GROUNDWATER MODELING STUDY AND WELL HEAD PROTECTION AREA DELINEATION FOR HASTINGS UTILITIES MUNICIPAL WELLFIELD

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Prepared for

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Executive Summary

Layne GeoSciences, Inc. was retained by Hastings Utilities to perform a groundwater modeling study of the aquifer underlying the City of Hastings (Figure 1.1). The project goals were pro-active in nature and specifically defined to:

- Estimate the aquifer safe yield in the area.
- Delineate Well Head Protection Areas (WHPA) for municipal wells in order to ensure appropriate utilization of land in the contributing zone, and to develop a monitoring well program for providing adequate warning of deteriorating water quality.
- Identify suitable locations and spacing of new wells to replace existing wells rendered inoperational due to water quality problems and deteriorating well condition.
- Assess impact if advectuve-dispersive processes on migration of nitrates in groundwater.

A review of literature and well log data indicates that the Pleistocene aquifer, composed of unconsolidated deposits of clay, silt, and gravel, is the principal water bearing formation in the area. It responds primarily as an unconfined system, with saturated thickness varying from 50 to 200 feet in the study area. Depth to the water table varies considerably. In the central regions of the study area, the water table is over 100 feet deep. In the alluvial valleys along the Platte and Little Blue rivers, perched groundwater is less than ten feet below land surface. The general direction of groundwater flow is towards the east-southeast (Figure 2.4). Groundwater levels in the Hastings area have declined by over 25 feet since development. However further west, water levels have increased from predevelopment levels by over 20 feet in the study area (Figure 2.7a and 2.7b). This is attributed to leakage from canals constructed to divert water from the Platte River for irrigation purposes. Although water levels have changed with development, they appear to have stabilized. Hydrographs at monitoring wells in the overall study area indicate that while there is water level fluctuations due to climatic variations, no discernible long-term trend in water levels can be inferred.

Groundwater withdrawals in the area are primarily for agricultural and municipal uses with relatively minor quantities withdrawn for industrial and domestic purposes. Average withdrawal at the Hastings Utilities wellfield is approximately 6,700 acre-feet per year (6 million gallons per day, MGD). This is relatively small quantity compared to groundwater withdrawals for irrigation purposes. Assuming an average groundwater application of 12 inches per year for irrigation purposes, approximately 160,000 acre-feet per year is estimated to be withdrawn in Adams county alone. Groundwater quality appears to have deteriorated with time. Rising nitrate levels in groundwater is of particular concern to Hastings Utilities. Nitrate concentration in the municipal wells have increased from approximately 1 part per million (ppm) in the late sixties and early seventies to approximately 4-5 ppm in recent years.

The project objectives outlined above were accomplished in two distinct and sequential phases. The first phase involved construction of a regional groundwater flow model (Figure 1.1) to

estimate the aquifer safe yield. The second phase of the study involved construction of a relatively local site model in the immediate vicinity of Hastings (Figure 1.1). The smaller area of the Phase II model permitted specification of a finer model grid. This was required to delineate the WHPA precisely and also to assess the impact of mutual well interference for optimal placement of new wells. A particle tracking module was coupled with the groundwater flow model to delineate the WHPAs. This model is referred to as the capture zone model.

The regional flow model developed for Phase I of the study was first calibrated to post-development water levels and then validated to pre-development conditions. The model provides useful information of various hydrologic components in the subsurface. Simulation results indicate that groundwater withdrawals for municipal and irrigation purposes have been supplanted with increased recharge from the Platte River, infiltration from the Phelps Canal, and irrigation return flow (Table 3.2). Development scenarios simulated by the model suggest that additional withdrawals to meet anticipated municipal demands can be supported by the aquifer. An additional municipal withdrawal of 5 MGD was estimated to induce a drawdown of approximately 10 feet from existing levels. Similarly, a 10 percent increase in irrigation withdrawal along with 5 MGD increase in municipal pumpage is expected to cause approximately 16 feet of additional drawdown from existing levels.

The capture zone model developed for Phase II of the study was utilized for delineating wellhead protection areas. Five, ten, and twenty year capture zones were delineated for each municipal well (Figure 4.3-4.6). In general, the 20 year time-of-travel zone extends approximately 5 miles upstream in the northwest direction towards the Platte River. The delineated zones will enable Hastings Utilities to monitor groundwater quality, thereby providing early warning of encroaching contamination. WHPA delineations also provides a basis for conducting a contamination source inventory and ensuring proper land use within the delineated zones. Since the capture zone model does not extend till the Platte River, it was not possible to calculate the groundwater travel time between the municipal wellfield and the Platte River. However, from the potentiometric surface of the aquifer, it is estimated that the travel time is approximately 50-75 years. This implies that it takes approximately 50-75 years for groundwater to travel from the Platte River to the Hastings Utilities wellfield.

The groundwater flow component of the capture zone model was also utilized for evaluating the impact of withdrawals from proposed and existing wells in northwest Hastings. This region is being considered for wellfield development since groundwater in this area appears to be of better quality than in other areas in the vicinity of Hastings. Simulation results indicate that withdrawals form a cluster of wells in the northwest to meet the entire municipal demand is feasible with well spacing of between 1,000 to 1,500 feet.

Finally, hypothetical simulations were conducted with an advection-dispersion model in order to assess the importance of these processes on migration of nitrates in the subsurface. Simulation results indicate that given adequate data for calibration, it is possible to construct a fate and transport model to obtain estimates of future nitrate levels at the municipal wellfield. Such a model could be utilized for evaluating the impacts of better agricultural management practices,

identifying the relative impact of point and non-point sources of contamination in the area, track the migration of contaminant plume developing at storage facilities due to improper handling of contamination sources, and various other scenarios of interest.

Section 1

Introduction

1.1 Study Objectives

Layne GeoSciences, Inc. (LGI) was retained by Hastings Utilities (HU) to perform a groundwater modeling study of the aquifer underlying the City of Hastings (Fig. 1.1). The project goals were pro-active in nature. Although the aquifer currently does not appear to be mined, significant water level declines have occurred in the past. HU is therefore interested in determining the capability of the aquifer to support present and future municipal water demand. Additionally, the land surrounding Hastings is under heavy agricultural cultivation. Therefore, the impact of agriculture on the groundwater quality is of concern to HU. The objectives of the modeling study were therefore specifically defined to:

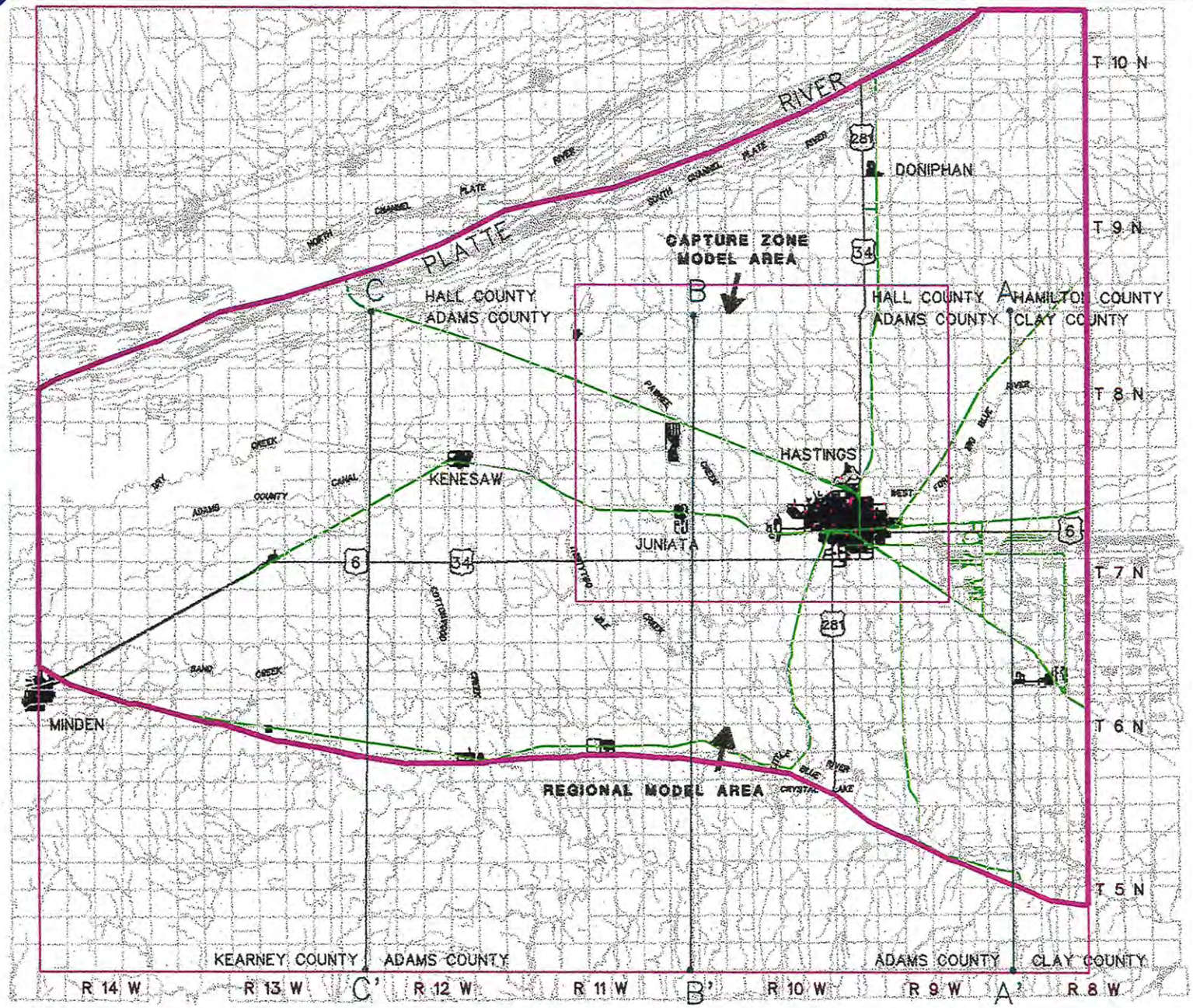
- Estimate the aquifer safe yield in the area.
  Delineate Well Head Protection Areas (WHPA) for municipal wells in order to ensure appropriate utilization of land in the contributing zone, and to develop a monitoring well program for providing adequate warning of deteriorating water quality.
- Identify suitable locations and spacing of new wells to replace existing wells rendered inoperational due to water quality problems and deteriorating well condition.
- Assess impact of advective-dispersive processes on migration of nitrates in groundwater.

1.2 Technical Approach

LGI approached the study in two distinct and sequential phases:

Phase 1 involved the construction of a regional groundwater flow model to estimate the aquifer safe yield. The extent of the model area is presented in Figure 1.1. The majority of the groundwater withdrawal in the area is for irrigation purposes and the subsurface hydrology is affected by flow in the Platte River and diversion of water from the Platte River to the west of Hastings. Because of these factors, a relatively large area was modeled for estimating the aquifer safe yield.

Phase 2 of the study involved construction of a relatively local site model in the immediate vicinity of Hastings. The extent of the model domain is presented in Figure 1.1. The smaller area of the Phase 2 model permitted specification of a finer model grid. This was required to delineate the WHPA precisely and also to assess the impact of mutual well interference for optimal placement of new wells. A particle tracking module was coupled with the groundwater flow model to delineate the WHPAs. Finally, a representative transport model was developed to assess the need for simulating advective-dispersive processes for estimating future nitrate levels in the municipal wells.



Scale = 1:250,000

Model Study Area

FIGURE: 1.1

Layne GeoSciences, Inc.

1.3 Project Scope of Work

The scope of work included a detailed review of existing reports and compiled data. The reviewed data was then incorporated as initial estimates of the hydrogeologic parameters in the regional flow model, which was calibrated to observed post-development water level conditions. The calibrated model was thereafter validated to estimated predevelopment water level conditions. Development scenarios were then simulated with the calibrated model in order to estimate the regional impact of additional future withdrawals in the area. A local site (flow) model was then developed from the regional model. A tightened grid configuration was specified around the municipal wells in order to obtain drawdown estimates near the wellfield, and also to accurately delineate the WHPA. A hypothetical but representative model was developed to ascertain the impact of advective-dispersive processes on nitrate levels in the municipal wells.

1.4 Organization of Report

Section 1 describes the project objectives and technical approach. Section 2 provides an overview of the geologic, hydrogeologic, and climatic setting in the area and changes in groundwater conditions since pre-development times. Model calibration and results of the regional flow model are presented and discussed in Section 3. The local groundwater flow model is described in Section 4, along with the delineated Well Head Protection Areas, and development scenarios involving withdrawals from proposed well sites in northwest Hastings. The impact of advection-dispersion on nitrate migration is also addressed in this section. Finally, project summary and conclusions are presented in Section 5.

Section 2

Hydrogeolgic Setting and Groundwater Withdrawals

2.1 Geographic Setting

The model study area (Fig. 1.1) covers an area of approximately 760 square miles within Adams, Kearney, Hall, Clay, and Hamilton counties. It is bounded to the north by the Platte River while the southern boundary of the study area is approximately along a groundwater flow line with water south of this line draining to the Little Blue and Republican Rivers. The western boundary is along a north-south line through the town of Minden, while the eastern boundary is located approximately 3 miles east of the Adams-Clay county border. The land surfaces slopes gently to the south-southeast except in the areas where streams sharply dissect the uplands.

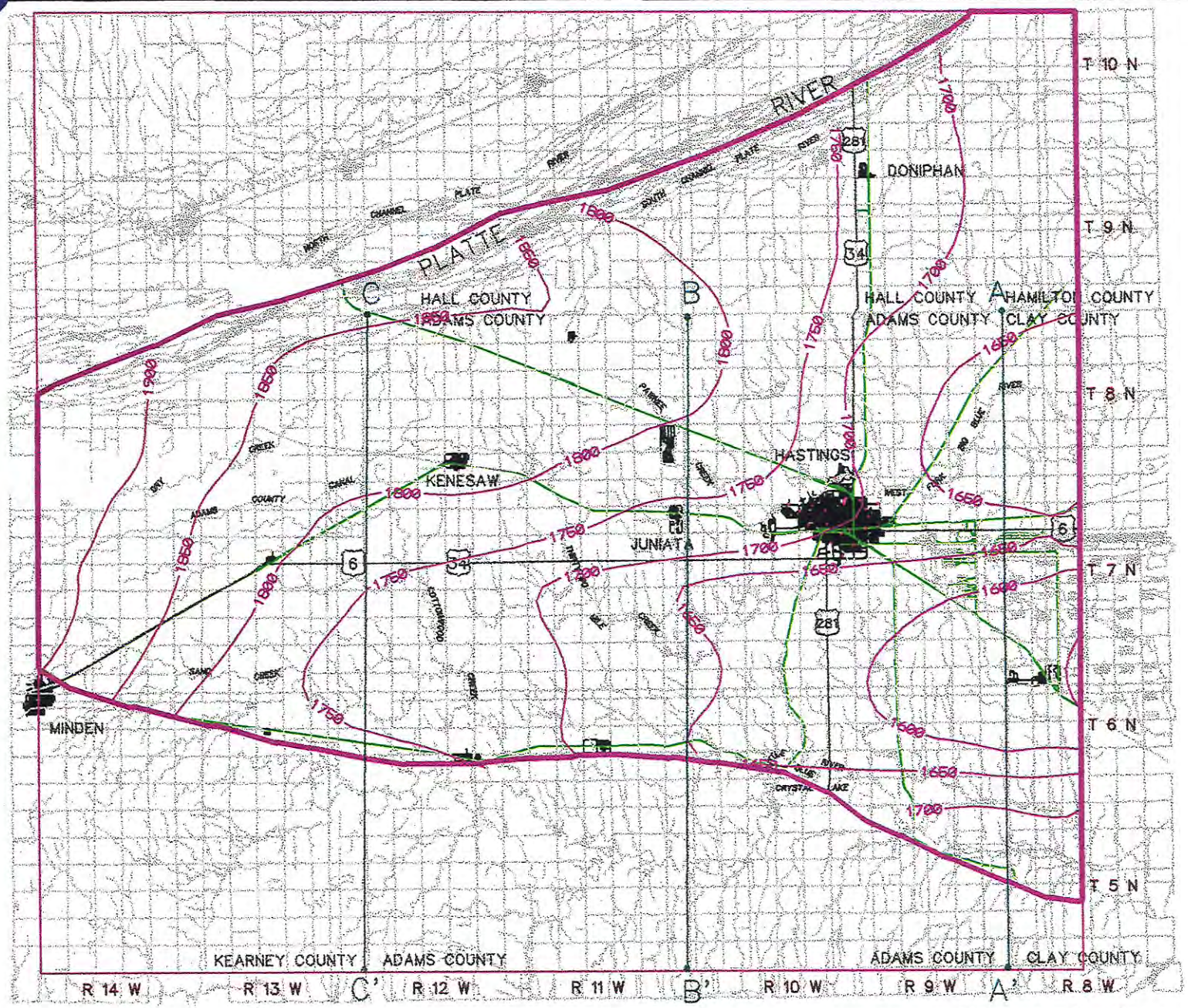
2.2 Hydrogeology

The area is underlain by unconsolidated deposits of Pleistocene (Quaternary) age and semiconsolidated deposits of Pliocene (tertiary) age having a combined thickness of about 100 to 500 feet (Keech and Dreeszen, 1968). The composition of the deposits consists of unconsolidated clay, silt, sand, and gravel. The unconsolidated deposits rests on the semi-permeable Ogallala Formation of Tertiary age where present. Underlying these deposits is a thick sequence of consolidated rocks of Cretaceous and other ages (Keech and Dreeszen, 1968), which due to its low permeability constitutes the base of the aquifer. The bedrock formation consists chiefly of marine deposits of sandstone, shale, chalk, and limestone. The bedrock surface, constructed from data in previous studies (Keech and Dreeszen; 1968, USGS; 1980) and presented in Figure 2.1, is a buried erosional terrain developed on the Niobrara Formation and Pierre Shale of late Cretaceous age; it slopes gently to the east. Profiles of geologic cross-section through Adams County are presented in Figures 2.2a, 2.2b, and 2.2c.

The Pleistocene aquifer is a very productive aquifer capable of producing large quantities of water (Keech and Dreeszen, 1968). It is the principal aquifer in the area and is considered unconfined, based on the location of the potentiometric surface relative to pumping and the response of the aquifer to pumping (PRC, 1990). Present day thickness of the water bearing formation ranges from 100-200 feet in the study area (Fig. 2.3). No major faults traverse the study area which could influence groundwater movement (Boyle, 1983).

