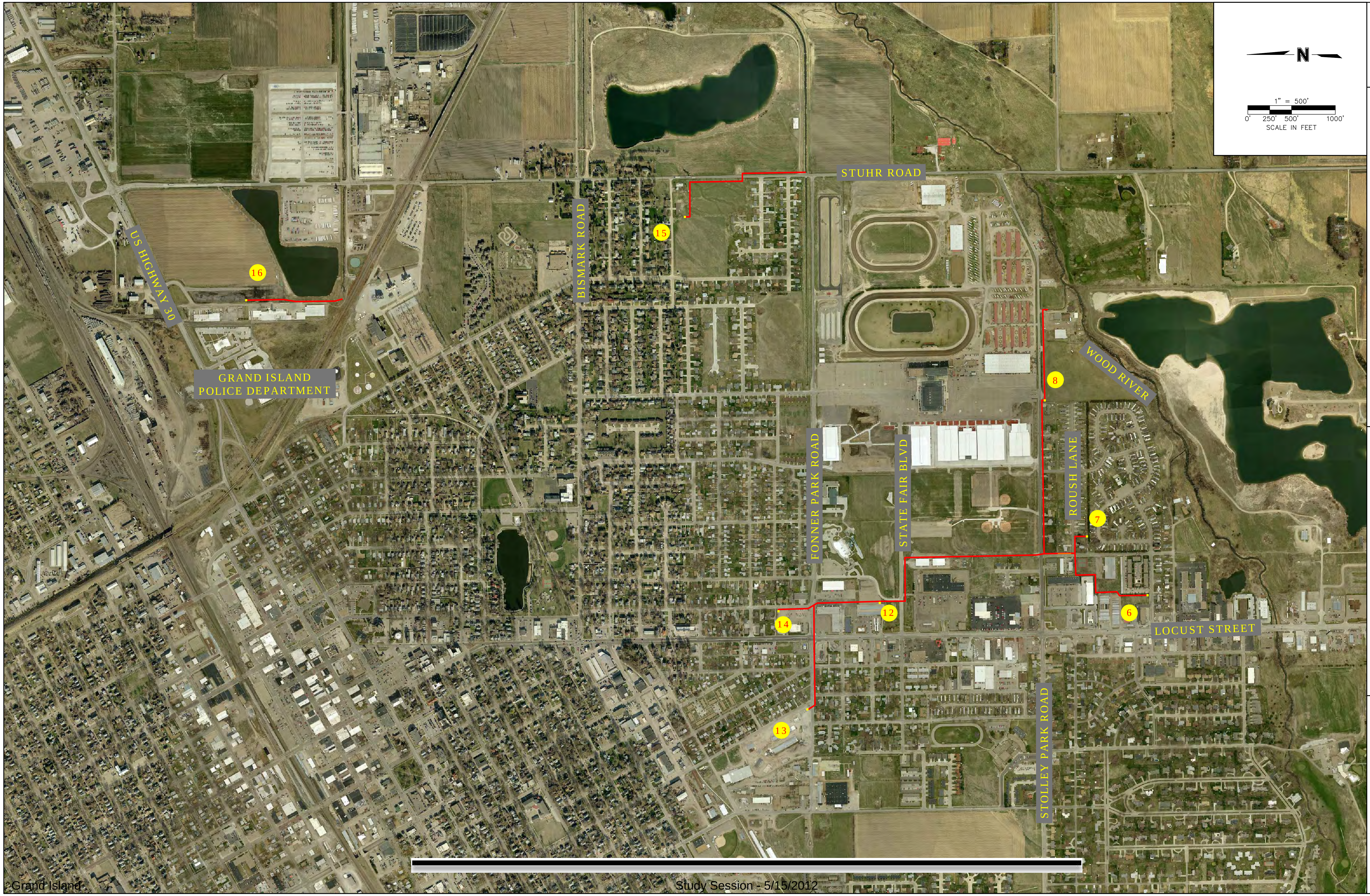

EXHIBIT 4

OLSSON ASSOCIATES

201 East 2nd Street
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Grand Island, NE 68802-1072
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FAX 308.384.8752