2.3 Groundwater Levels

As indicated in section 2.2, the Pleistocene aquifer is the principal water bearing formation in the subsurface. Depth to the water table varies considerably in the area. In the central regions of the study area, the water table is over 100 feet deep. In the alluvial valleys along the Platte and Little Blue rivers, perched groundwater may be as shallow as seven to ten feet below ground surface (Woodward-Clyde, 1987). The configuration of the water table surface in the Pleistocene aquifer, constructed by the USGS for Spring 1979, is presented in Figure 2.4. The general direction of



Bedrock Surface Elevation (ft, amsl)

FIGURE: 2.1

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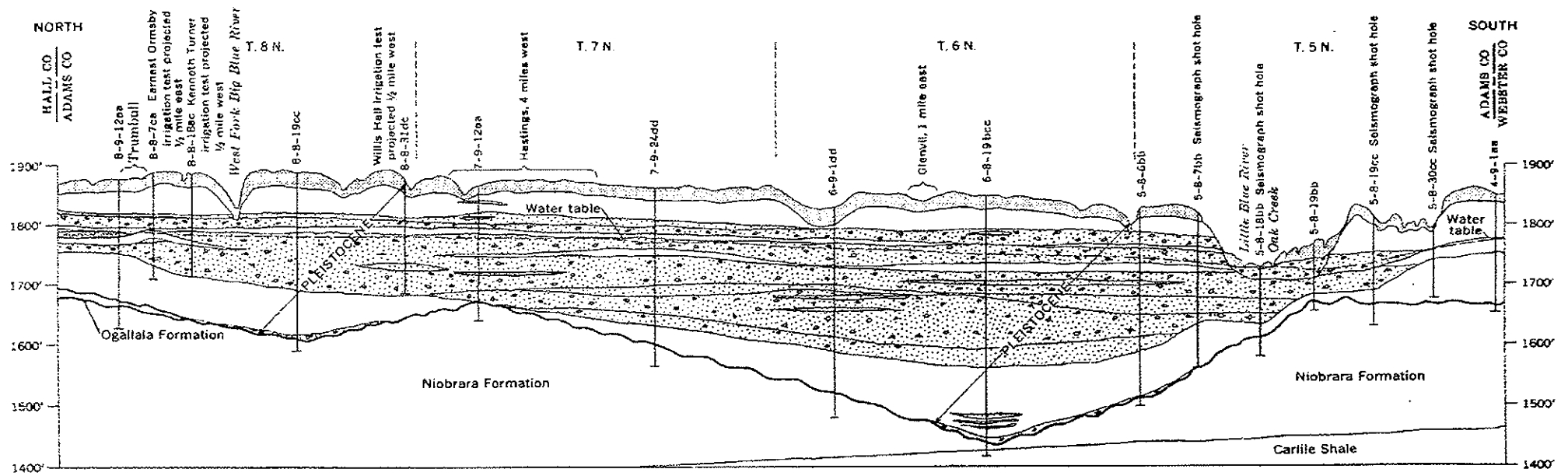

EXPLANATION

- Loess — *Includes modern soil and fine-grained alluvium*
- Silt and clay
- Silt and silty sand
- Sand
- Sand and gravel
- Marl and sandy limestone
- Sandstone

NORTH-SOUTH SECTION
ALONG THE ADAMS-CLAY COUNTY LINE
(SOURCE: KEECH AND DREESZEN, 1968)

FIGURE: 2.2a

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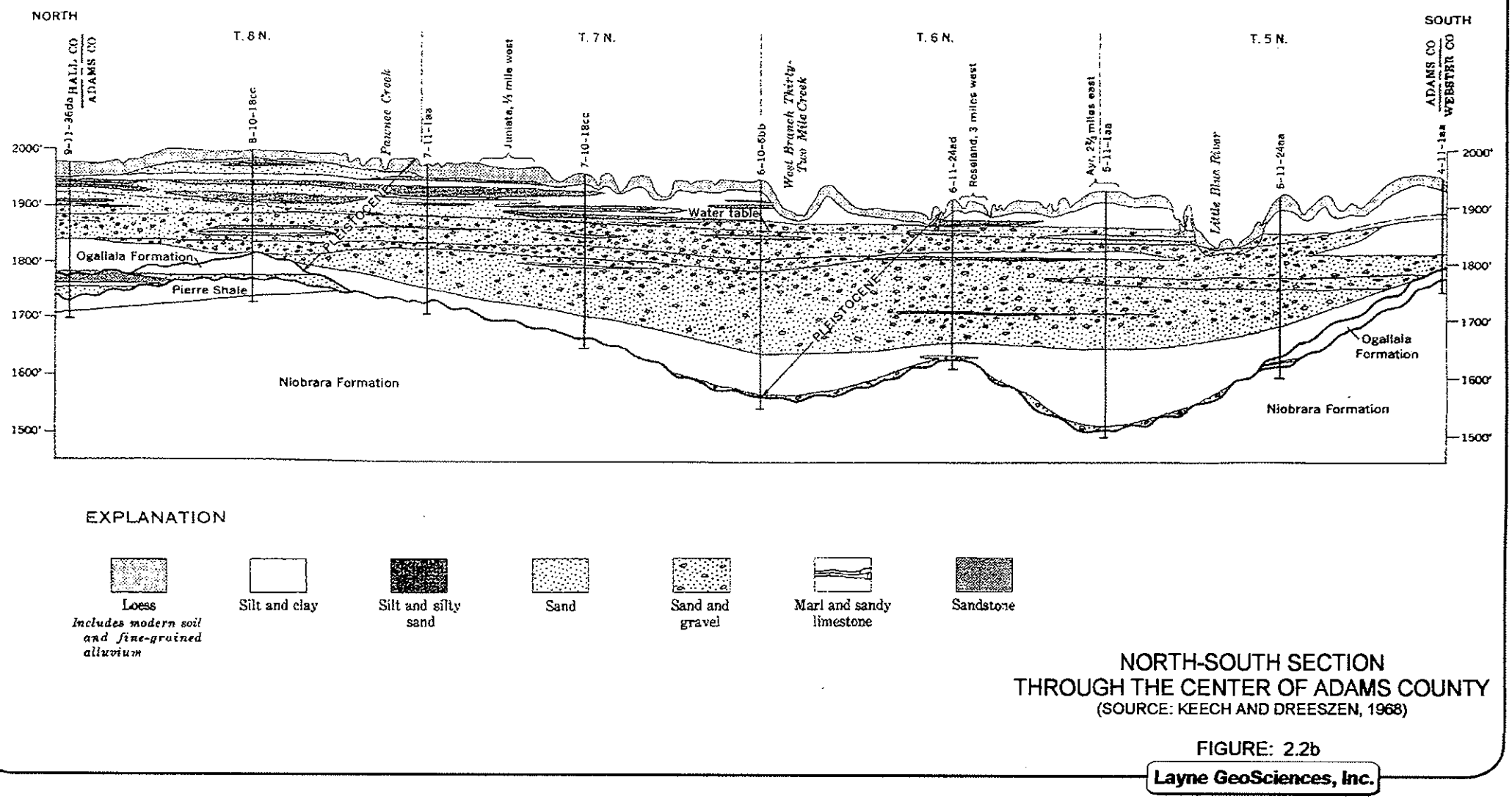
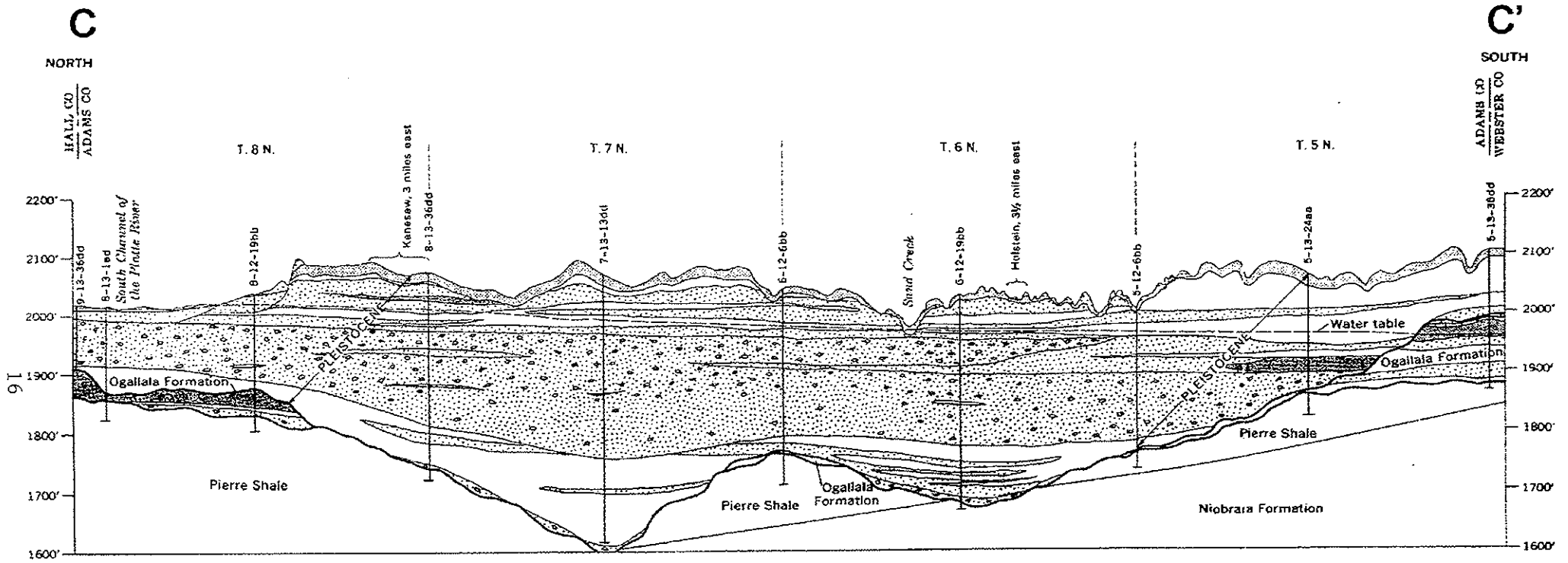


NORTH-SOUTH SECTION
THROUGH THE CENTER OF ADAMS COUNTY
(SOURCE: KEECH AND DREESZEN, 1968)

FIGURE: 2.2b

Layne GeoSciences, Inc.



EXPLANATION

Loess	Silt and clay	Silt and silty sand	Sand	Sand and gravel	Marl and sandy limestone	Sandstone

Includes modern soil and fine-grained alluvium

NORTH-SOUTH SECTION
ALONG THE ADAMS-KEARNEY COUNTY LINE
(SOURCE: KEECH AND DREESZEN, 1968)

FIGURE: 2.2c

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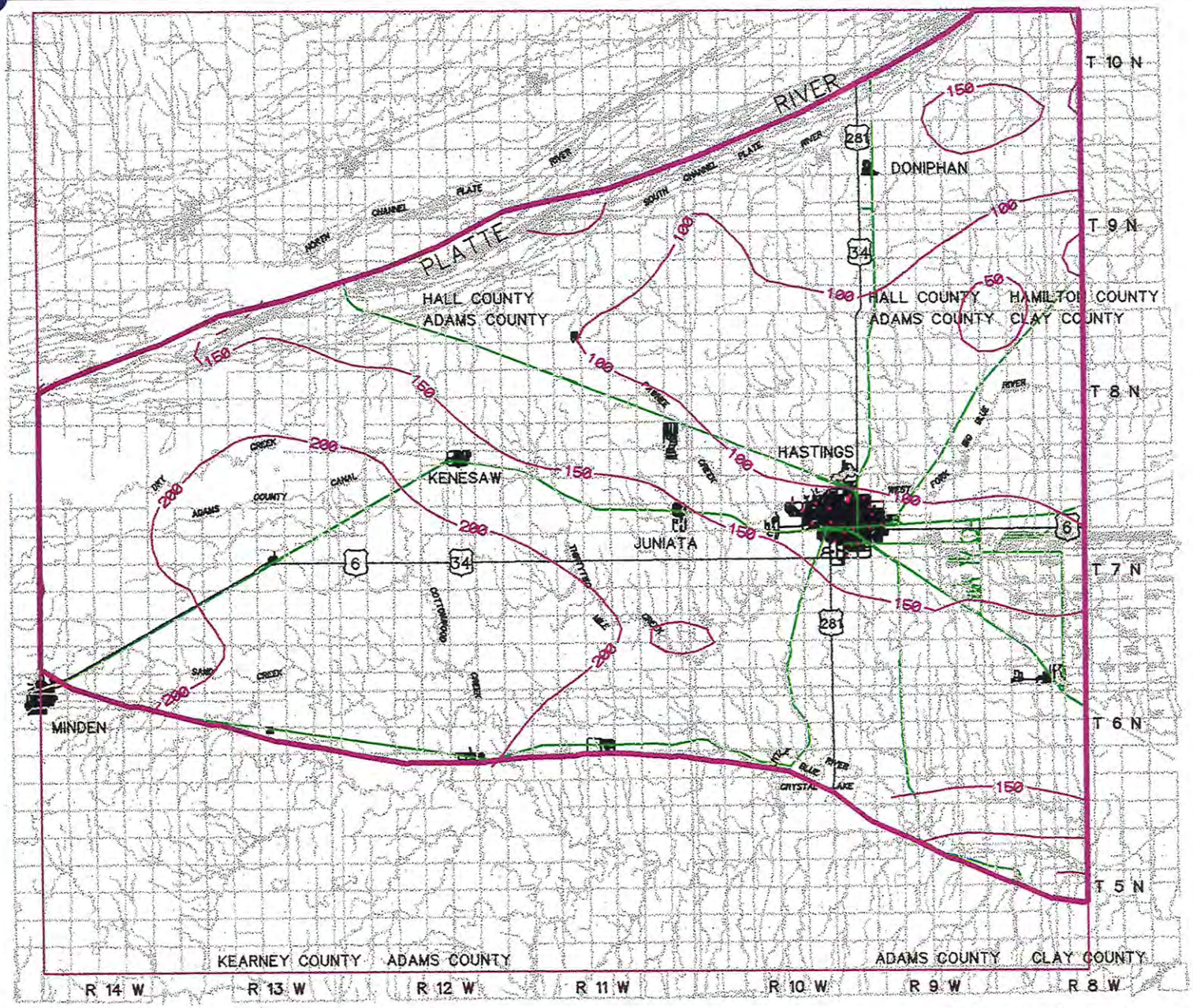

Saturated Thickness
of the Principal Aquifer (ft)

FIGURE: 2.3

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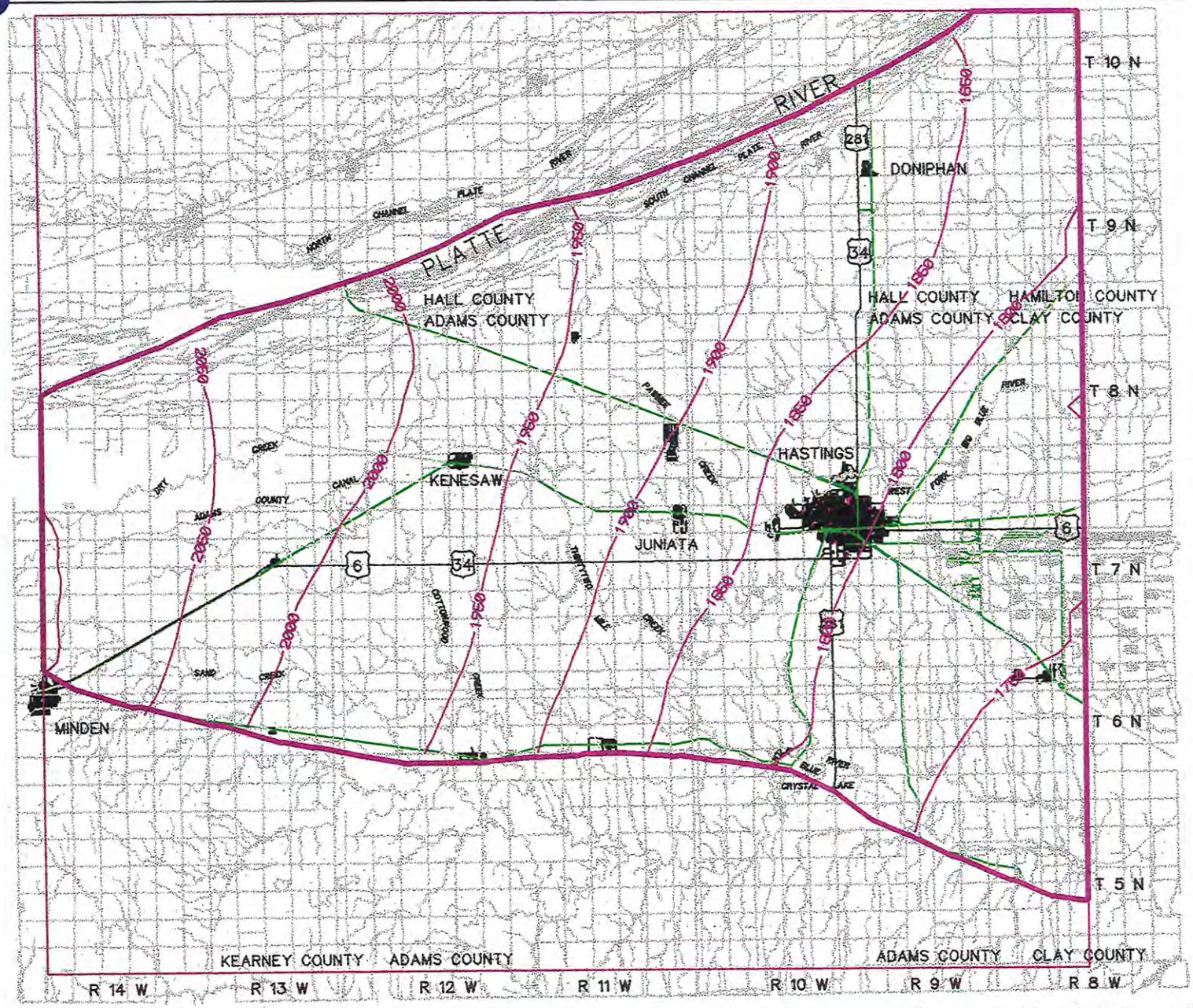


Water Table Configuration for Spring 1979 (ft, amsl)

FIGURE: 2.4

Layne GeoSciences, Inc.

groundwater flow is towards the east-southeast. The Platte River forms a hydraulic barrier with water north of the river not crossing into the study area. Along the western portions of the study area, the Platte River acts as a groundwater sink, while in the east it recharges the aquifer. The southern boundary is also approximately aligned along a groundwater divide that separates the drainage area of the Platte and the Little Blue River and its tributaries. A water table mound can be noted near the town of Minden along the western boundary. This is primarily due to seepage of water from the Phelps Canal (refer to section 2.6), which has raised water levels in the area since development. The average rate of groundwater flow in Adams County ranges from approximately ½ to 1 ft/day (Keech and Dreeszen, 1968).

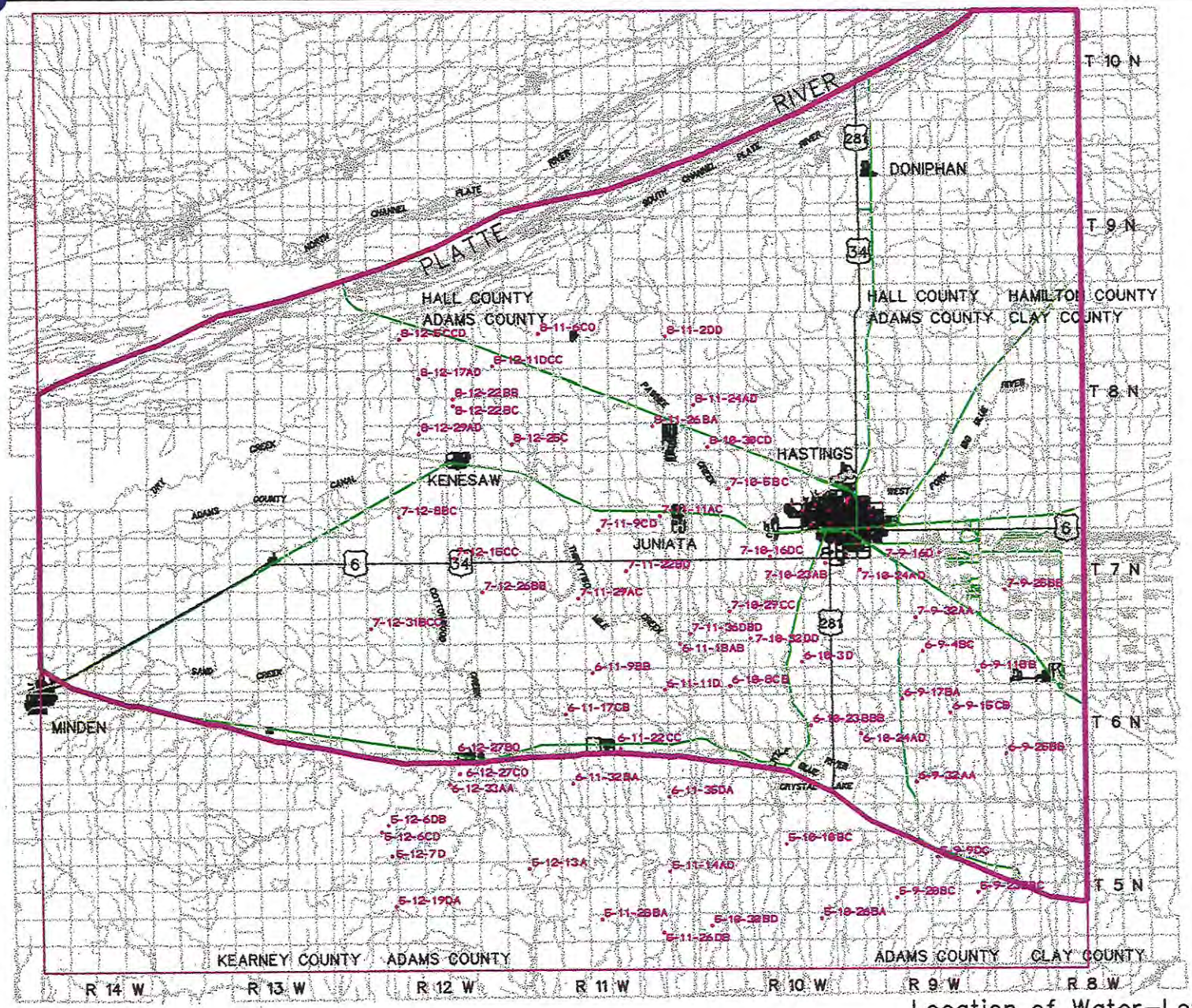
Water levels in observation wells throughout Adams County were analyzed for this study. The location of the monitoring wells are presented in Figure 2.5. Water level hydrographs at the monitoring sites are presented in Appendix A. Selected hydrographs are presented in Figure 2.6a-e. It is apparent from the hydrographs that although water levels have declined since development, they appear to have stabilized. There is annual and seasonal fluctuations in the water levels, but regionally, water levels appear to have remained fairly stable for the past decades even though groundwater withdrawals have increased during this period. This is likely due to increased recharge from the Platte River with development, increased recharge due to seepage from the Phelps Canal, and irrigation return flow. Water levels in monitoring wells near Hastings do show relatively greater fluctuations and drawdowns. This is attributed to the influence of pumpage at the Hastings Utilities wellfield.

The USGS and the Nebraska Conservation and Survey Division intermittently publishes maps depicting changes in water levels since predevelopment times. The groundwater water-level changes from predevelopment up to 1982 and 1990 are presented in Figure 2.7a and b. On comparing the figures, it is apparent that although water levels in the study area have declined, they have remained relatively stable. No discernible trend in water level changes is noticeable on a regional basis for recent times. A drawdown of up to 30 feet at Hastings has occurred since development. Also, a fairly persistent groundwater mound (of greater than 20 feet) is noted near the town of Minden along the western boundary. This is likely due to seepage of surface water from the Phelps Canal. The absence of a persistent trend in recent times suggests that although groundwater levels do fluctuate with climatic conditions, on a long-term average basis the aquifer system may have stabilized with increased withdrawals supplanted with induced recharge.

2.4 Aquifer Hydraulic Conductivity and Saturated Thickness

The aquifer transmissivity, a measure of the systems ability to transmit water, is reported to vary between 50,000 to over 200,000 gpd/ft in the study area (Boyle, 1983). The transmissivity is a product of the saturated thickness and the hydraulic conductivity of the unconsolidated deposits.

The saturated thickness varies from approximately 50 to over 200 feet in the study area (Fig. 2.3). The hydraulic conductivity varies significantly in the area reflecting the complex stratigraphy in the subsurface. Aquifer pump tests at 15 sites (Geraghty & Miller, Inc., 1994) near Hastings



7-12-31BCC TOWNSHIP–RANGE–SECTION

Location of Water–Level Monitoring Wells in Adams County

FIGURE: 2.5

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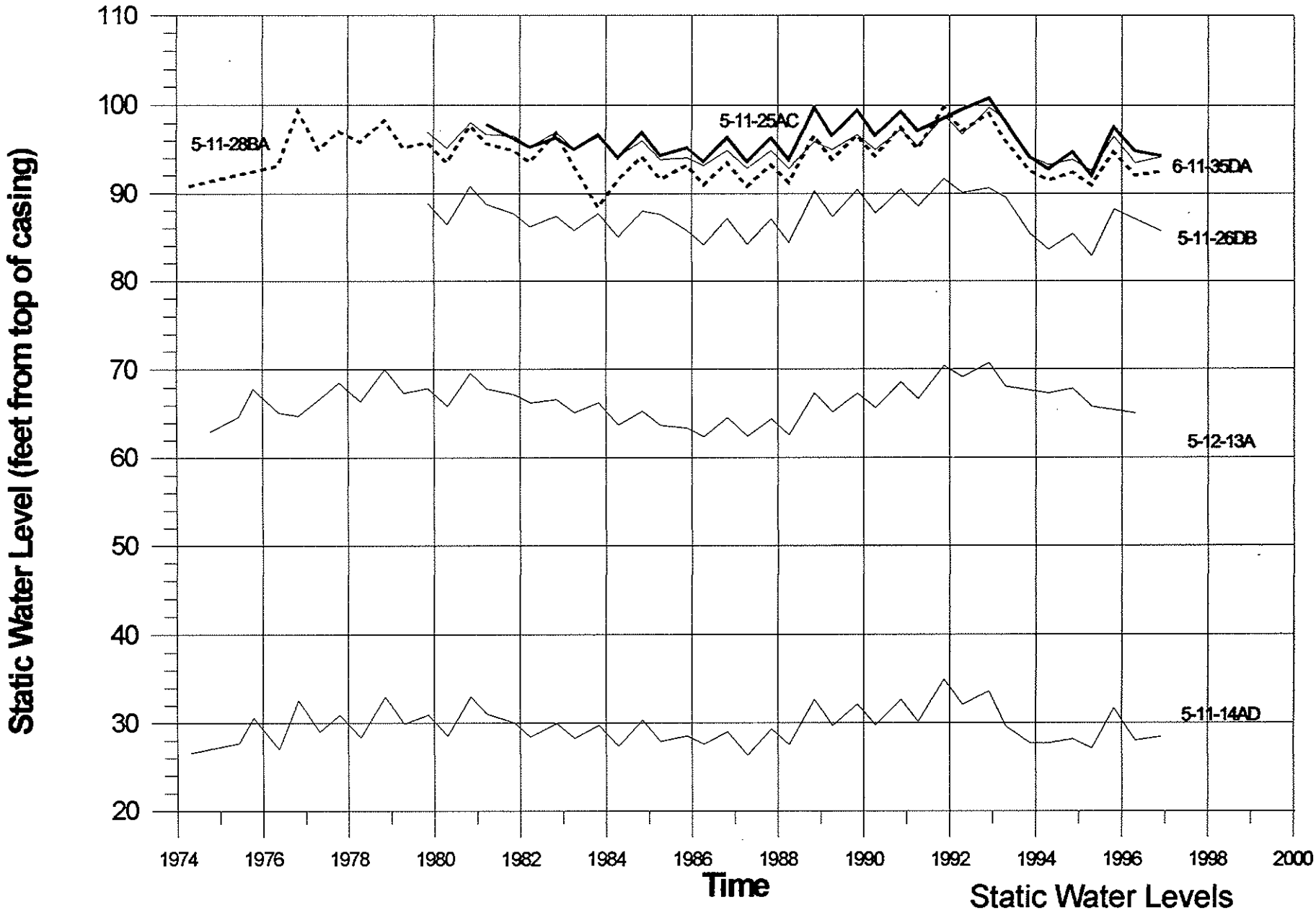

Static Water Levels
in Observation Wells
Southwest of Hastings

FIGURE: 2.6a

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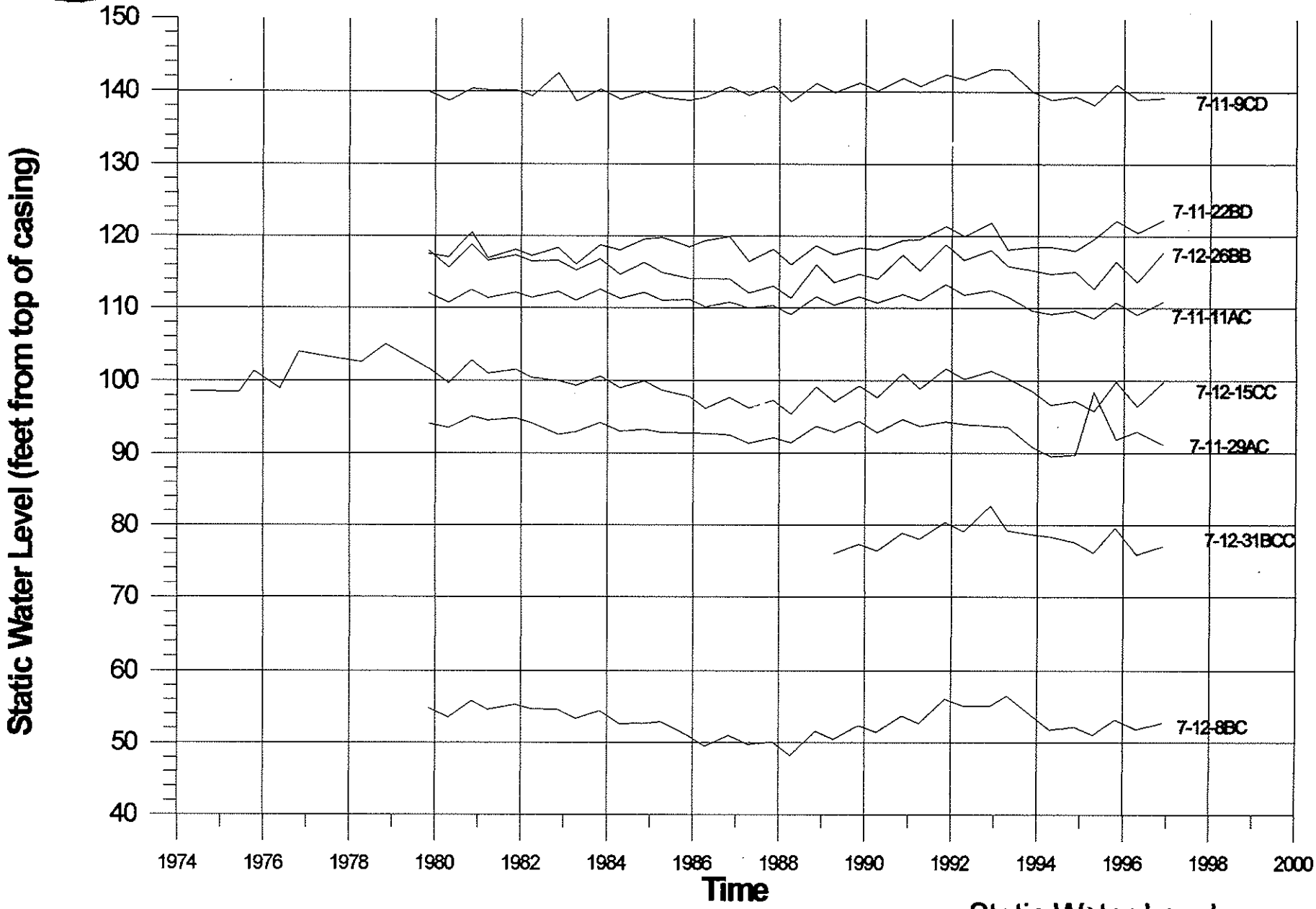

Static Water Levels
in Observation Wells
West of Hastings

FIGURE: 2.6b

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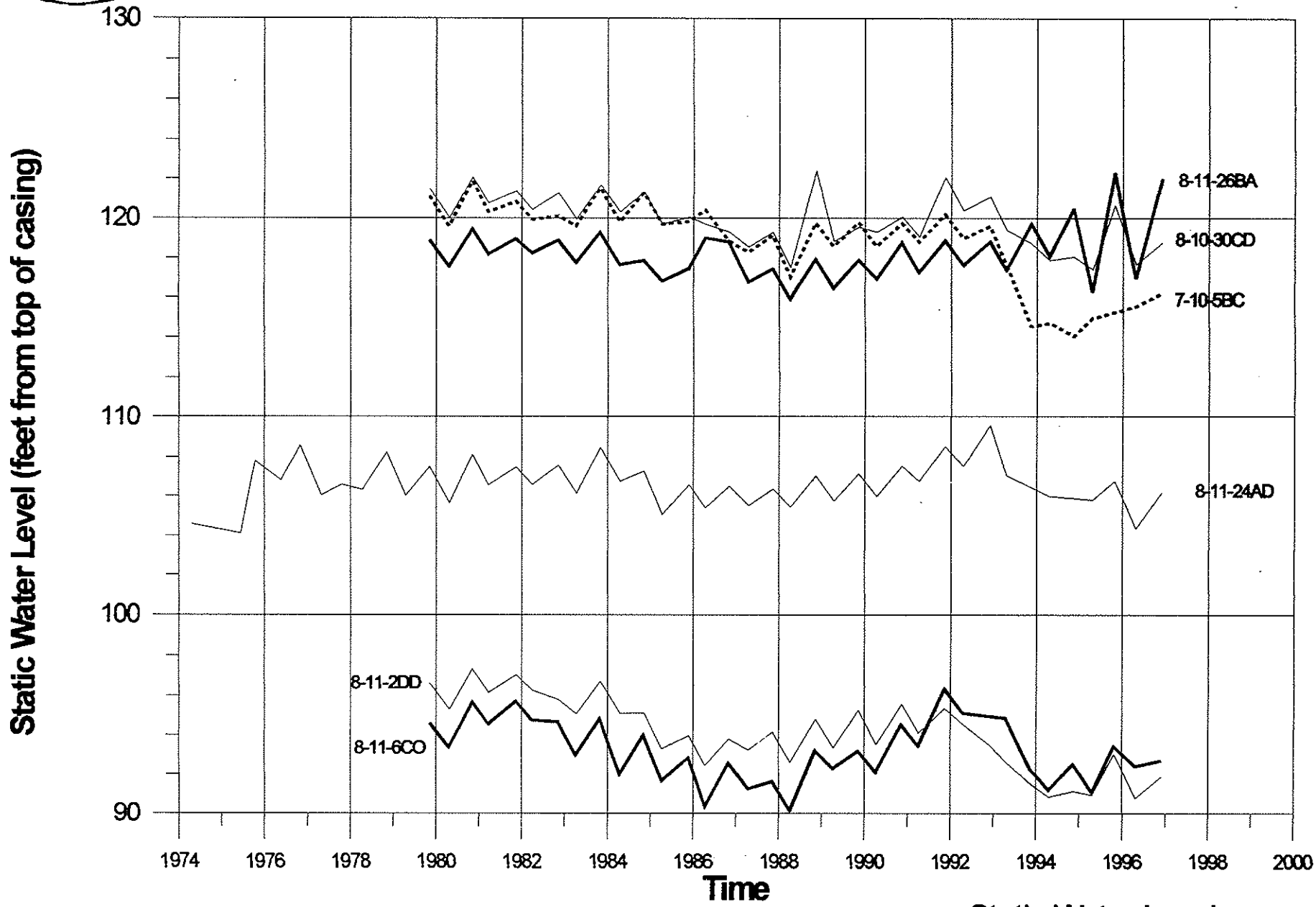

Static Water Levels
in Observation Wells
Northwest of Hastings

FIGURE: 2.6c

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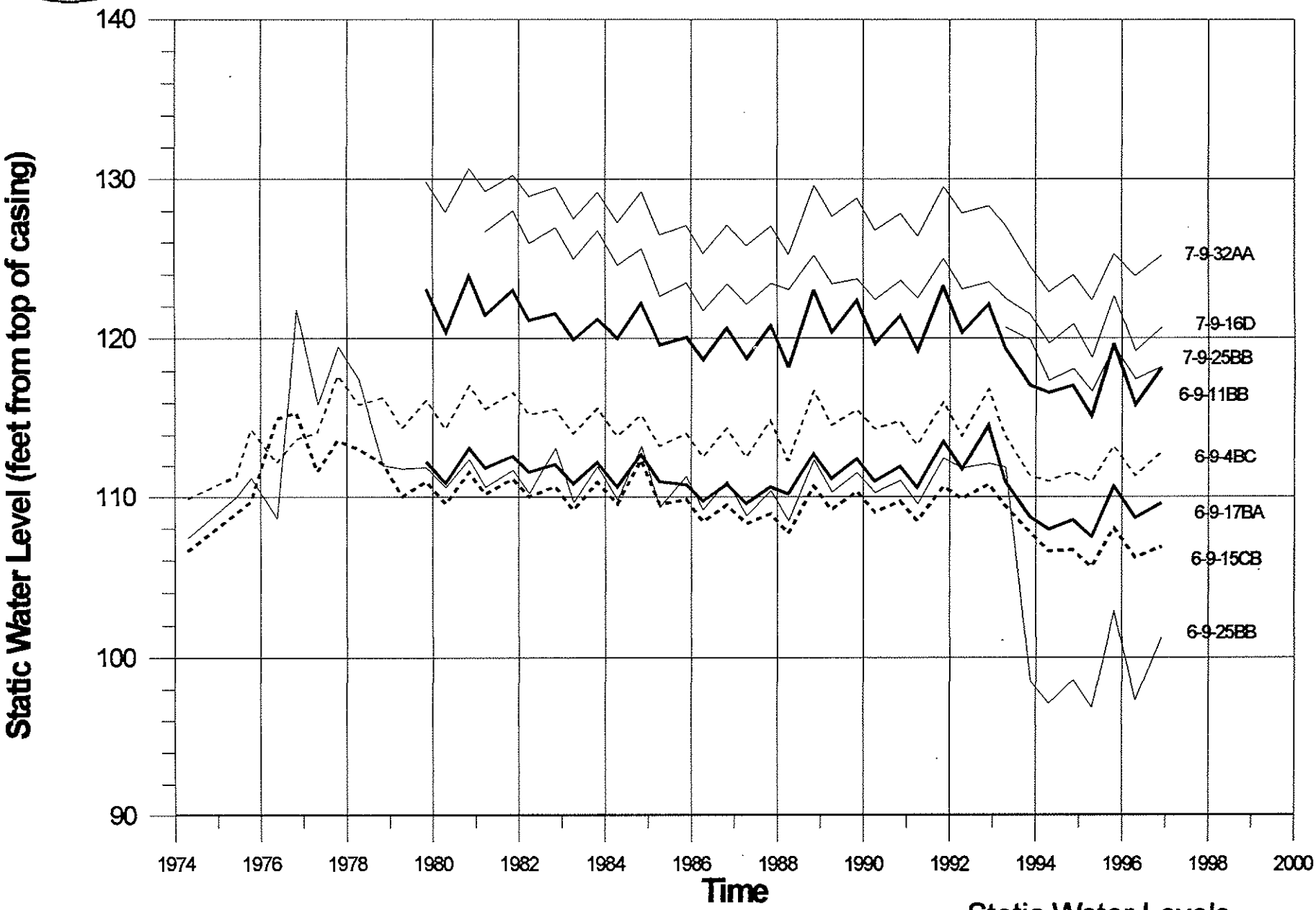
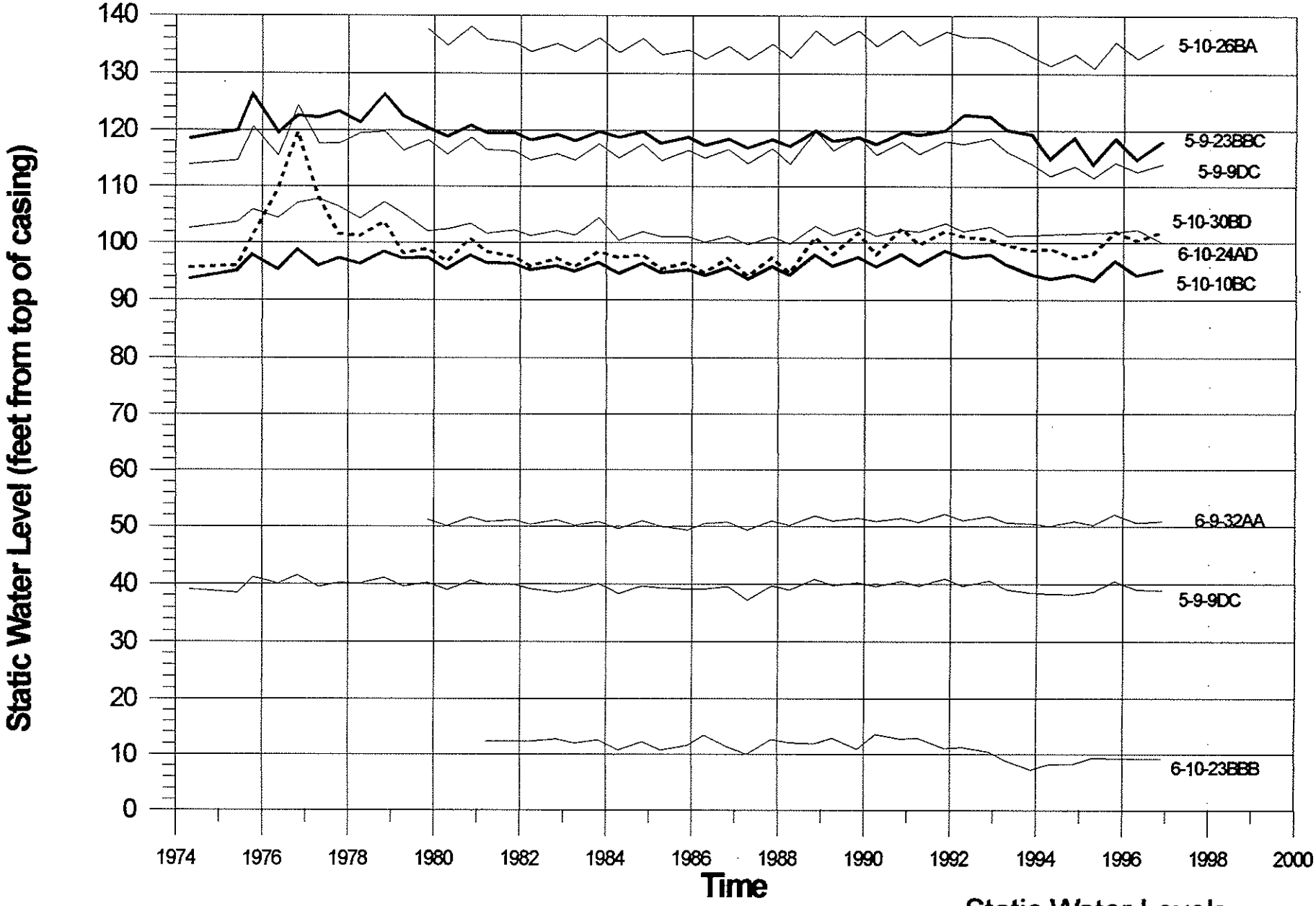