GRAND ISLAND DEWATERING
COLLECTION SYSTEM LAYOUT

PROJECT NO: 011-2231
DRAWN BY: EJS
DATE: 05/07/12

N
1" = 250'
0' 125' 250' 500'
SCALE IN FEET
EXHIBIT
5
201 East 2nd Street
P.O. Box 1072
Grand Island, NE 68802-1072
TEL 308.384.8750
FAX 308.384.8752
OLSSON® ASSOCIATES
GRAND ISLAND DEWATERING
COLLECTION SYSTEM LAYOUT
PROJECT NO: 011-2231
DRAWN BY: EJS
DATE: 05/07/12
WOOD RIVER
SHADY BEND ROAD
US HIGHWAY 30
SEEDLING MILE ROAD
SWIFT ROAD
17
18
19
20
21
22

APPENDIX C

Detailed Breakdown of Probable Opinion of Cost

PHASE 1				
Item	Estimated Quantity	Unit	Unit $	Total $
8" PVC Water Main	10560	LF	$ 25.00	$264,000
10" PVC Water Main	725	LF	$ 29.00	$21,025
12" PVC Water Main	900	LF	$ 37.00	$33,300
14" PVC Water Main	2985	LF	$ 42.00	$125,370
16" PVC Water Main	3090	LF	$ 46.00	$142,140
18" PVC Water Main	1170	LF	$ 61.00	$71,370
20" PVC Water Main	6755	LF	$ 75.00	$506,625
24" PVC Water Main	17780	LF	$ 110.00	$1,955,800
30" DI Water Main	2540	LF	$ 140.00	$355,600
36" DI Water Main	2735	LF	$ 188.00	$514,180
Utility Crossing	18	EA	$ 1,720.00	$30,960
Pump System	11	EA	$ 81,650.00	$898,150
Pump Controls	11	EA	$ 3,350.00	$36,850
Decommission Well	3	EA	$ 1,000.00	$3,000
Observation Well	5	EA	$ 7,500.00	$37,500
Observation Well Control System	7	EA	$ 6,550.00	$45,850
Remove Driveway	1867	SY	$ 5.00	$9,333
6" P.C. Concrete Driveway	1867	SY	$ 36.00	$67,200
Gravel Surfacing	294	TON	$ 28.00	$8,227
Remove and Replace 4" P.C. Concrete Walk	2950	SF	$ 4.50	$13,275
Remove and Replace 6" Concrete Bikeway	100	SY	$ 45.00	$4,500
Remove Ashp./Conc. Roadway	1333	SY	$ 11.00	$14,667
Replace Concrete Roadway	1333	SY	$ 65.00	$86,667
6" Concrete Pavement	750	SY	$ 36.00	$27,000
Seeding	38	AC	$ 1,250.00	$46,875
Slip Form Channel Lining	25015	SY	$ 35.00	$875,521
Hand Pour Channel Lining	1944	SY	$ 38.00	$73,889
Steel for Outlet Structure	670	LB	$ 1.50	$1,005
Concrete for Outlet Structure	10	CY	$ 600.00	$6,000
Dewatering - 8" Water Main	9240	LF	$ 15.00	$138,600
Dewatering - 10" Water Main	725	LF	$ 15.00	$10,875
Dewatering - 12" Water Main	1000	LF	$ 15.00	$15,000
Dewatering - 14" Water Main	2985	LF	$ 20.00	$59,700
Dewatering - 16" Water Main	3490	LF	$ 20.00	$69,800
Dewatering - 18" Water Main	3270	LF	$ 20.00	$65,400
Dewatering - 20" Water Main	2285	LF	$ 20.00	$45,700
Dewatering - 24" Water Main	8360	LF	$ 20.00	$167,200
Dewatering - 30" Water Main	2620	LF	$ 20.00	$52,400
Dewatering - 36" Water Main	3005	LF	$ 20.00	$60,100
Directional Drilling - 8" Water Main	595	LF	$ 90.00	$53,550
Directional Drilling - 10" Water Main	0	LF	$ 110.00	$0
Directional Drilling - 12" Water Main	100	LF	$ 120.00	$12,000
Directional Drilling - 14" Water Main	0	LF	$ 155.00	$0
Directional Drilling - 16" Water Main	400	LF	$ 175.00	$70,000
Directional Drilling - 18" Water Main	2100	LF	$ 200.00	$420,000
Directional Drilling - 20" Water Main	795	LF	$ 242.00	$192,390
Directional Drilling - 24" Water Main	870	LF	$ 300.00	$261,000
Directional Drilling - 30" Water Main	80	LF	$ 400.00	$32,000
Directional Drilling - 36" Water Main	270	LF	$ 500.00	$135,000
			TOTAL	$ 8,136,593.83