Static Water Levels
in Observation Wells
Southeast of Hastings

FIGURE: 2.6d

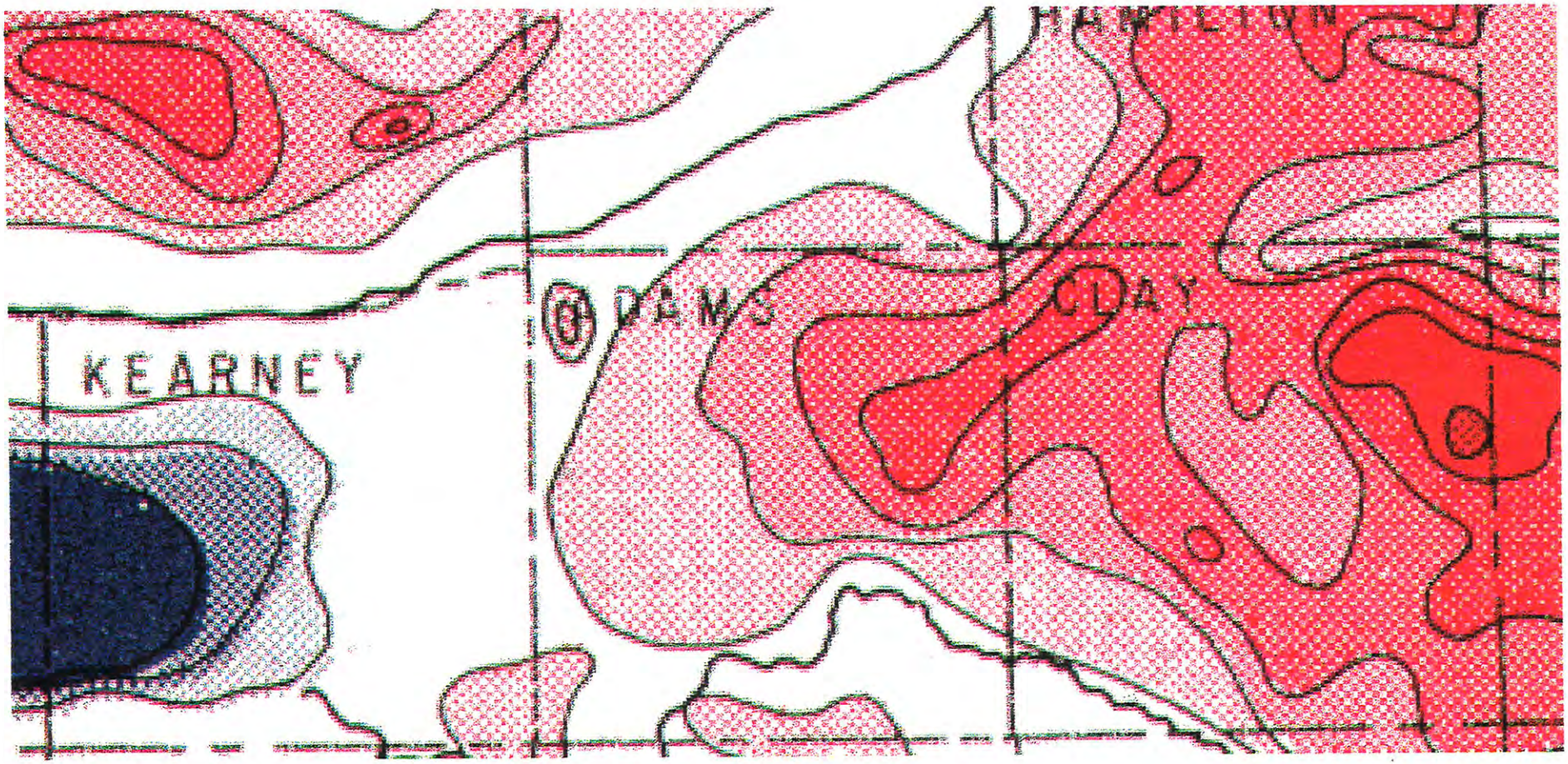
Layne GeoSciences, Inc.



Static Water Levels
in Observation Wells
South of Hastings

FIGURE: 2.6e

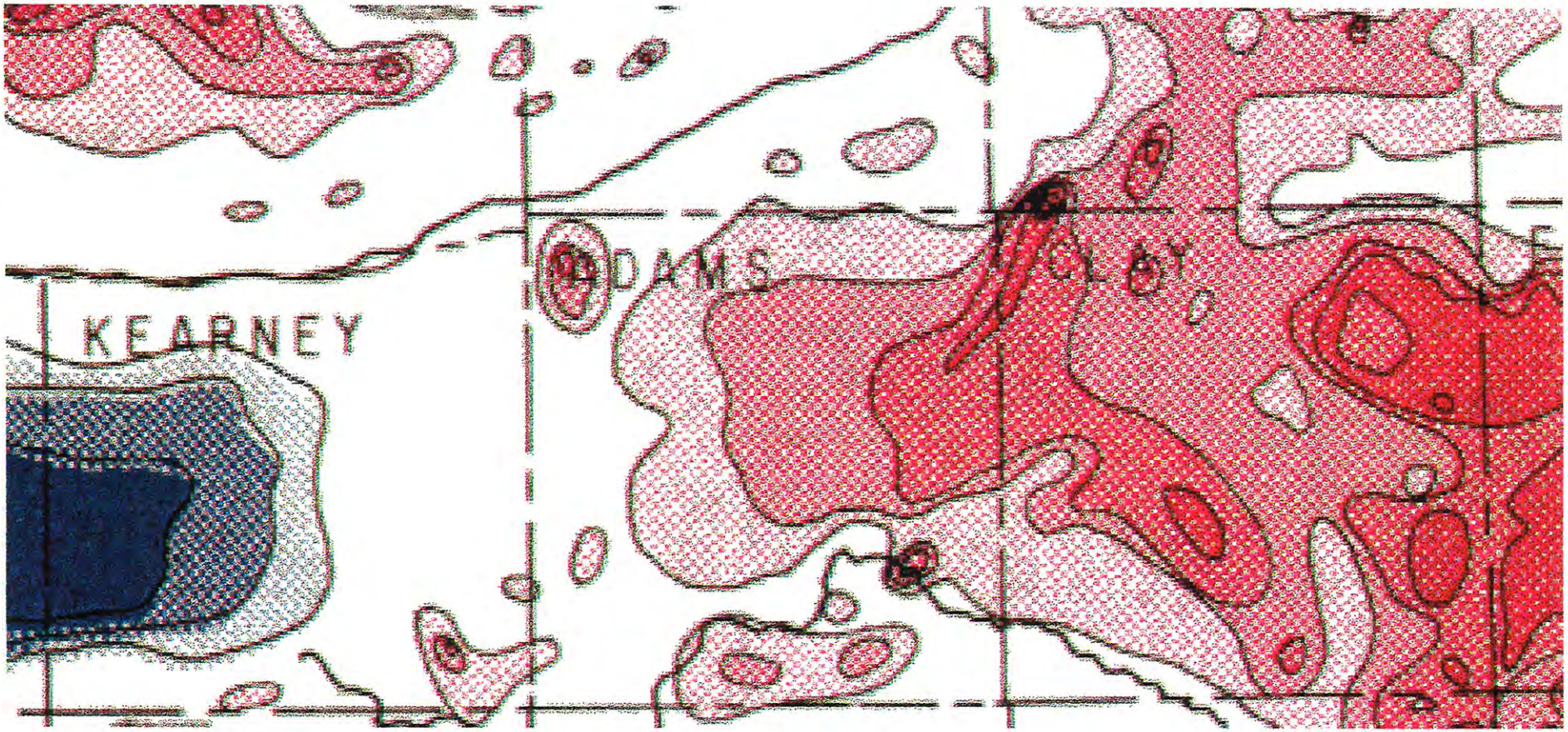
Layne GeoSciences, Inc.



CHANGES IN GROUNDWATER LEVELS FROM PREDEVELOPMENT TO 1982

FIGURE: 2.7a

Layne GeoSciences, Inc.



CHANGES IN GROUNDWATER LEVELS FROM PREDEVELOPMENT TO 1990

FIGURE: 2.7b

Layne GeoSciences, Inc.

yielded hydraulic conductivity values ranging from 28.8 to 609.1 ft/day. A value of 200 ft/d was specified by Woodward-Clyde Consultants (1990) for a modeling study at the Hastings East Industrial Park.

2.5 Aquifer Recharge

Groundwater recharge due to precipitation varies considerably in the study area. The normal yearly precipitation at five precipitation stations in the area for the period 1941 to 1970 is presented in Table 2.1. The average annual precipitation at the five stations is approximately 25 inches.

The amount of recharge varies significantly both spatially and temporally. In general, recharge varies with precipitation, depth to the water table, soil type, presence of confining units, and topography. In the vicinity of Hastings, where depth to the water table exceeds 100 feet, recharge is reported to vary from 1 to 2 in/yr (PRC, 1990). Further north-west, along the sand dunes in the Platte River Valley and near Sand Creek, southwest of Holstein, significantly higher recharge rates are expected (Keech and Dreeszen, 1968). Significant quantities of water also infiltrate back into the aquifer system due to irrigation return flow. This phenomenon can cause the postdevelopment rates of recharge to be much higher than predevelopment rates of recharge (Woodward-Clyde, 1990).

2.6 Surface Water

The Platte River is hydraulically well connected with the underlying aquifer (Lappala et al., 1979). This is also evident from the water table contours presented in Figure 2.4, which indicates a groundwater divide along the Platte River. In the western regions of the study area, the Platte River acts a drain for the groundwater system, while it is a source of water in the eastern regions. There are several major streams and creeks in the study area (Fig. 1.1), the flow in which is intermittent except in the Little Blue River in the southeast and Lost Creek in the northwest which appear to be perennial streams. Other creeks in the region flow from runoff following high precipitation events. Due to their elevation with respect to the water table, the creeks are a source of recharge to the aquifer.

The Central Nebraska (Tri-County) Public Power and Irrigation District constructed the Phelps County Canal (Fig. 1.1) to deliver irrigation water from the North Platte River to areas south of the Platte River. However, the portion of this canal in the study area was never used because litigation resulted in a court order that forbade transporting water from one drainage basin to another, in this case the Platte to the Blue River basin (Keech and Dreeszen, 1968). In recent years, flow of diverted water in the Phelps Canal is controlled such that the effective terminus of the canal system is near the town of Minden along the western boundary of the model domain. Therefore, although no surface water flows into the study area, infiltration through the canal bed further west has caused water levels near Minden to rise by approximately 20 to 30 feet since development (refer to Figs. 2.7a and b).

Table 2.1
Normal Yearly Precipitation (source: Boyle, 1983)

Station	Normal Annual Precipitation (inches)	Period of Record (Years)
Holdredge	24.9	91
Kearney	24.6	108
Minden	25.3	103
Grand Island	23.1	89
Hastings	26.6	90

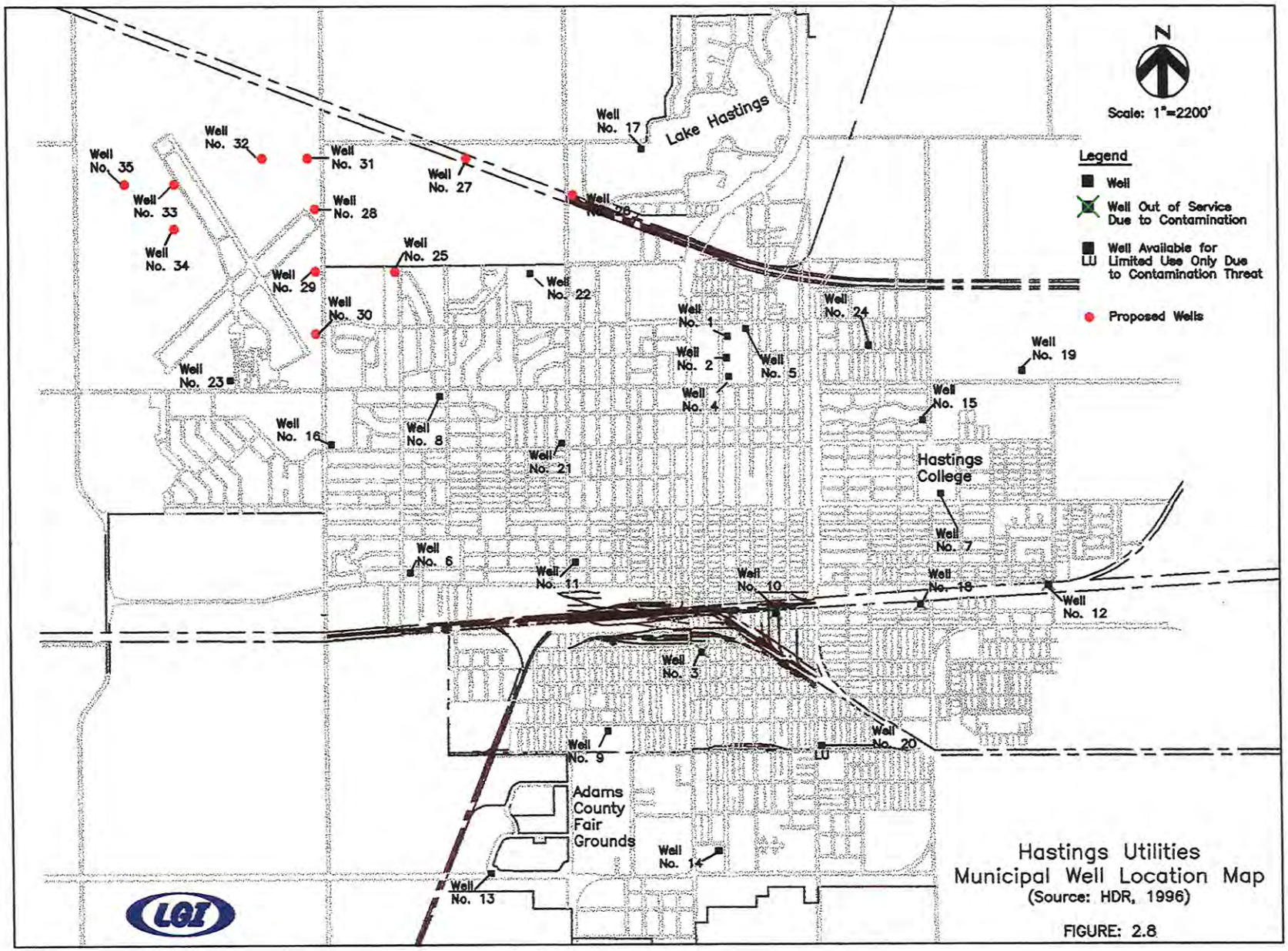
2.7 Groundwater Withdrawals

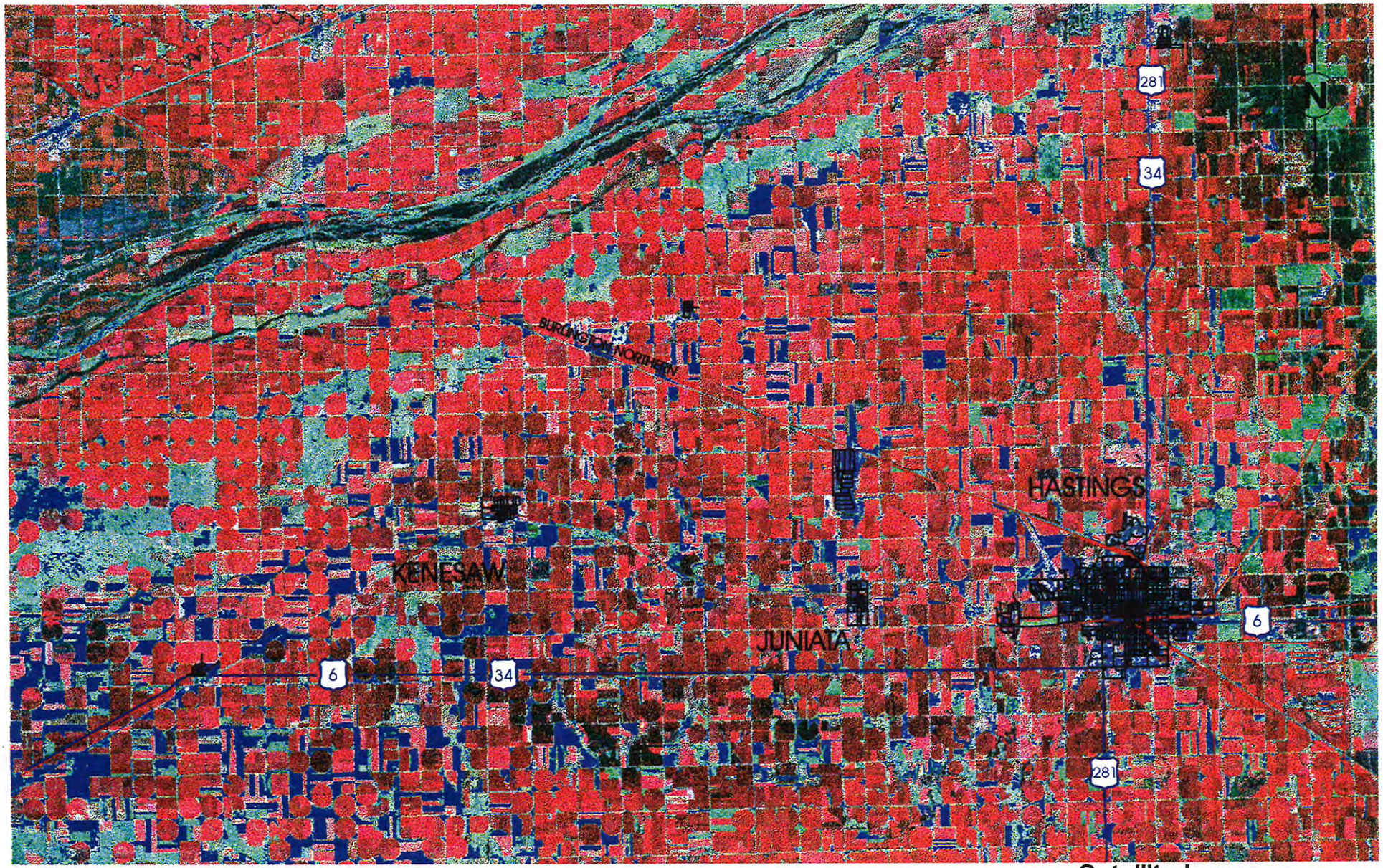
Groundwater withdrawals in the area are primarily for agricultural and municipal uses with relatively minor quantities withdrawn for industrial and domestic purposes (Steele, 1988). The average daily withdrawal at the Hastings Utilities (HU) wellfield (Fig. 2.8) is approximately 6 MGD. The average annual withdrawal at the municipal wells for the period 1993-95 is presented in Table 2.2.

Virtually all water for irrigation purposes in Adams County is from groundwater (Keech and Dreeszen, 1968). The amount of water applied each season varies significantly with precipitation, crop type, and irrigation method. The gradual switch to center-pivot versus flood irrigation has resulted in reduced withdrawals of groundwater (Steele, 1988). By far the primary determinant of the amount of water applied each season is rainfall. Based on analyses of 9 years (1982-1990) of water use data for a 5 county region, and a 1984 Adams County water use study (Mike Onen, Little Blue Natural Resources District, personal communication, Table 2.3a and 2.3b), an average usage of 12 in/year in cultivated areas was derived for irrigation purposes. Since 1983 and 1988 were dry years, therefore estimated water use for those years is high in Table 2.3b. Estimated water use for 1984 in Table 2.3b is more likely to represent average conditions in Adams County.

A large portion of the land available and suitable for farming is under cultivation. A Landsat Thematic Mapper (EROS) satellite image of the study area for August 19th, 1992 is presented in Figure 2.9. The red and black areas represent land under agricultural cultivation, while green indicates grassland. The blue areas represent developed residential and industrial areas. As is obvious from Figure 2.9, a large portion of the study area is under cultivation. Northwest of Hastings, almost 85 percent of the available land appears to be cultivated.

Irrigation extraction was specified to the model as a function of the percent of land use under cultivation as estimated from the satellite image. Five zones were established (Fig. 2.10) based on varying percentages of land under cultivation. The amount of groundwater extracted for irrigation in each zone was assumed to be a fraction of the 12 in/yr average groundwater withdrawal estimated above. Therefore, in the zone where cultivated land represents


N
Scale: 1"=2200'
Legend
Well
Well Out of Service Due to Contamination
LU Well Available for Limited Use Only Due to Contamination Threat
Proposed Wells
Lake Hastings
Well No. 17
Well No. 32
Well No. 31
Well No. 35
Well No. 27
Well No. 33
Well No. 28
Well No. 34
Well No. 25
Well No. 29
Well No. 22
Well No. 30
Well No. 1
Well No. 24
Well No. 2
Well No. 5
Well No. 19
Well No. 23
Well No. 4
Well No. 15
Well No. 16
Well No. 8
Well No. 21
Hastings College
Well No. 7
Well No. 6
Well No. 11
Well No. 10
Well No. 18
Well No. 12
Well No. 3
Well No. 9
LU
Well No. 20
Adams County Fair Grounds
Well No. 14
Well No. 13
LGI
Hastings Utilities
Municipal Well Location Map
(Source: HDR, 1996)
FIGURE: 2.8



Satellite Image
of Study Area

FIGURE: 2.9

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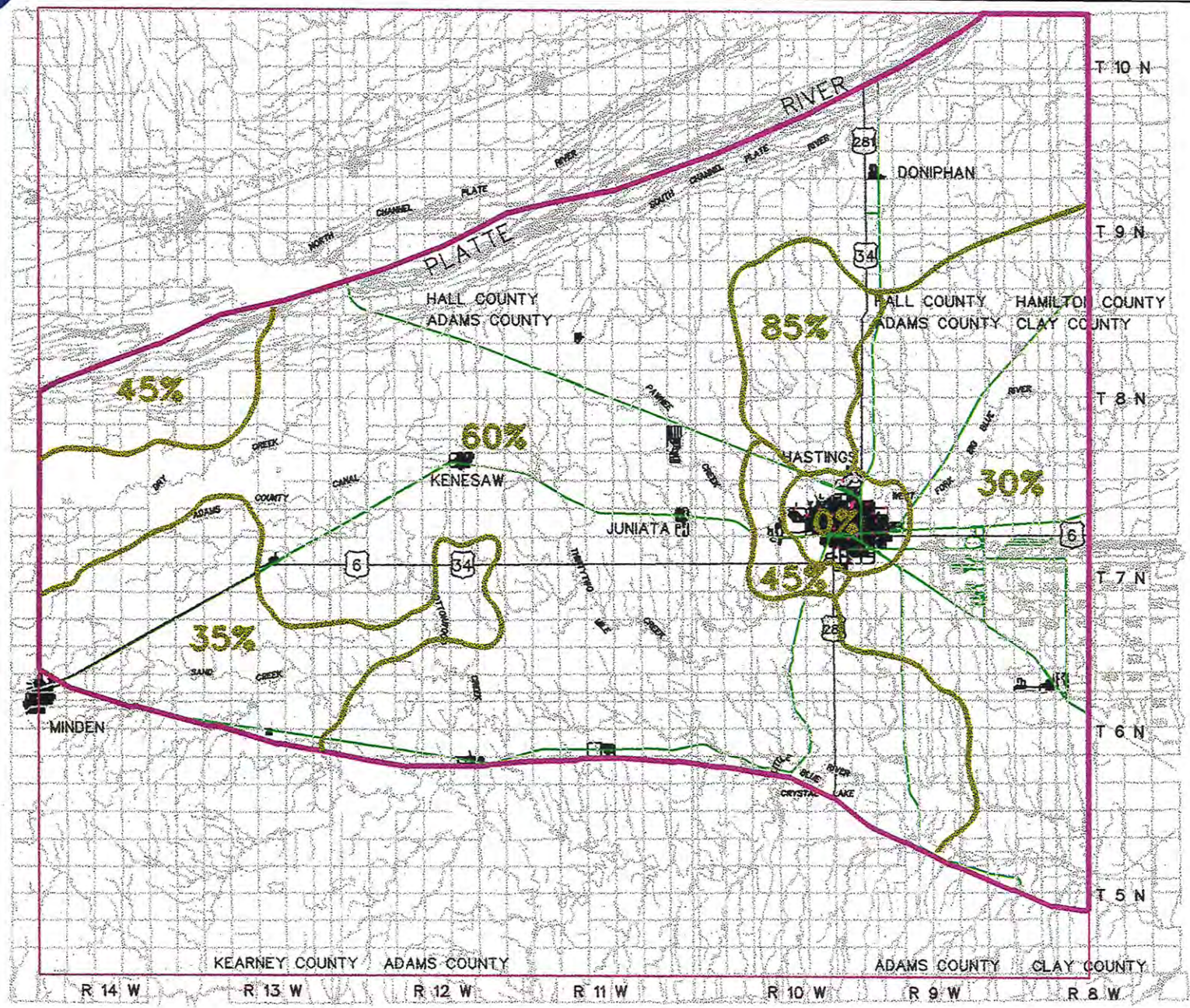

Irrigation Zones
(Percent of Land Under Irrigation)

FIGURE: 2.10

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approximately 50 percent of the total land area, a uniform extraction rate of 6 in/yr is estimated for the entire zone.

Table 2.2
Average Annual Withdrawal at HU Production Wells for the Period 1993-95

Well No.	Average 1993-95 Annual Withdrawal
	(Gallons)
1	132,487,976
2	12,050,429
3	0
4	119,271,614
5	0
6	34,168,951
7	49,640,249
8	57,058,599
9	125,674,138
10	0
11	168,813
12	0
13	181,176,082
14	44,317,062
15	9,261,500
16	361,201,466
17	187,434,317
18	0
19	153,951,930
20	63,000
21	315,396,042
22	63,700,474
23	48,122,970
24	99,060,721

Table 2.3a
Estimated Irrigation Usage (inches/year) in Portions of Adams, Clay, Fillmore, Thayer, and Nuckolls Counties

	ALL CROPS			CORN ONLY		
Year	Gravity	Center Pivot	Average	Gravity	Center Pivot	Average
1982	9.12	4.81	7.65	10.86	5.46	8.87
1983	17.74	12.28	16.12	20.03	13.56	17.57
1984	12.30	8.80	10.30	NA	NA	NA
1985	8.20	6.30	7.30	10.65	7.98	9.69
1986	7.72	6.70	7.43	9.93	8.25	9.07
1987	11.40	7.94	9.92	12.45	8.52	10.98
1988	21.20	14.80	19.05	24.60	15.14	20.61
1989	12.11	10.47	11.41	14.12	11.93	13.40
1990	13.26	7.73	10.78	15.10	9.29	12.43
Average	12.56	8.87	11.11	14.72	10.02	12.83

Table 2.3b

Estimated Irrigation Usage (inches/year) in Adams County

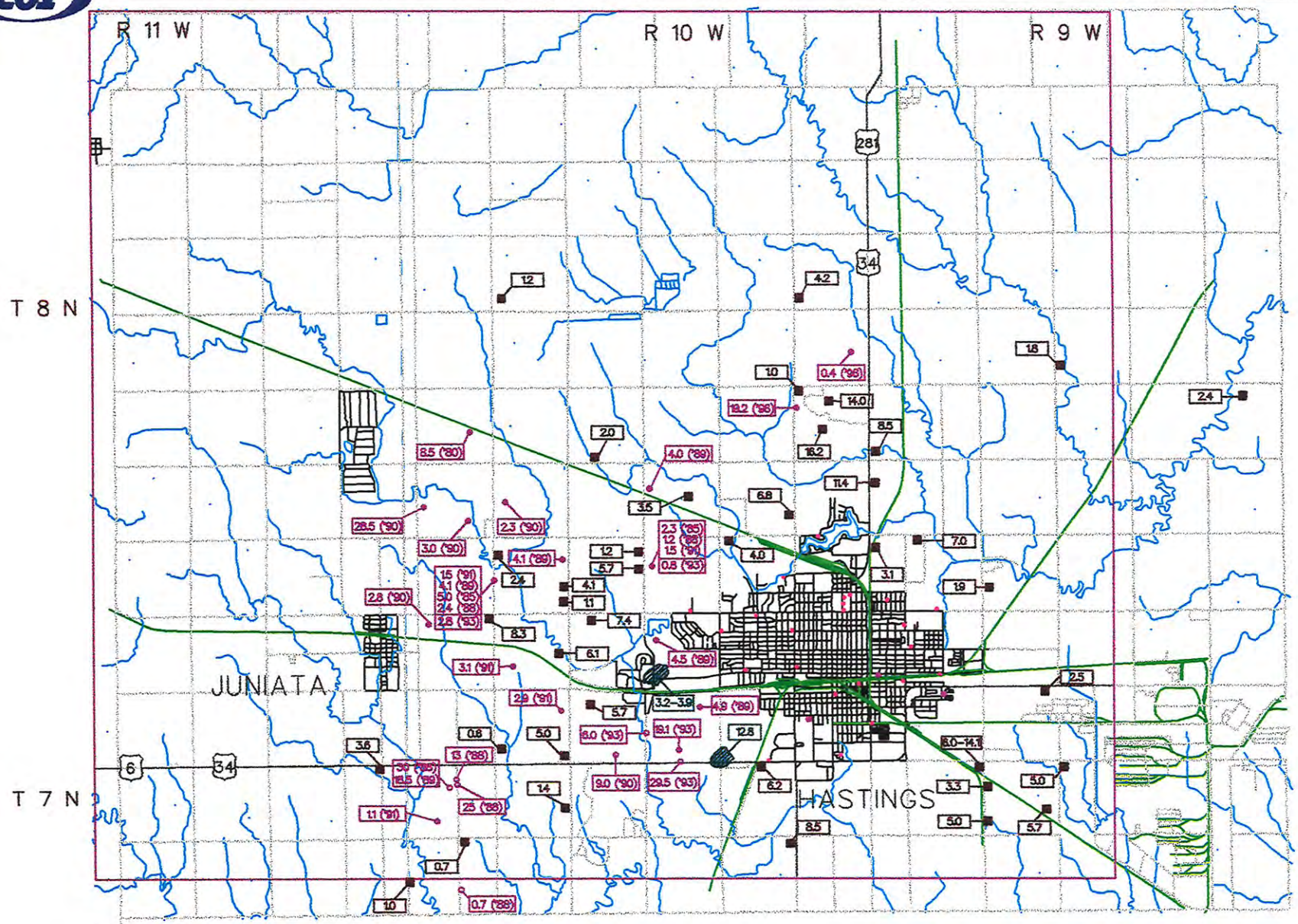
Gravity Irrigation				
	1983		1984	
CROP	acres	in/acre	acres	in/acre
Corn	1121	22.10	2904	16.20
Beans	78	9.50	60	13.30
Corn / Beans	217/208	23.10	1416/596	15.30
Corn / Alfalfa	119/21	16.80	80/13	16.90
Corn/ Beans/ Milo	62/35/32	19.80	295/170/172	9.50
Corn/ Beans/ Alfalfa			118/99/172	10.60
Beans / Milo			46/185	5.80
Total	1893	21.26	6,164	12.50
Center Pivot				
Irrigation				
	1983		1984	
CROP	acres	in/acre	acres	in/acre
Corn	628	13.50	2509	10.90
Corn / Beans	128/121	14.70	253/93	11.70
Corn/Beans/Milo/Alfalfa			41/85/92/2	10.20
Corn/ Alfalfa			115/5	17.40
Beans	191	7.70	90	5.40
Beans & Other			90/14	8.50
Total	1068	12.60	3389	10.70
Total Combined	2961	18.41	9553	11.60
1988 Totals	Reported for All Crops			
	acres	in/acre		
Gravity Irrigation	2117	21.80		
Center Pivot irrigation	1132	14.50		

2.8 Nitrates in Groundwater

Due to the application of fertilizers for lawns, parks, and crop cultivation, nitrate levels in groundwater have been steadily increasing over the years throughout the Plain states (Schepers, 1983). Additionally, feed-lots and septic tanks are (point) sources of nitrate contamination in the groundwater system. Nitrates in excess of 10 parts per million (ppm, measured as Nitrogen) in drinking water is considered unsafe for consumption by infants and pregnant women by the U.S. EPA as it can lead to a possibly fatal medical condition known as methemoglobinemia.

The distribution of nitrates in groundwater under Hastings and the surrounding areas is presented in Figure 2.11. Most of the sample measurements were obtained from tap water drawn from domestic wells. The concentration should be viewed as vertically averaged concentration and are presented to provide only an estimate of nitrate levels in the area. No spatial trend should be inferred from this figure as the data is not derived from a controlled study which accounts for the multitude of variables affecting nitrate concentrations such as, well depth, well screen location, hydrostratigraphy, proximity of point sources such as septic tanks, etc.

Nitrate levels in the Hastings Utilities wells have been displaying an upward trend. Nitrate concentration in the HU municipal wells is presented in Appendix B. Hydrographs at selected wells are presented in Figure 2.12. As is evident from the figure, nitrate concentrations in general have increased gradually from about 1 ppm in the late sixties and early seventies to about 4-5 ppm in recent years. Occasional spikes in the concentration levels could be the result of localized plumes passing through the well sites (e.g. Well No. 11). A number of remedial options can be considered for treating pumped groundwater with elevated nitrate levels. Effective technologies include ion exchange and reverse osmosis (HDR, 1996). Less common treatment measures involve electrodialysis and biological means of denitrification. However, all these are costly remedial measures. Therefore, better land management practices within the HU wellfield groundwater zone of contribution should be attempted in order to minimize contamination of groundwater.



Area Nitrate Levels

FIGURE: 2.11

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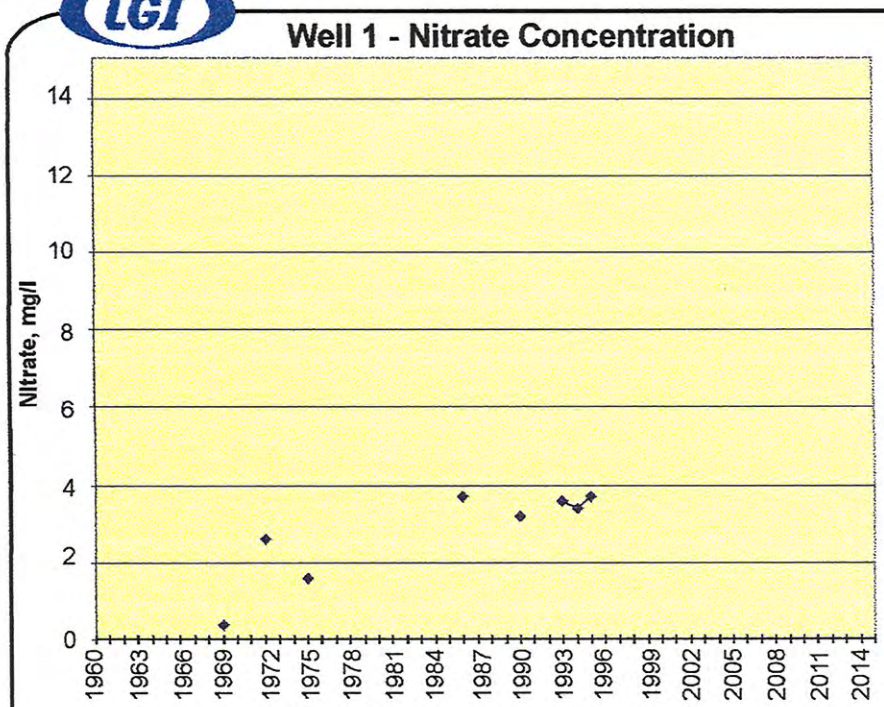