PHASE 2				
Item	Estimated Quantity	Unit	Unit $	Total $
8" PVC Water Main	16045	LF	$ 25.00	$401,125
10" PVC Water Main	2775	LF	$ 29.00	$80,475
12" PVC Water Main	2260	LF	$ 37.00	$83,620
14" PVC Water Main	3995	LF	$ 42.00	$167,790
16" PVC Water Main	0	LF	$ 46.00	$0
18" PVC Water Main	2765	LF	$ 61.00	$168,665
20" PVC Water Main	0	LF	$ 75.00	$0
24" PVC Water Main	0	LF	$ 110.00	$0
30" DI Water Main	0	LF	$ 140.00	$0
36" DI Water Main	0	LF	$ 188.00	$0
Utility Crossing	16	EA	$ 1,720.00	$27,520
Pump System	16	EA	$ 81,650.00	$1,306,400
Pump Controls	16	EA	$ 3,350.00	$53,600
Decommission Well	3	EA	$ 1,000.00	$3,000
Observation Well	5	EA	$ 7,500.00	$37,500
Observation Well Control System	7	EA	$ 6,550.00	$45,850
Remove Driveway	1150	SY	$ 5.00	$5,750
6" P.C. Concrete Driveway	1150	SY	$ 36.00	$41,400
Gravel Surfacing	581	TON	$ 28.00	$16,281
Remove and Replace 4" P.C. Concrete Walk	2950	SF	$ 4.50	$13,275
Remove and Replace 6" Concrete Bikeway	67	SY	$ 45.00	$3,000
Remove Ashp./Conc. Roadway	1200	SY	$ 11.00	$13,200
Replace Concrete Roadway	1200	SY	$ 65.00	$78,000
6" Concrete Pavement	1133	SY	$ 36.00	$40,800
Seeding	21	AC	$ 1,250.00	$25,986
Slip Form Channel Liner	0	SY	$ 35.00	$0
Hand Pour Channel Liner	0	SY	$ 38.00	$0
Steel for Outlet Structure	0	LB	$ 1.50	$0
Concrete for Outlet Structure	0	CY	$ 600.00	$0
Dewatering - 8" Water Main	8785	LF	$ 15.00	$131,775
Dewatering - 10" Water Main	2960	LF	$ 15.00	$44,400
Dewatering - 12" Water Main	383	LF	$ 15.00	$5,738
Dewatering - 14" Water Main	0	LF	$ 20.00	$0
Dewatering - 16" Water Main	0	LF	$ 20.00	$0
Dewatering - 18" Water Main	1085	LF	$ 20.00	$21,700
Dewatering - 20" Water Main	0	LF	$ 20.00	$0
Dewatering - 24" Water Main	0	LF	$ 20.00	$0
Dewatering - 30" Water Main	0	LF	$ 20.00	$0
Dewatering - 36" Water Main	0	LF	$ 20.00	$0
Directional Drilling - 8" Water Main	915	LF	$ 90.00	$82,350
Directional Drilling - 10" Water Main	185	LF	$ 110.00	$20,350
Directional Drilling - 12" Water Main	765	LF	$ 120.00	$91,800
Directional Drilling - 14" Water Main	380	LF	$ 155.00	$58,900
Directional Drilling - 16" Water Main	0	LF	$ 175.00	$0
Directional Drilling - 18" Water Main	100	LF	$ 200.00	$20,000
Directional Drilling - 20" Water Main	0	LF	$ 242.00	$0
Directional Drilling - 24" Water Main	0	LF	$ 300.00	$0
Directional Drilling - 30" Water Main	0	LF	$ 400.00	$0
Directional Drilling - 36" Water Main	0	LF	$ 500.00	
			TOTAL	$ 3,090,249.69