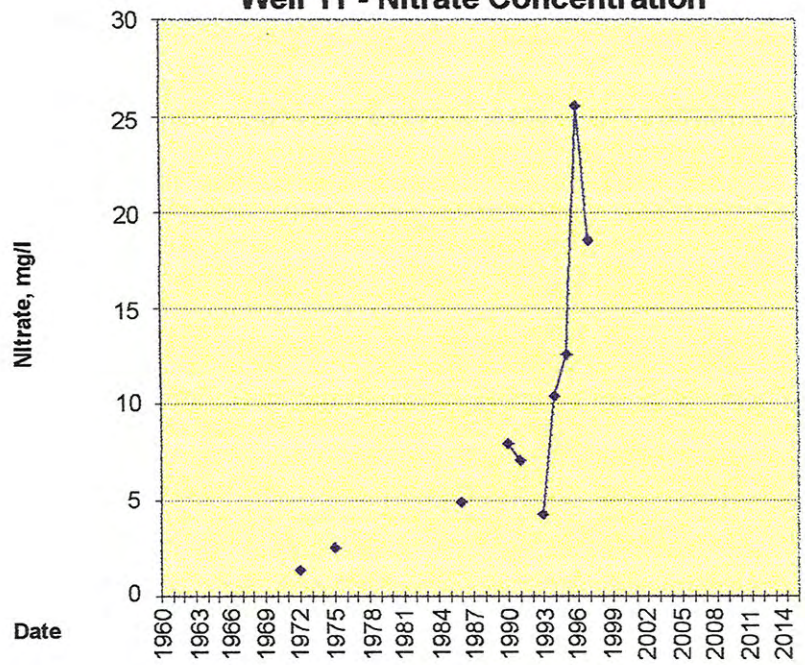

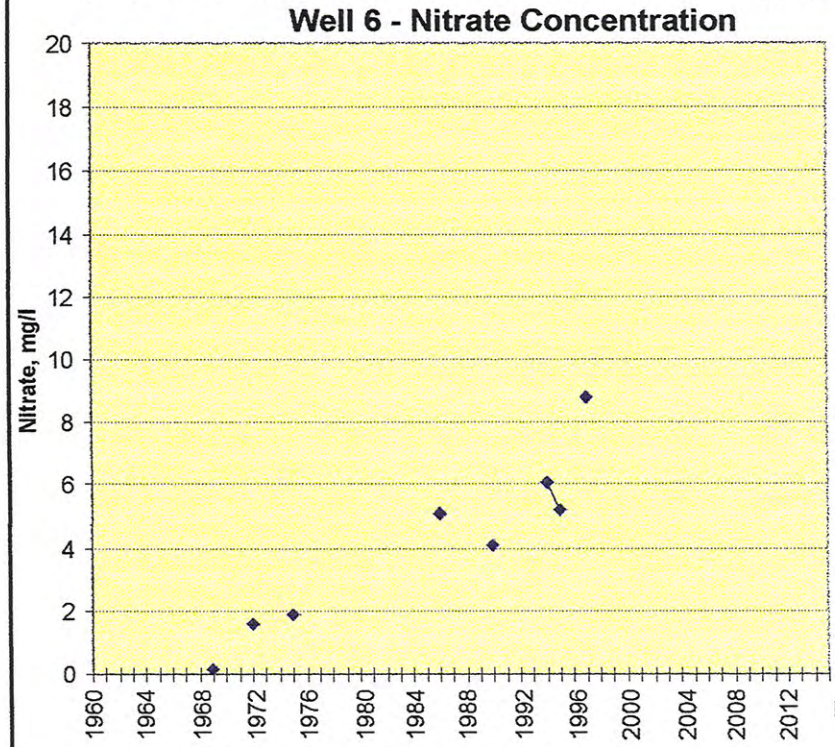

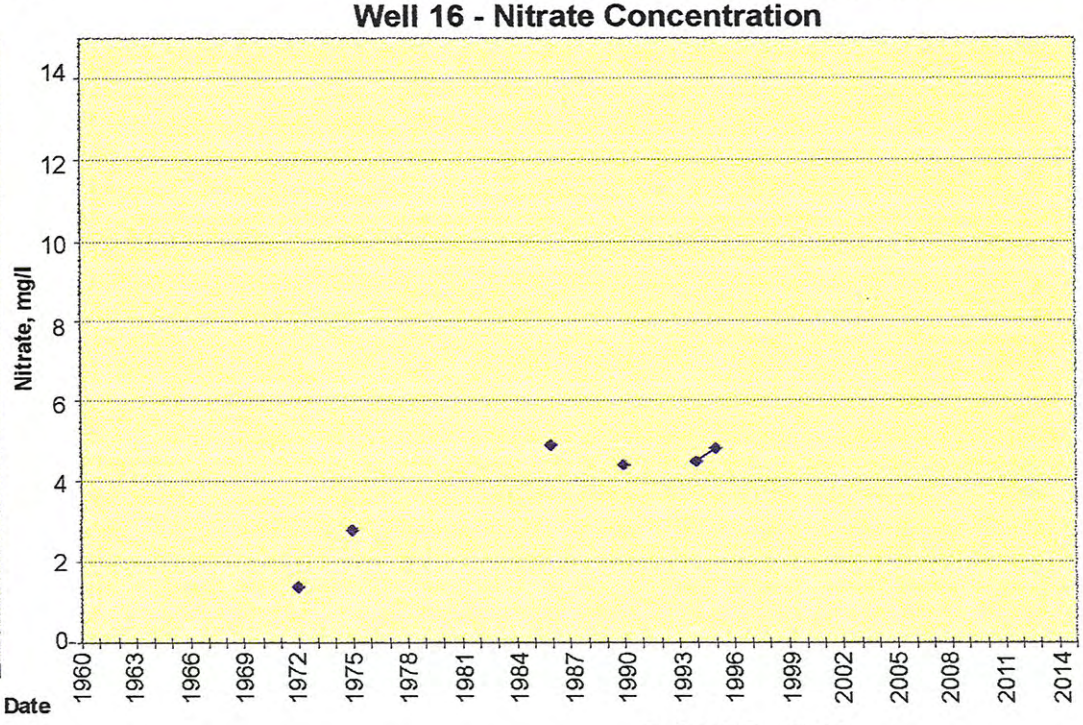

Nitrate Concentration at Selected Hastings Utilities Production Wells

FIGURE: 2.12

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Section 3

Regional Groundwater Model

3.1 Introduction and Code Selection

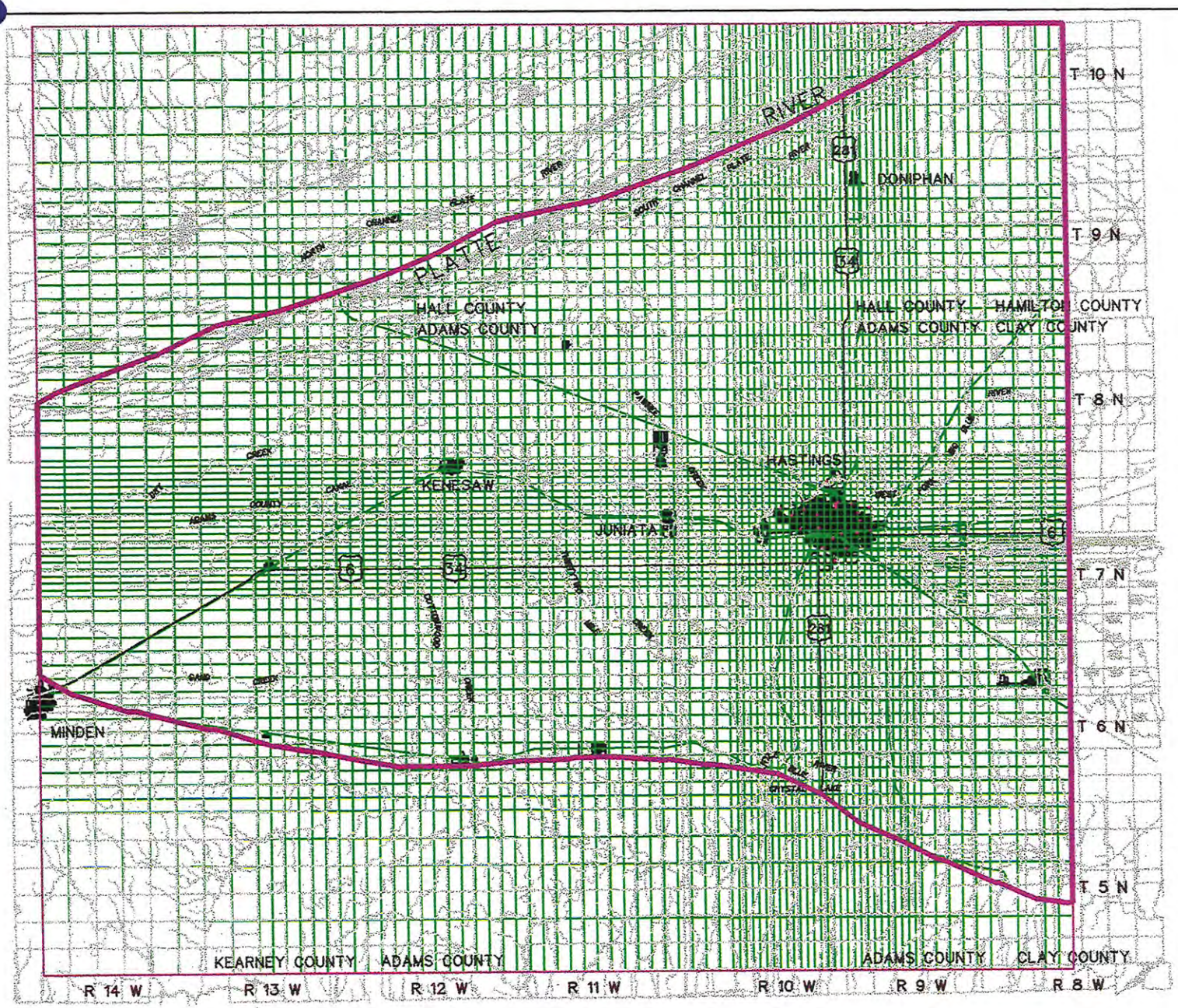
The overall modeling approach for the present study involved the development of a regional groundwater flow model, a local groundwater flow model, and a groundwater particle tracking simulator. The regional model, discussed in this section, was developed to estimate the apparent aquifer safe yield. The large areal extent of the regional model (Fig. 1.1) was required to ensure that the impacts of groundwater development in the area were adequately assessed in the primary area of interest, that is the HU wellfield. Furthermore, the negative impacts of boundary uncertainties are minimized by extending the model domain to natural boundaries further away, such as the Platte River and the groundwater divide south of Hastings.

As discussed in section 2.2, the Pleistocene aquifer was modeled as a single layer unconfined unit. The flow model was constructed using the USGS three-dimensional modeling code MODFLOW (McDonald and Harbaugh, 1988). MODFLOW is designed to simulate steady-state or transient groundwater flow through heterogeneous, anisotropic porous media in three dimensions. It uses a modular method of data entry to simulate various aspects of the flow system, such as: pumping wells, rivers, recharge, evapotranspiration, aquifer characteristics, and boundary conditions. The aquifer is represented in the model by a series of grid cells. Information about the aquifer characteristics such as aquifer thickness, hydraulic conductivity, storativity, recharge, etc. are input to each model cell. This enables specifying heterogeneous conditions to the model by varying the value of a particular parameter from cell to cell.

3.2 Regional Model Grid and Boundary Conditions

The regional study area was discretized by 66 rows and 80 columns representing 5280 cells, of which 4186 cells lie in the active model domain (Fig. 3.1). Grid spacing varies from 1320 feet to 10560 feet. A finer discretization was specified near the HU well sites in order to obtain reliable estimates of drawdowns due to municipal pumpage.

Model boundary conditions are presented in Figure 3.2. The northern boundary coincides with the Platte River which is hydraulically well connected with the Pleistocene aquifer (Lappala, 1979). A no-flow boundary condition in the subsurface was therefore specified along this boundary since the Platte river forms a hydraulic barrier with negligible baseflow occurring across the river. Stream-aquifer interactions along the Platte River were simulated by the River module of MODFLOW according to the following equation:



Regional Model Grid

FIGURE: 3.1

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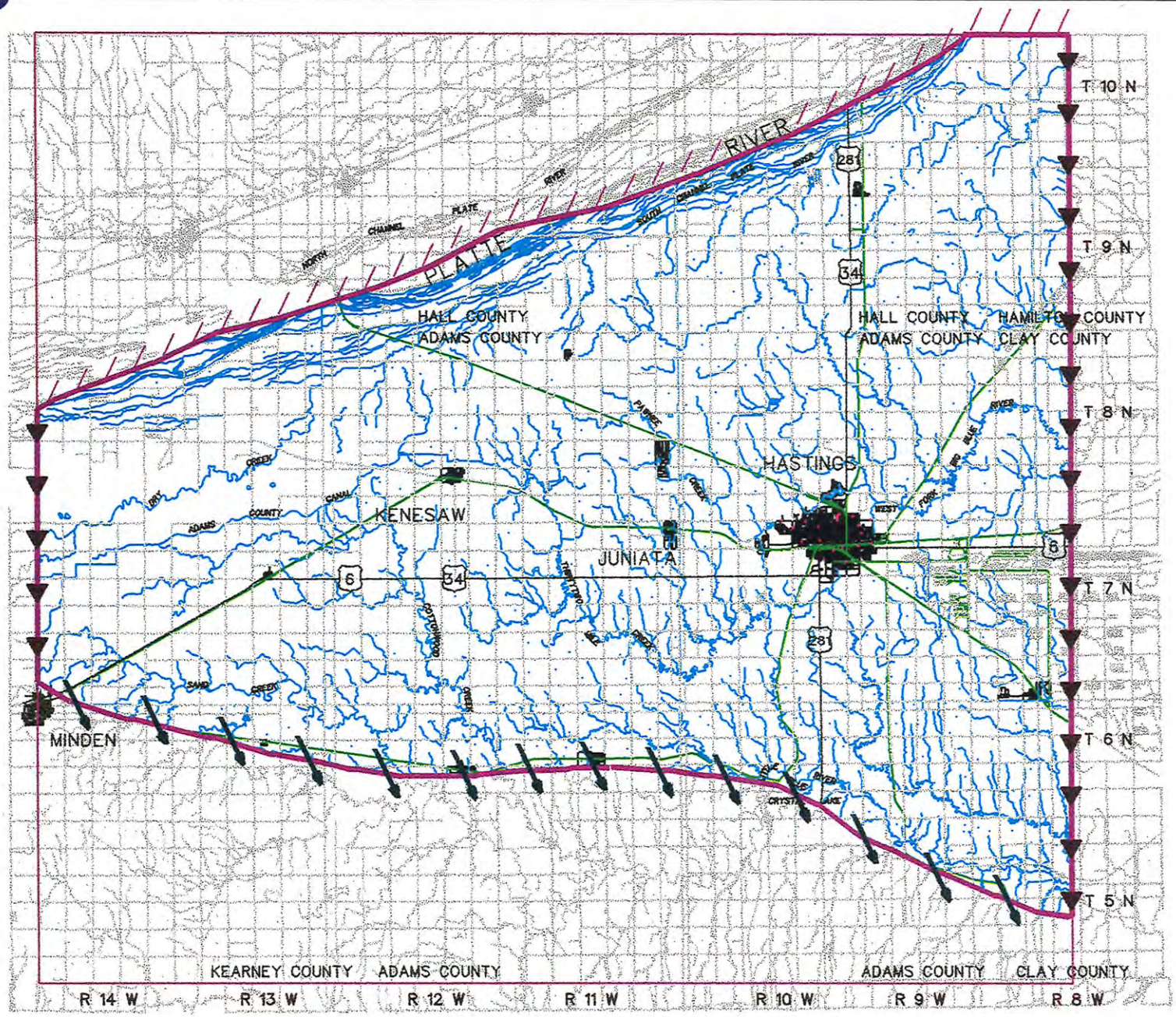

Regional Model Boundry Conditions

FIGURE: 3.2

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$$Q = CD(Hs - Ha) \tag{3.1}$$

where,

Q = Flow between the aquifer and the Platte River (ft^3/day)
Hs = Hydraulic head elevation in the Platte River (ft)
Ha = Hydraulic head elevation in the aquifer (ft)
CD = Hydraulic conductance of the riverbed (ft^2/day)

Since the stage of the Platte River is fairly shallow, the hydraulic head in the Platte River was specified from USGS topographic maps (scale; 1:240000). The riverbed conductance, which is a measure of the hydraulic connection between the river and the aquifer, is equal to the product of the grid cell area and the riverbed hydraulic conductivity divided by the riverbed thickness. The conductance was specified to each river cell based on a conductivity value of 6.0 ft/day derived in a previous study (Ted Zorich & Associates, 1989), and a riverbed thickness of 1 feet estimated for the Platte River near Omaha (Layne GeoSciences, 1996). Sensitivity simulations indicated that the water levels in the aquifer were insensitive to variations in this parameter. Increasing and decreasing the riverbed conductance by 50 percent resulted in negligible changes in the simulated water levels. Since the simulated water levels under the Platte River are similar in magnitude to the stage of the river, the Platte River and the aquifer appear to be very well connected. This is in agreement with conclusions from a previous study (Lappala, 1979), and can also be deduced from maps depicting changes in groundwater levels (Figures 2.7a and 2.7b).

Constant heads were prescribed along the western boundary of the model domain. As discussed below in section 3.3, the model was first calibrated to (1979) postdevelopment conditions and then validated to predevelopment conditions. For post-development simulations, heads along this boundary were specified from the hydraulic heads distribution for 1979 presented in Figure 2.4. The predevelopment heads along this boundary were specified by subtracting post-development heads based on the magnitude of change specified in Figure 2.7a (changes in water level conditions from predevelopment to 1982). As discussed in section 2.3, water levels have remained fairly stable during the past decades. Therefore, 1979 and 1982 conditions are assumed to represent general postdevelopment conditions.

The southern boundary of the model domain is approximately along a groundwater divide formed by discharge of groundwater into the Little Blue River and its tributaries in the area. During model calibration it became obvious that a small amount of groundwater flow was exiting the study area across this boundary. A model-calibrated discharge of approximately 10 MGD was specified along this boundary. As discussed in section 3.4, compared to the total quantity of estimated groundwater moving into and out of the model domain (318 MGD for postdevelopment conditions and 200 MGD for predevelopment conditions), this is a relatively minor quantity of groundwater flux.

A constant head boundary was also specified along the eastern boundary. Similar to the implementation along the western boundary, the 1979 hydraulic heads were specified for post

development simulations. For predevelopment conditions, the heads along this boundary were raised according to the magnitude of change in water levels from predevelopment to 1982 (refer to Figs. 2.7a).

The major streams in the study area (Fig. 1.1) were simulated by the Stream module of MODFLOW (Prudic, 1989). The stream-aquifer interaction in this module is represented by equ. 3.1. The stream head, *Hs* (which is the sum of the stream stage and the riverbed elevation), is calculated according to the following equation for flow in open channels:

$$d = \left[\frac{Qn}{CwS^{1/2}} \right]^{3/5}$$

where,

d=Stream stage, (ft)
Q=Stream discharge, (ft^3/day)
n=Manning's roughness coefficient, (dimensionless)
C=Constant (1.486)
w=width of channel, (ft)
S=Slope of stream channel, (ft/ft)

A streambed slope of 0.05 was specified based on USGS topographic maps of the area, while 0.035 was specified for the Manning's roughness coefficient. This is a representative value of the roughness coefficient for streams in the Plain States (Sophocleous, 1997). Sensitivity simulations indicated that the model results were not sensitive to variations in this parameter.
elected Hastings Utilities Production Wells

3.3 Model Calibration and Validation

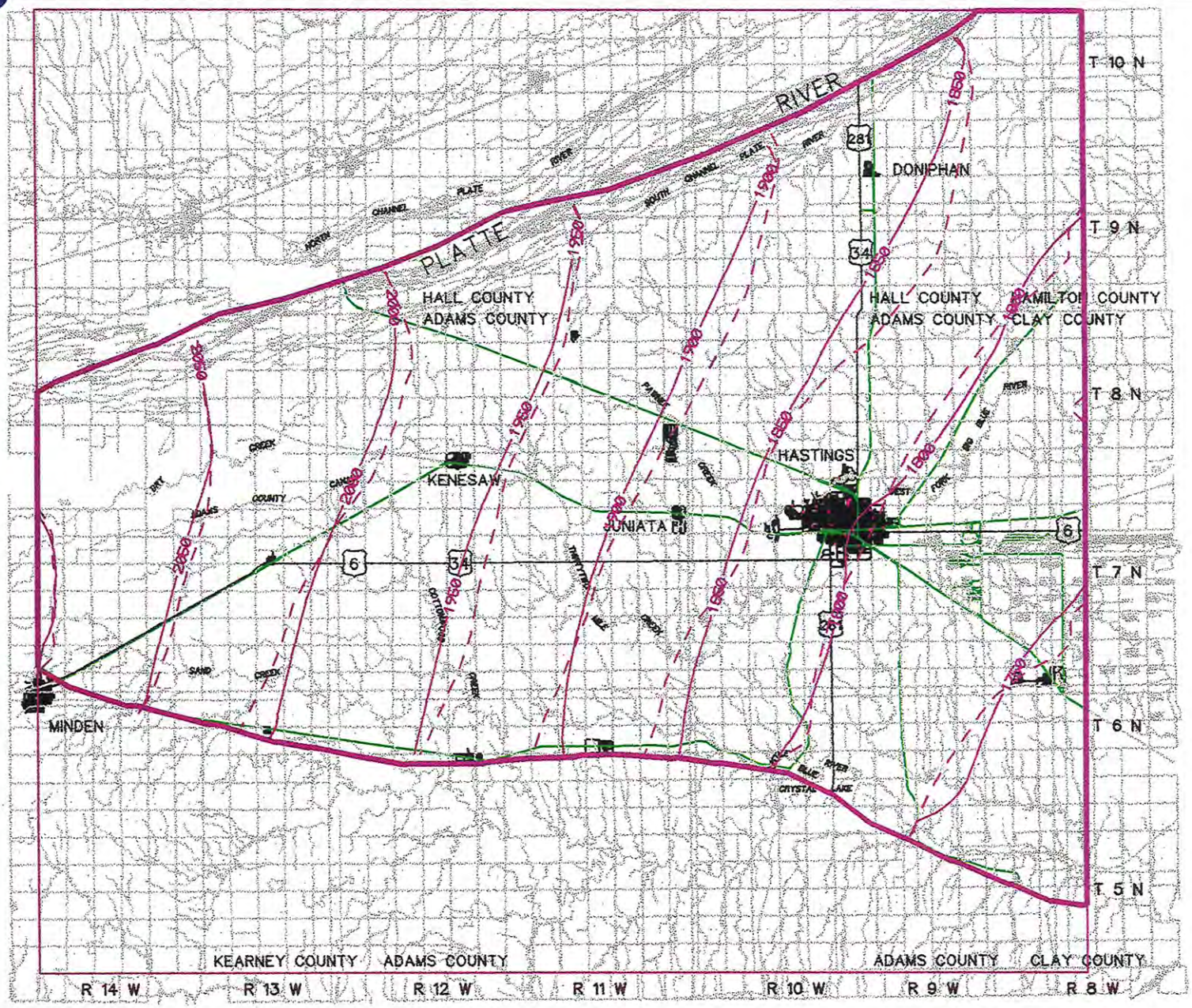
The process of model calibration involves adjusting model input parameters within reasonable ranges until the model output (water levels) replicate observed field conditions. The 1979 potentiometric surface contour map (Fig. 2.4) developed for a previous study (USGS, 1980) was established as the calibration target. As discussed in section 2.3, the 1979 potentiometric surface is expected to be similar to present day conditions and can be considered as representative of average postdevelopment conditions.

The aquifer hydraulic conductivity, recharge, and average groundwater pumpage for irrigation purposes were the primary model calibration parameters. The river/stream hydraulic parameters and the aquifer base (Fig. 2.1) were specified initially and not changed during the calibration process. Numerous simulations were conducted, in which the calibration parameters were varied until the simulated water levels matched the observed water levels satisfactorily. The average postdevelopment water levels simulated by the calibrated regional model is presented in Figure

3.3. The interpreted postdevelopment water level surface is also presented in this figure for comparison purposes. A satisfactory match between the observed and simulated water levels can be inferred from this figure. Given the expected variations in a contoured water level surface due to the particular interpolation scheme employed, and natural fluctuation of water levels on a seasonal and annual basis, the degree of match achieved is deemed adequate. In the immediate vicinity of Hastings, the simulated water levels are approximately five feet lower than the interpreted 1979 levels. This is the approximate difference between the 1979 and present-day water levels in the hydrographs presented in Figures 2.6a-e and Appendix A.

The calibrated values of the hydraulic conductivity is presented in Fig. 3.4. A value of 230-300 ft/day was specified in southern portions of the model domain. In the northern portions of the study area, a higher value (300-490 ft/d) was required for model calibration. The model calibrated values are within the range obtained from pump tests in the area (Geraghty & Miller, 1994). Simulations were conducted to assess the sensitivity of the hydraulic conductivity on the model results. The simulated water levels with hydraulic conductivity increased and decreased 25 percent from the base calibration case is presented in Figure 3.4a. Increasing the hydraulic conductivity results in water levels to increase by up to 7 feet. Decreasing the hydraulic conductivity results in a water level drop of less than 5 feet. Along the Platte River and the Little Blue River, water levels remain constants due to the good connection between these surface water features and the aquifer.

The calibrated (postdevelopment) average annual recharge distribution is presented in Figure 3.5. In the middle and southern portions of the study area, the effective recharge varies from 2 to 3 inches per year. In general, recharge rates vary with depth to the water table. In the central and southern portions of the study area, the effective recharge is low since depth to the water table exceeds 100 ft in the area. Along the Platte River valley in the northern portions of the study area, the calibrated recharge values are relatively higher (6 to 8 in/yr). This is due to the shallower depth to the water table in these areas. A higher effective recharge was also required during model calibration along the creeks in the study. This is a likely consequence of surface water runoff and infiltration from the creeks into the subsurface following high precipitation events. The recharge distribution also conforms with the soils configuration in the area. A relatively higher recharge rate was specified along the sandy soils west of Kenesaw (Keech and Dreeszen, 1968) and in the Platte River valley. Simulations were conducted to assess the sensitivity of effective recharge on the model results. The simulated water levels with recharge increased and decreased by 25 percent from the base calibration case is presented in Figure 3.5a. Increasing recharge results in a water level increase of up to 10 feet. Decreasing the effective recharge results in a water level drop of up to 5 feet.



SIMULATED GROUNDWATER LEVELS

INTERPRETED 1979 GROUND WATER LEVELS

Water Level Surface
in the Primary Aquifer
Under Developed Conditions (ft,amsl)

FIGURE: 3.3

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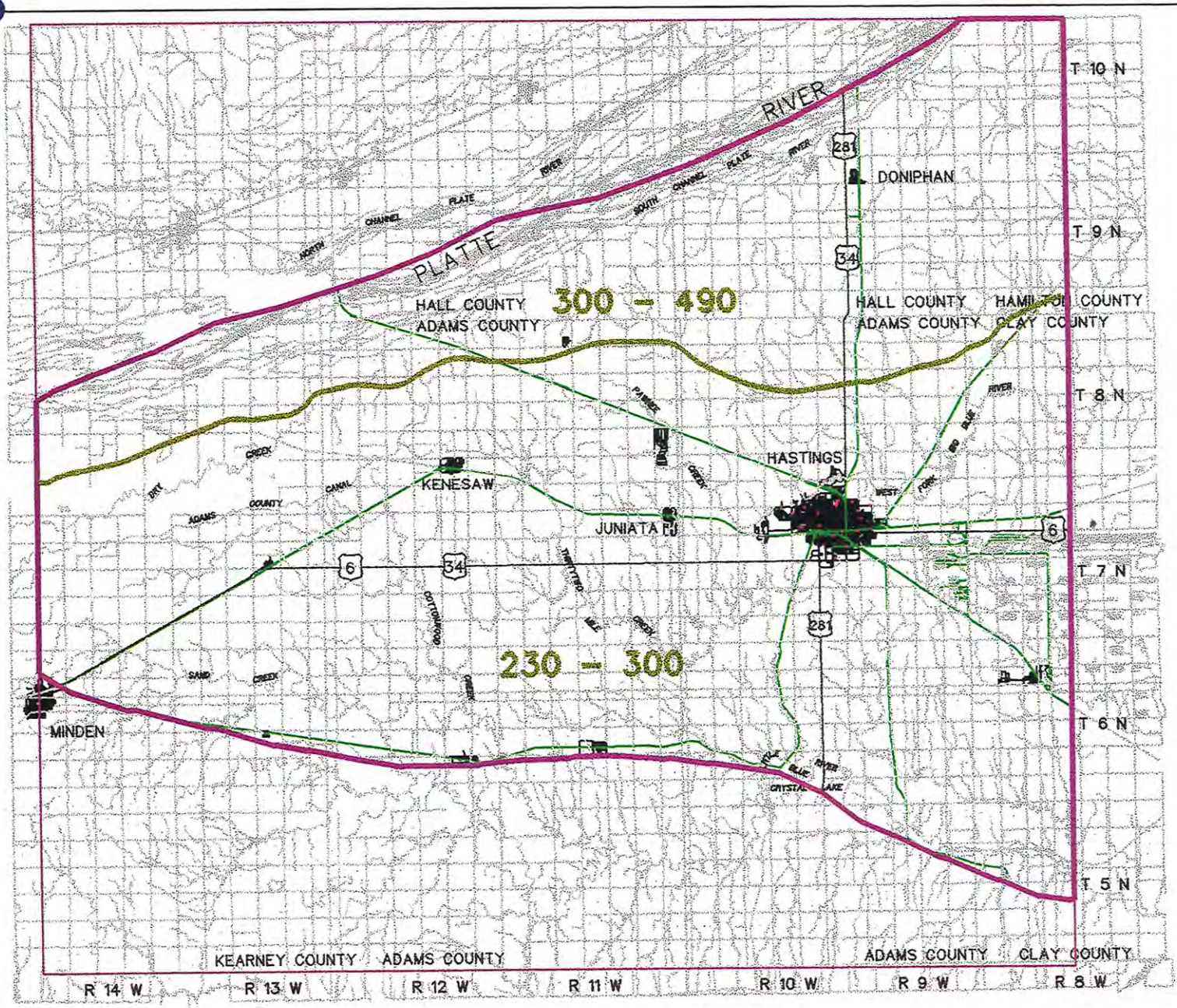
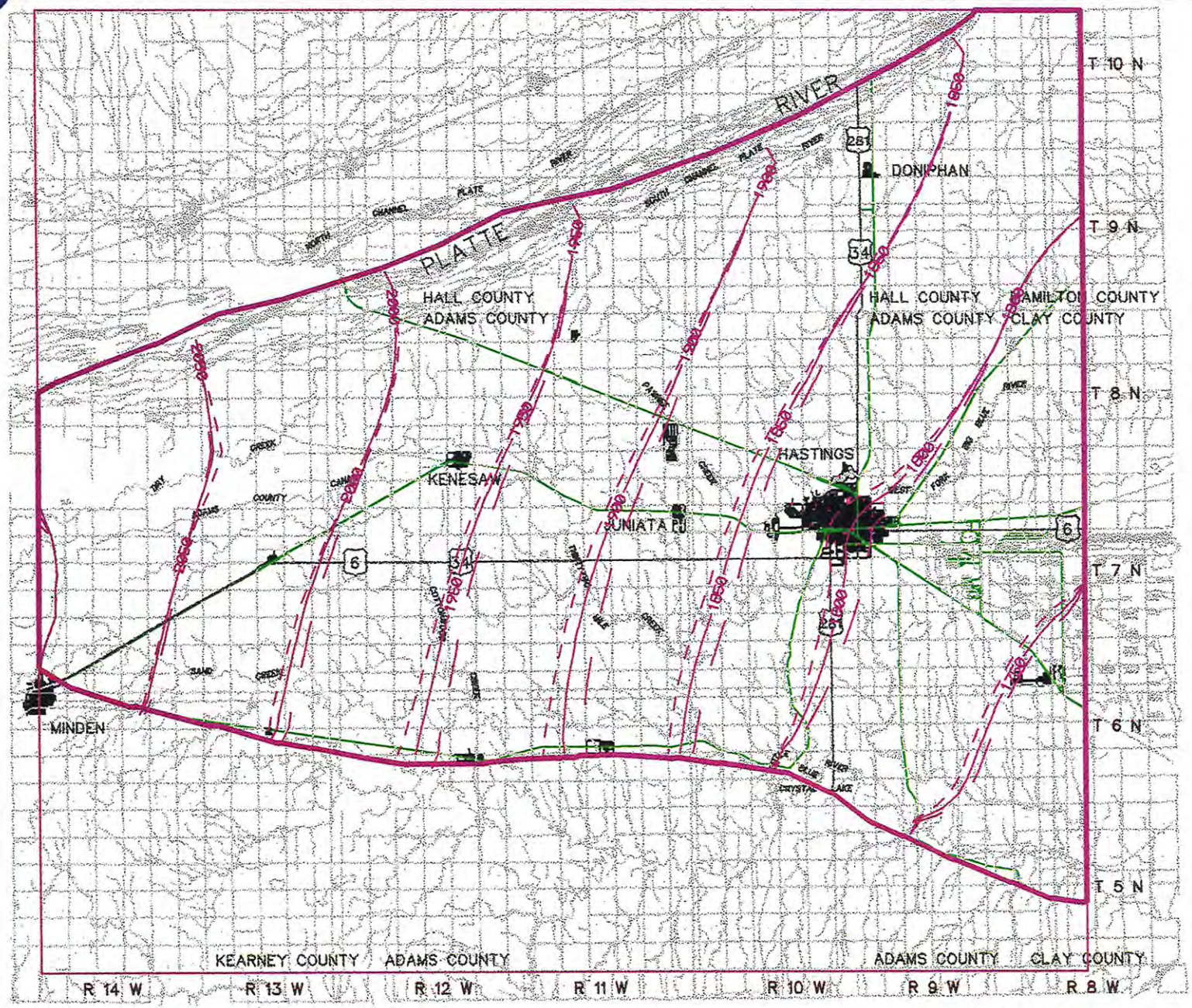


Calibrated Hydraulic Conductivity Distribution (ft/day)

FIGURE: 3.4

Layne GeoSciences, Inc.



CALIBRATION RUN

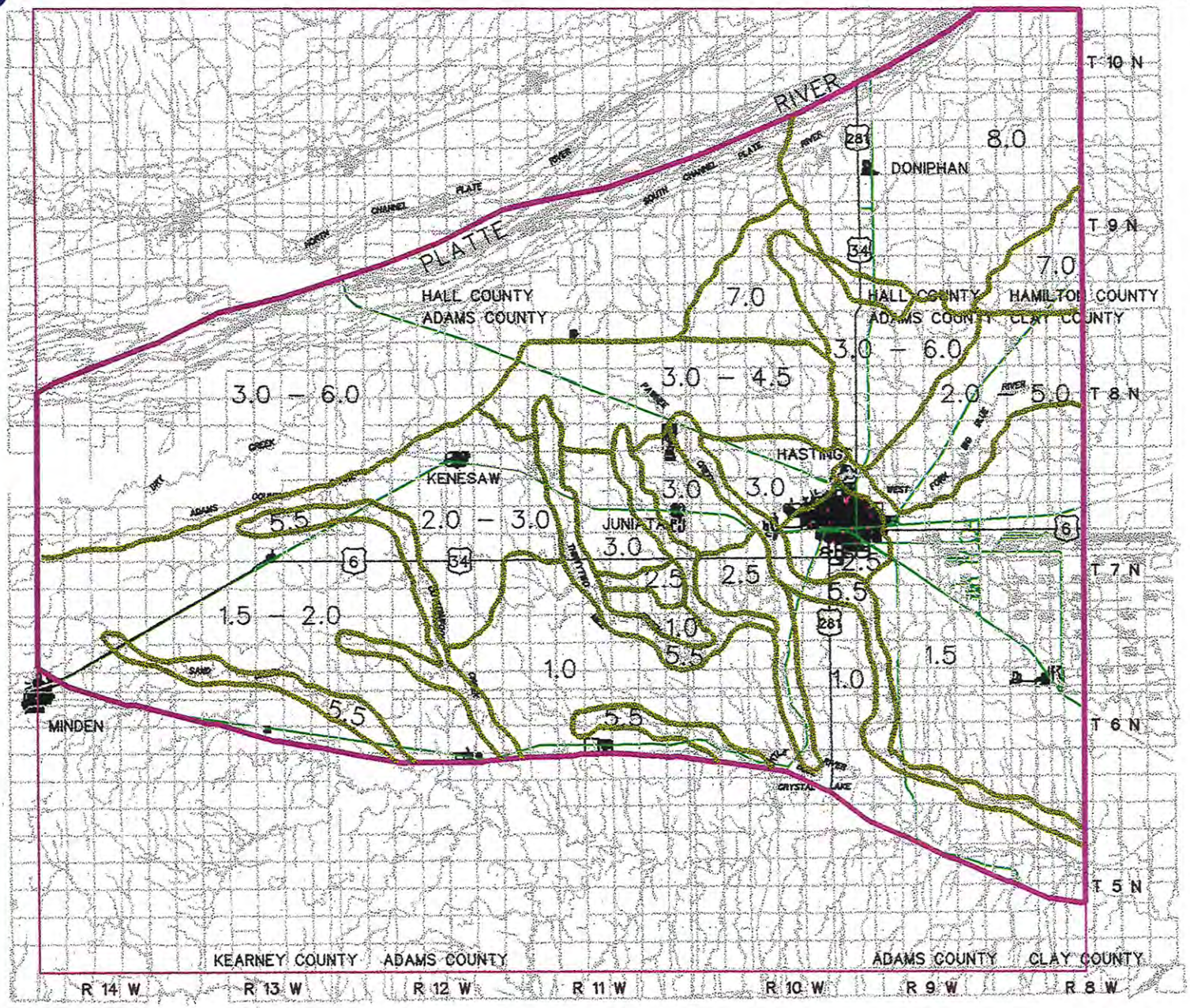
HYDRAULIC CONDUCTIVITY INCREASED 25%

HYDRAULIC CONDUCTIVITY DECREASED 25%

Simulated Water Levels with Hydraulic Conductivity Increased and Decreased 25 Percent

FIGURE: 3.4a

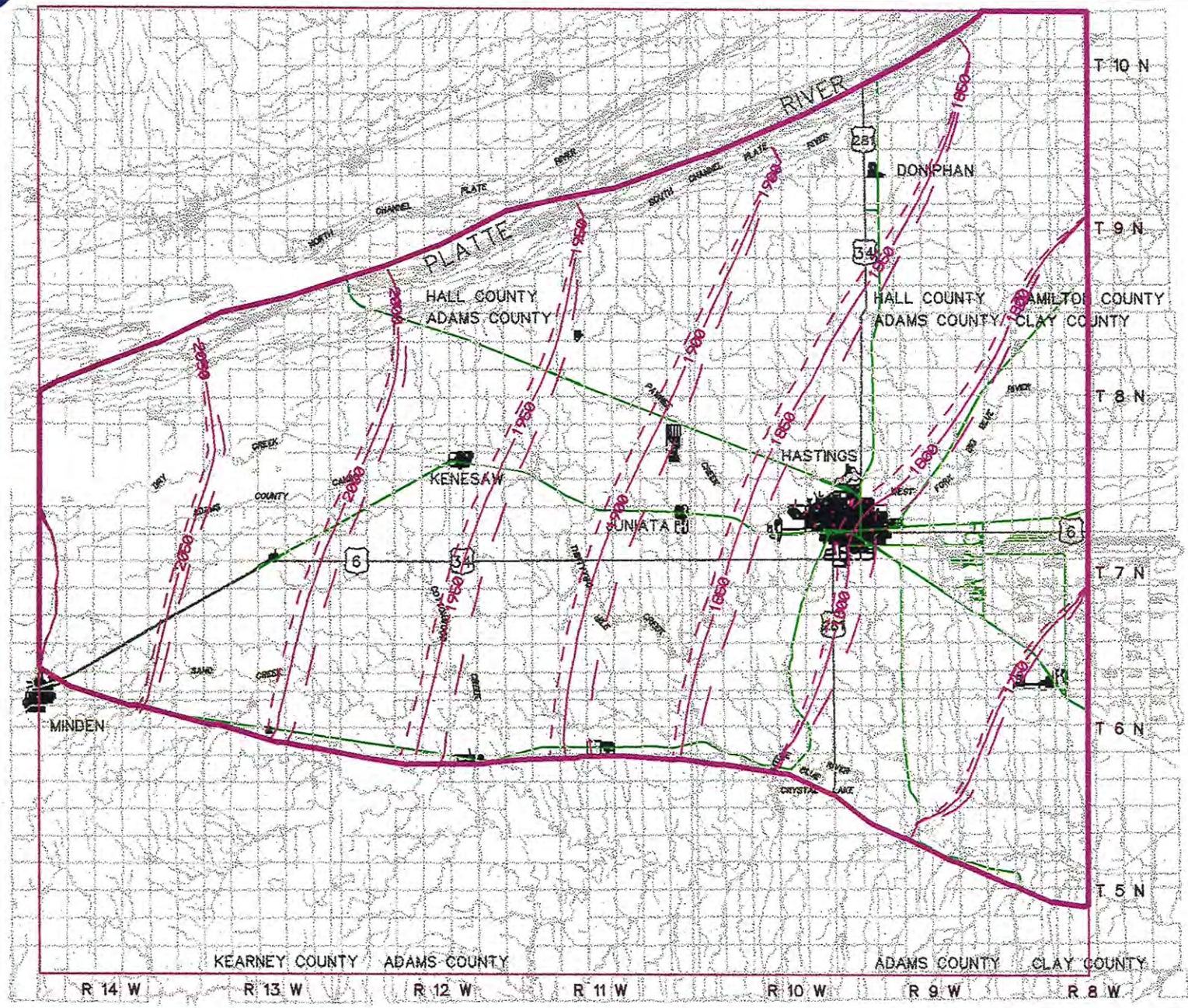
Layne GeoSciences, Inc.



Calibrated Average Annual Recharge
Under Developed Conditions (in/yr)

FIGURE: 3.5

Layne GeoSciences, Inc.



CALIBRATION RUN

EFFECTIVE RECHARGE INCREASED 25%

EFFECTIVE RECHARGE DECREASED 25%

Simulated Water Levels with Effective Recharge Increased and Decreased 25 Percent

FIGURE: 3.5a

Layne GeoSciences, Inc.

Average 1976-78 municipal pumpage in the HU wellfield (Table 3.1) were specified for calibration purposes. Additionally, industrial pumpage of 0.5, 1.5, 1.5, and 1.1 MGD were specified at the HEIP Industries, Chief Ethanol, Farmland Industries, and the Hastings Energy Center sites east of Hastings. The industrial pumpage rates were obtained from data in a previous study (Wood-Ward Clyde, 1990).

As discussed in section 2.7, the primary withdrawal in the area is for irrigation purposes. Due to the fact that reporting water use for irrigation purposes is not mandatory, it is difficult to estimate water use from reported data. For example, estimating water use from issued permits is unreliable since, " some permit application pumping rates are highly inaccurate, showing as much as 2000 gpm extracted from 8 inch wells or 2000 gpm applied to a little over 50 acres (4.5 gpm will provide 18 inches of water for one acre during a 76-day irrigation season) " (Woodward-Clyde, 1990). Irrigation withdrawal was therefore considered a variable during the calibration process. The calibrated values of this parameter were compared with estimated ranges as described below.

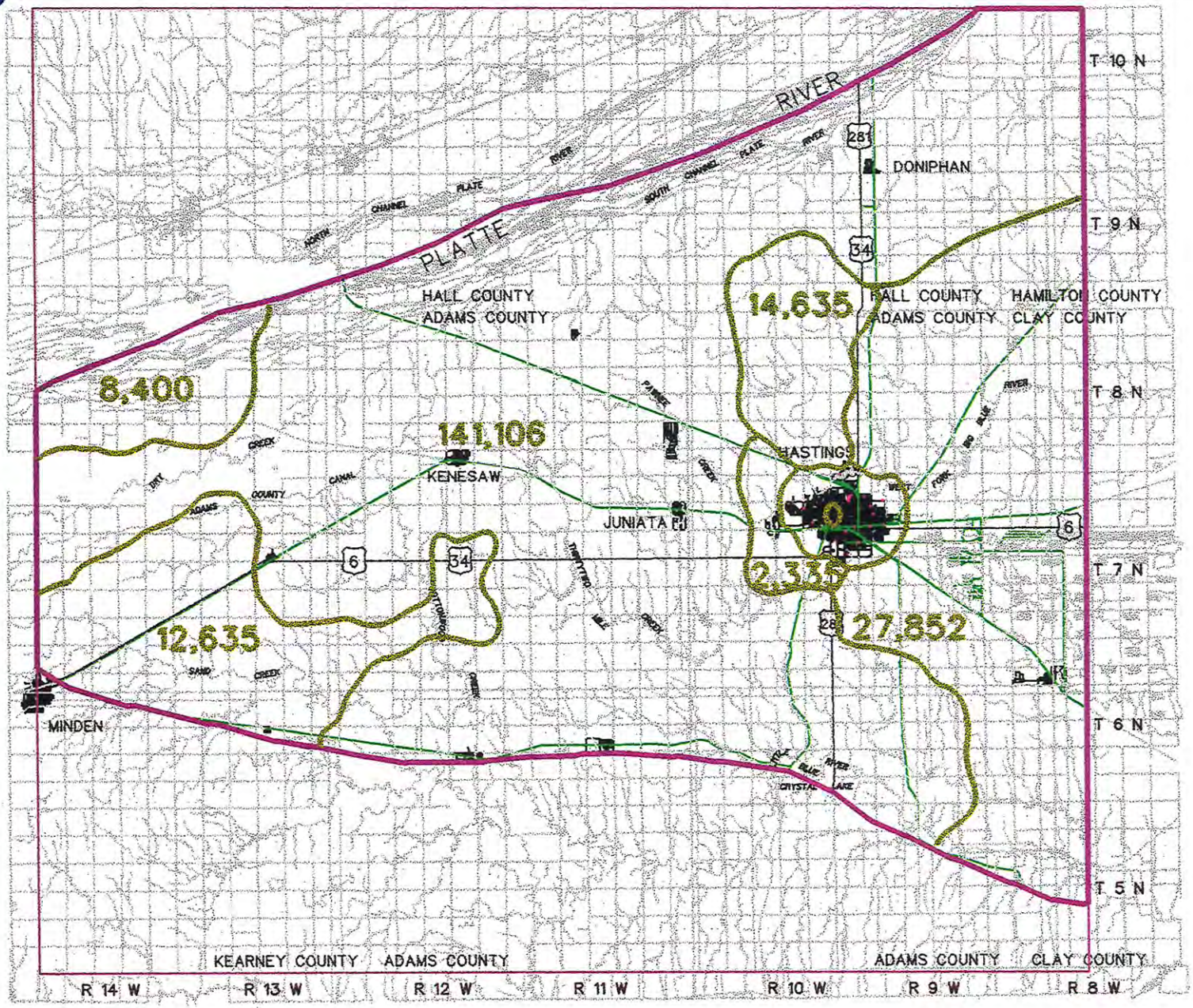
The average annual irrigation demand is estimated to be approximately 12 inches for land under irrigation (refer to section 2.7). The percent of irrigated land in the area varies considerably (Fig. 2.9), but can be grouped into seven land zones as indicated in Figure 2.10. This zonation was also maintained in the flow model. The amount of withdrawal in each zone was modified independently during the calibration process. Since the extent of the model area entailed grid spacing of 1/4 mile to 2 miles, irrigation withdrawals were specified as a uniformly distributed rate in each zone. The final calibrated irrigation withdrawals are presented in Figure 3.6 as the total quantity extracted on an average annual basis in each of the seven irrigation zones. An alternate representation of this quantity, in terms of inches of water withdrawn per year, is presented in Figure 3.7 assuming uniform withdrawal in each irrigation zone.

Total acreage under irrigation in Adams County for 1985 was estimated at 164,960 acres (Steele, 1989). The total area of Adams County is approximately 355,817 acres. This implies approximately 46 % of the land in Adams County is irrigated. For the present study, approximately 254,114 acres of the land in Adams County is included in the groundwater model. The total groundwater withdrawal for irrigation in Adams County simulated by the model is approximately 104,980 acre-feet. Assuming an average irrigation withdrawal of 12 inches/year, the model estimated area under irrigation in Adams County is therefore approximately 42 percent. This is fairly close to the 46 percent estimate derived above. If an average withdrawal of 11 inches/year is assumed, 45 percent of the land is estimated by the model to be under irrigation.