PHASE 3				
Item	Estimated Quantity	Unit	Unit $	Total $
8" PVC Water Main	2705	LF	$ 25.00	$67,625
10" PVC Water Main	735	LF	$ 29.00	$21,315
12" PVC Water Main	0	LF	$ 37.00	$0
14" PVC Water Main	3935	LF	$ 42.00	$165,270
16" PVC Water Main	0	LF	$ 46.00	$0
18" PVC Water Main	0	LF	$ 61.00	$0
20" PVC Water Main	0	LF	$ 75.00	$0
24" PVC Water Main	0	LF	$ 110.00	$0
30" DI Water Main	0	LF	$ 140.00	$0
36" DI Water Main	0	LF	$ 188.00	$0
Utility Crossing	2	EA	$ 1,720.00	$3,440
Pump System	6	EA	$ 81,650.00	$489,900
Pump Controls	6	EA	$ 3,350.00	$20,100
Decommission Well	0	EA	$ 1,000.00	$0
Observation Well	1	EA	$ 7,500.00	$7,500
Observation Well Control System	1	EA	$ 6,550.00	$6,550
Remove Driveway	478	SY	$ 5.00	$2,389
6" P.C. Concrete Driveway	478	SY	$ 36.00	$17,200
Gravel Surfacing	37	TON	$ 28.00	$1,037
Remove and Replace 4" P.C. Concrete Walk	0	SF	$ 4.50	$0
Remove and Replace 6" Concrete Bikeway	0	SY	$ 45.00	$0
Remove Ashp./Conc. Roadway	320	SY	$ 11.00	$3,520
Replace Concrete Roadway	320	SY	$ 65.00	$20,800
6" Concrete Pavement	0	SY	$ 36.00	$0
Seeding	5	AC	$ 1,250.00	$6,650
Slip Form Channel Liner	0	SY	$ 35.00	$0
Hand Pour Channel Liner	0	SY	$ 38.00	$0
Steel for Outlet Structure	0	LB	$ 1.50	$0
Concrete for Outlet Structure	0	CY	$ 600.00	$0
Dewatering - 8" Water Main	3005	LF	$ 15.00	$45,075
Dewatering - 10" Water Main	735	LF	$ 15.00	$11,025
Dewatering - 12" Water Main	0	LF	$ 15.00	$0
Dewatering - 14" Water Main	3985	LF	$ 20.00	$79,700
Dewatering - 16" Water Main	0	LF	$ 20.00	$0
Dewatering - 18" Water Main	0	LF	$ 20.00	$0
Dewatering - 20" Water Main	0	LF	$ 20.00	$0
Dewatering - 24" Water Main	0	LF	$ 20.00	$0
Dewatering - 30" Water Main	0	LF	$ 20.00	$0
Dewatering - 36" Water Main	0	LF	$ 20.00	$0
Directional Drilling - 8" Water Main	300	LF	$ 90.00	$27,000
Directional Drilling - 10" Water Main	0	LF	$ 110.00	$0
Directional Drilling - 12" Water Main	0	LF	$ 120.00	$0
Directional Drilling - 14" Water Main	50	LF	$ 155.00	$7,750
Directional Drilling - 16" Water Main	0	LF	$ 175.00	$0
Directional Drilling - 18" Water Main	0	LF	$ 200.00	$0
Directional Drilling - 20" Water Main	0	LF	$ 242.00	$0
Directional Drilling - 24" Water Main	0	LF	$ 300.00	$0
Directional Drilling - 30" Water Main	0	LF	$ 400.00	$0
Directional Drilling - 36" Water Main	0	LF	$ 500.00	$0
			TOTAL	$ 1,003,846.24

Combined Total				
Item	Estimated Quantity	Unit	Unit $	Total $
8" PVC Water Main	29310	LF	$ 25.00	$732,750
10" PVC Water Main	4235	LF	$ 29.00	$122,815
12" PVC Water Main	3160	LF	$ 37.00	$116,920
14" PVC Water Main	10915	LF	$ 42.00	$458,430
16" PVC Water Main	3090	LF	$ 46.00	$142,140
18" PVC Water Main	3935	LF	$ 61.00	$240,035
20" PVC Water Main	6755	LF	$ 75.00	$506,625
24" PVC Water Main	17780	LF	$ 110.00	$1,955,800
30" DI Water Main	2540	LF	$ 140.00	$355,600
36" DI Water Main	2735	LF	$ 188.00	$514,180
Utility Crossing	36	EA	$ 1,720.00	$61,920
Pump System	33	EA	$ 81,650.00	$2,694,450
Pump Controls	33	EA	$ 3,350.00	$110,550
Decommission Well	6	EA	$ 1,000.00	$6,000
Observation Well	11	EA	$ 7,500.00	$82,500
Observation Well Control System	15	EA	$ 6,550.00	$98,250
Remove Driveway	3494	SY	$ 5.00	$17,472
6" P.C. Concrete Driveway	3494	SY	$ 36.00	$125,800
Gravel Surfacing	912	TON	$ 28.00	$25,546
Remove and Replace 4" P.C. Concrete Walk	5900	SF	$ 4.50	$26,550
Remove and Replace 6" Concrete Bikeway	167	SY	$ 45.00	$7,500
Remove Ashp./Conc. Roadway	2853	SY	$ 11.00	$31,387
Replace Concrete Roadway	2853	SY	$ 65.00	$185,467
6" Concrete Pavement	1883	SY	$ 36.00	$67,800
Seeding	64	AC	$ 1,250.00	$79,511
Slip Form Channel Lining	25015	SY	$ 35.00	$875,521
Hand Pour Channel Lining	1944	SY	$ 38.00	$73,889
Steel for Outlet Structure	670	LB	$ 1.50	$1,005
Concrete for Outlet Structure	10	CY	$ 600.00	$6,000
Dewatering - 8" Water Main	21030	LF	$ 15.00	$315,450
Dewatering - 10" Water Main	4420	LF	$ 15.00	$66,300
Dewatering - 12" Water Main	1383	LF	$ 15.00	$20,738
Dewatering - 14" Water Main	6970	LF	$ 20.00	$139,400
Dewatering - 16" Water Main	3490	LF	$ 20.00	$69,800
Dewatering - 18" Water Main	4355	LF	$ 20.00	$87,100
Dewatering - 20" Water Main	2285	LF	$ 20.00	$45,700
Dewatering - 24" Water Main	8360	LF	$ 20.00	$167,200
Dewatering - 30" Water Main	2620	LF	$ 20.00	$52,400
Dewatering - 36" Water Main	3005	LF	$ 20.00	$60,100
Directional Drilling - 8" Water Main	1810	LF	$ 90.00	$162,900
Directional Drilling - 10" Water Main	185	LF	$ 110.00	$20,350
Directional Drilling - 12" Water Main	865	LF	$ 120.00	$103,800
Directional Drilling - 14" Water Main	430	LF	$ 155.00	$66,650
Directional Drilling - 16" Water Main	400	LF	$ 175.00	$70,000
Directional Drilling - 18" Water Main	2200	LF	$ 200.00	$440,000
Directional Drilling - 20" Water Main	795	LF	$ 242.00	$192,390
Directional Drilling - 24" Water Main	870	LF	$ 300.00	$261,000
Directional Drilling - 30" Water Main	80	LF	$ 400.00	$32,000
Directional Drilling - 36" Water Main	270	LF	$ 500.00	$135,000
			TOTAL	$12,230,689.75