Table 3.1

Average Annual Withdrawal at HU Production Wells for the Period 1976-78

Well No.	Average 1976-78 Annual Withdrawal
	(Gallons)
1	0
2	0
3	96,564,266
4	81,145,833
5	0
6	69,088,633
7	67,014,766
8	240,601,633
9	298,273,541
10	42,001,900
11	95,966,333
12	402,257,000
13	0
14	55,543,600
15	108,107,066
16	353,502,166
17	0
18	0
19	0
20	0
21	0
22	0
23	0
24	0



Model Estimated Annual Average Irrigation Withdrawls (ac-ft/yr)

FIGURE: 3.6

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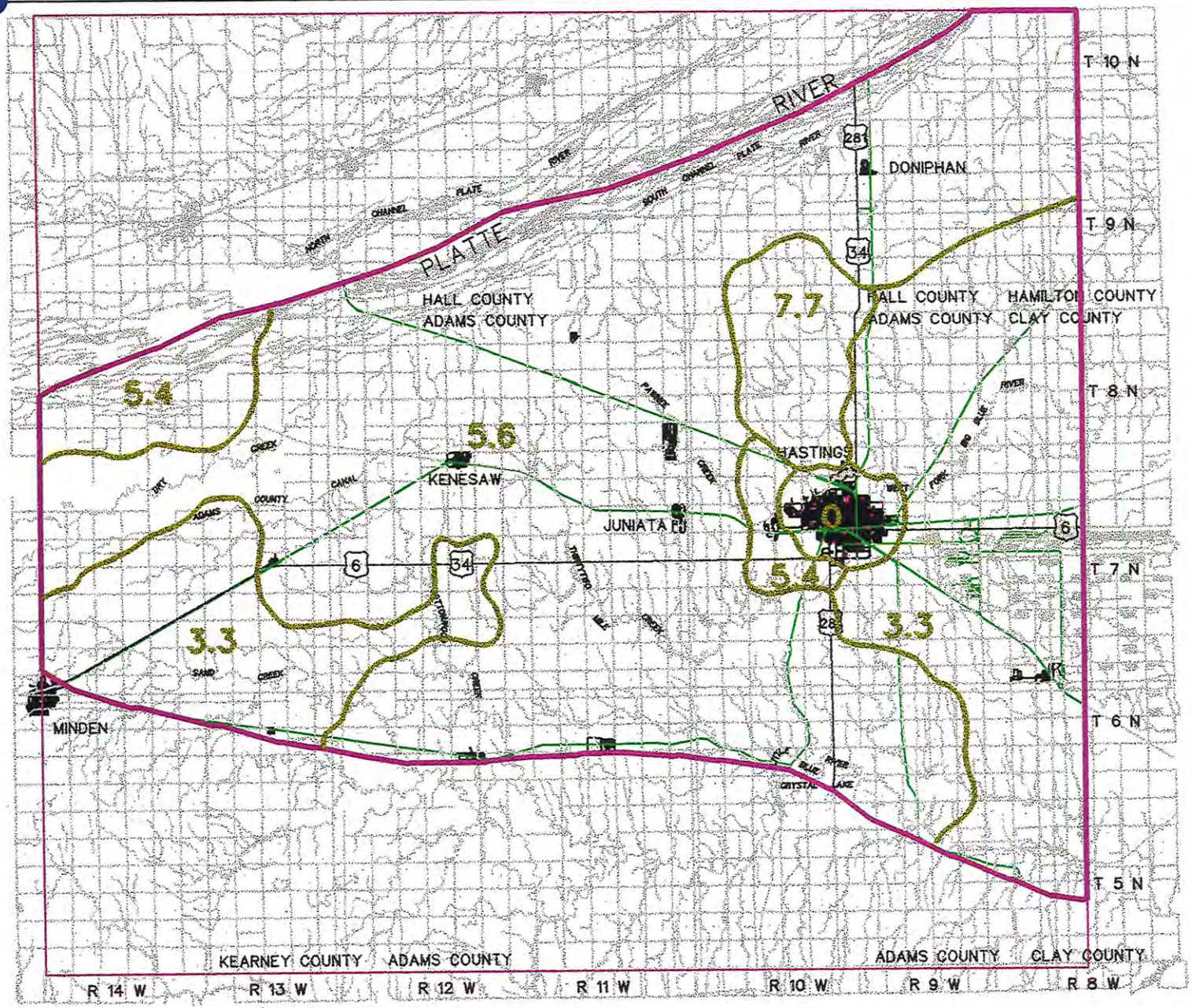

Model Estimated Average Annual Water Use for Irrigation (in/yr)

FIGURE: 3.7

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The calibrated model was next validated to predevelopment water level conditions. The process of model validation is an important step in model construction because the parameters obtained during model calibration may not constitute a unique set of parameters and stresses. Reproducing a hydrologic state for a period other than the calibration period by varying the hydrologic stresses (e.g. pumping), provides an independent check on the predictive capabilities of the model. Since information on predevelopment water level conditions was available in the form of changes in water level since development (Figs. 2.7a and 2.7b), the predevelopment water level state was chosen for model validation purposes.

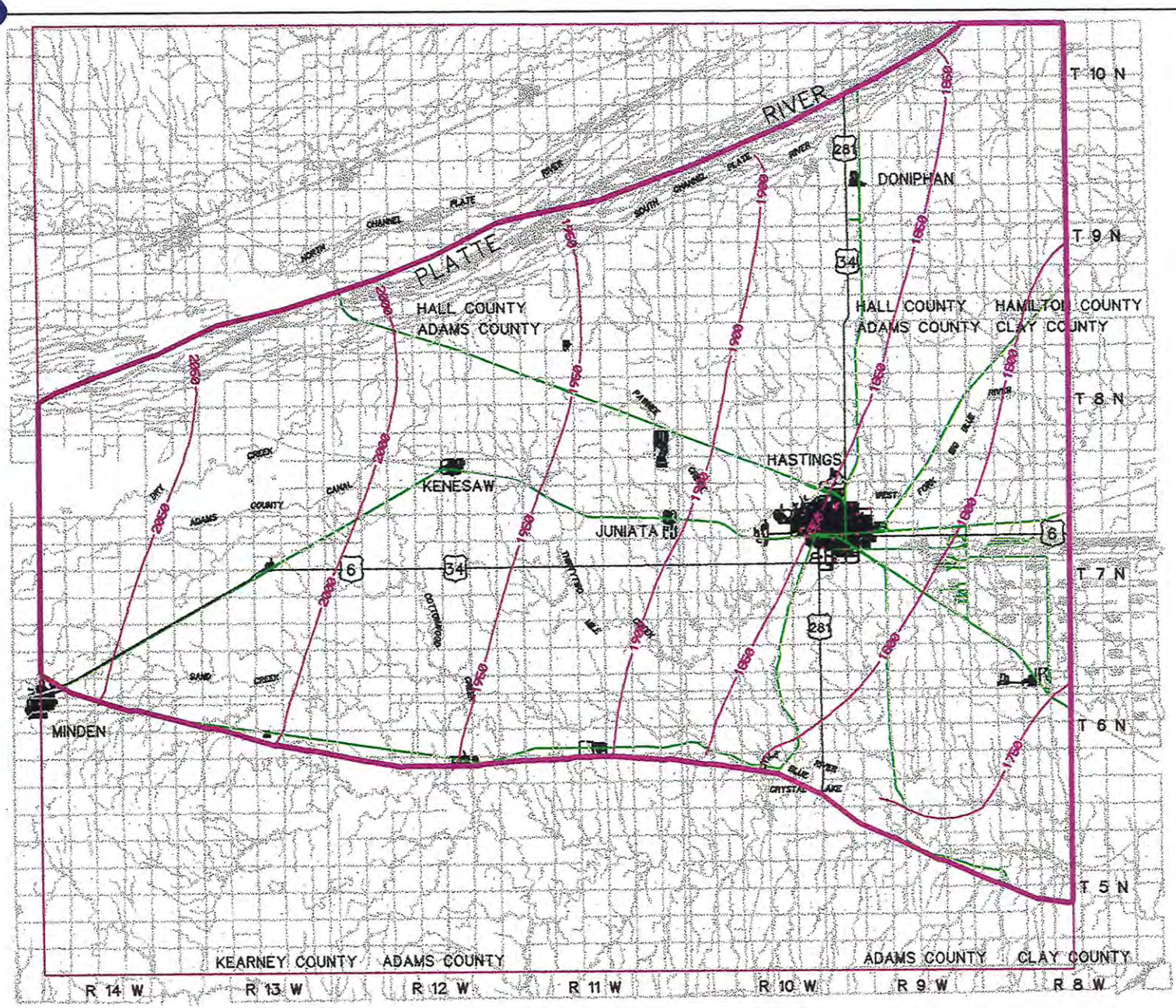
Predevelopment stresses were represented in the model by eliminating all groundwater withdrawals in the study area. The resulting predevelopment water levels are presented in Figure 3.8. The change in the simulated predevelopment and postdevelopment water levels is presented in Figure 3.9. This change is compared with observed changes for 1982 and 1990 presented in Figures 2.7a and b. The simulated changes between predevelopment and postdevelopment conditions (Fig. 3.9) compare favorably with the observed changes (Fig. 2.7a and b). A mound of approximately 20-30 feet is noted near Minden along the western boundary, while a decline of over 30 feet is noted around Hastings. Along the alluvial valley of the Platte River, water levels have remained fairly unchanged due to recharge from the Platte River.

The model simulated decrease in water levels of over 30 feet at Hastings is larger than the 25 feet observed drop (Figs. 2.7). This is attributed to the relatively low hydraulic conductivity (230 feet/day) specified to the model in the vicinity of Hastings. It is likely that hydraulic conductivity is higher in this area. However, a relatively low value of this parameter was specified in order to obtain conservative estimates of drawdown due to additional withdrawals simulated in the development scenarios.

It should be noted that in addition to eliminating both irrigation and municipal withdrawals for predevelopment conditions, a lower recharge was also specified for predevelopment conditions. The overall recharge is lower by approximately 25 percent over postdevelopment rates. An increase in recharge with development was also noted in previous studies (Boyle; 1983, Woodward-Clyde;1990) and is a consequence of irrigation return flow. Field recharge studies would be helpful towards establishing the magnitude and distribution of this parameter more firmly. In this study, all calibration parameters were varied within reported and documented ranges in order to obtain a good match with the observed water level data. Results of future field studies aimed at better characterizing the aquifer should be incorporated in the model in order to further improve its predictive capabilities.

3.4 Mass Balance Changes from Predevelopment to Postdevelopment Conditions

Key hydrologic mass balance components for average predevelopment and postdevelopment conditions are presented in Table 3.2. The total groundwater flux into and out of the study area has changed significantly since predevlopment. Approximately 200 MGD was entering and exiting the model area on an average daily basis under predevelopment conditions. With



Simulated Predevelopment Water Levels (ft, amsl)

FIGURE: 3.8

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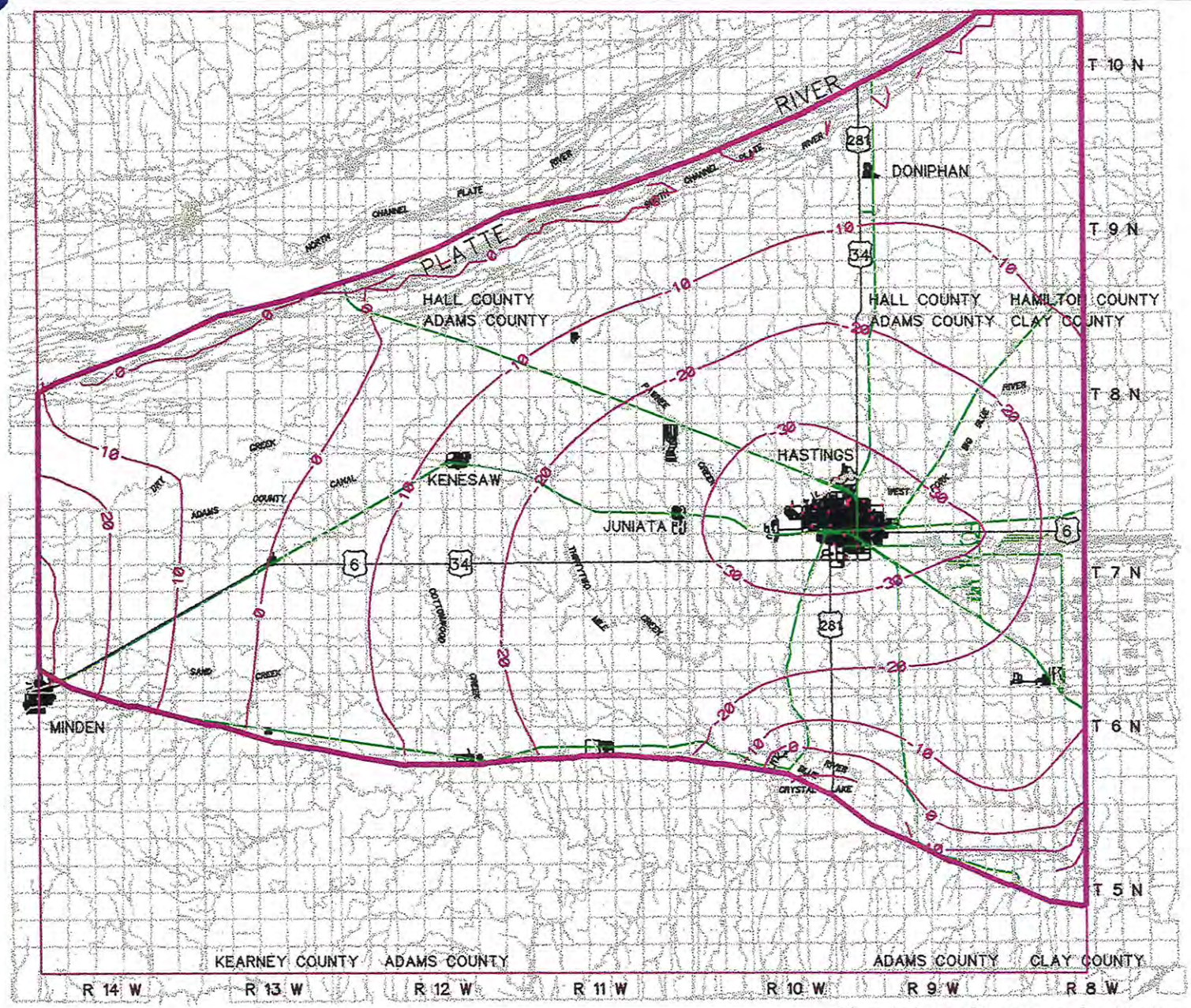

-30 WATER LEVEL DROP WITH DEVELOPMENT

+20 WATER LEVEL INCREASE WITH DEVELOPMENT

Simulated Changes in Water Levels from Predevelopment to Present–Day Levels (ft)

FIGURE: 3.9

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development, this has increased to approximately 318 MGD. This is primarily due to increased stress on the system due to irrigation withdrawals which in turn has induced a greater flux in the groundwater system. Approximately 118 MGD of recharge entered the system from surface infiltration prior to development. For developed conditions this quantity has increased to 154 MGD. Induced recharge from the Platte River has increased from 55 MGD to 79 MGD. Along the western boundary, approximately 25 MGD entered the system prior to development, while presently approximately 51 MGD enters the study area on an average daily basis. This increase is primarily due to seepage of water from the Phelps Canal. Groundwater flow out of the study area through the eastern boundary has decreased from 131 MGD under predevelopment conditions to 96 MGD with development in the area. The amount of water discharging into streams has decreased from 54 MGD to 15 MGD. Lost Creek in the northwest along the Platte River however appears to have gained flow with development due to groundwater mounding within and further west of the study area. This accounts for the increase in stream flow with development. The total pumpage from the aquifer with development is approximately 193 MGD, out of which approximately 182 MGD is for irrigation purposes.

Table 3.2

Model Water Budget

Component	Predevelopment (MGD)	Postdevelopment (MGD)
INFLOW		
Recharge	117	154
Platte River	55	79
Western Boundary	25	51
Streams	3	34
OUTFLOW		
Southern Boundary	10	10
Pumpage		193
Platte River	5	4
Eastern Boundary	131	96
Streams	54	15
TOTAL	200	318

3.5 Groundwater Development Scenarios

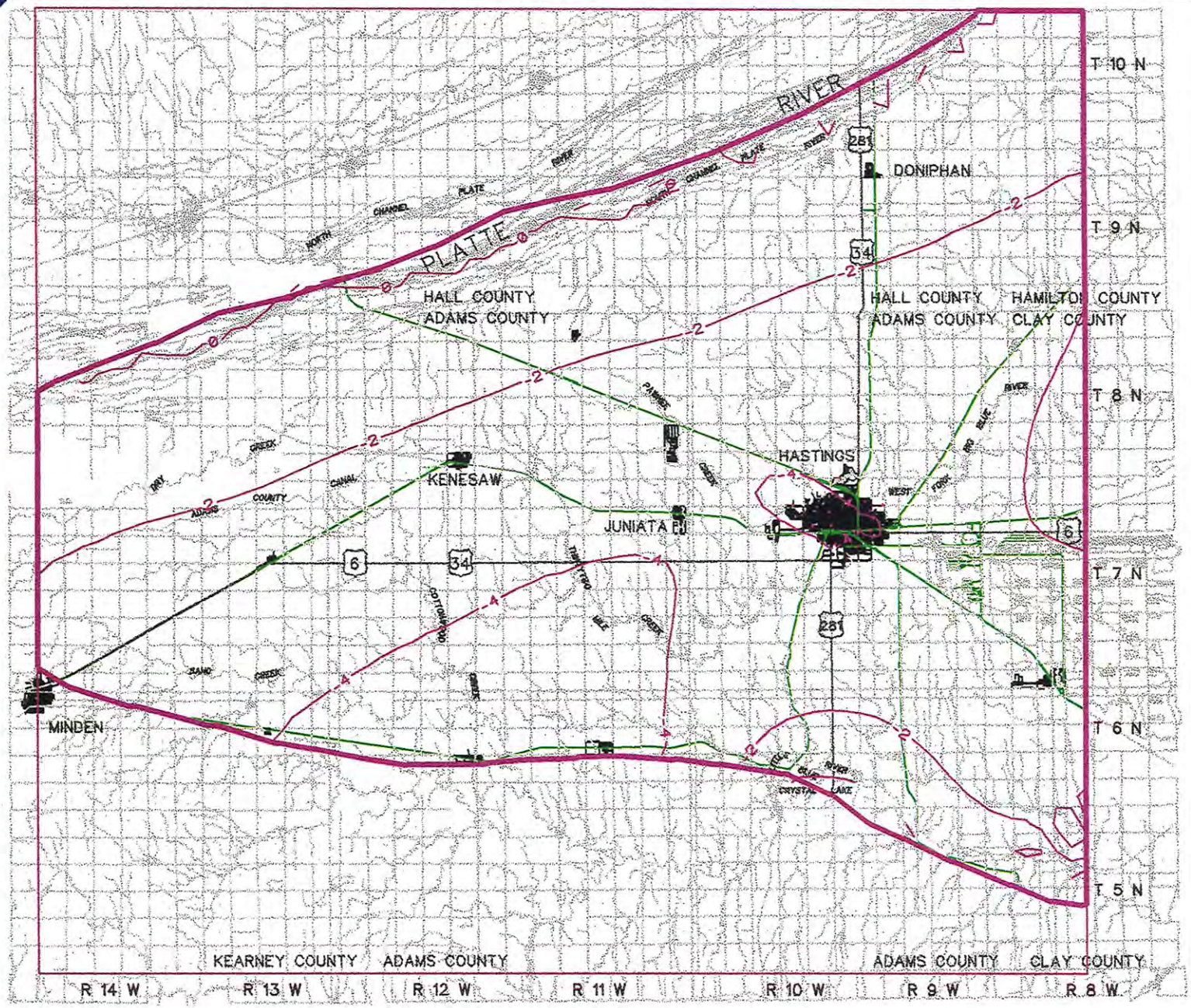
Several development scenarios were simulated in order to estimate the impact of additional groundwater withdrawals on regional water levels. The calibrated post-development recharge

rates were specified for all scenario simulations. Post-development fluxes along the western and eastern boundaries were specified in order to allow heads to vary across this boundary.

For the first scenario, irrigation withdrawals in the entire study area were increased by 10%. This may be interpreted as a 10 percent increase in irrigated acreage. Based on observation from a site visit, this may perhaps represent the upper limit on agricultural development in the area. The resulting drawdowns, from simulated post-development conditions, are presented in Figure 3.10. The drawdowns increase with distance from the Platte River, with up to 4 feet of additional drawdown occurring near the western boundary. It should be noted that the simulation results represent the long-term average drawdowns. The annual and seasonal drawdowns may vary from the simulated values based on climatic conditions in the area.

For the second and third scenarios, additional withdrawals of 2 and 5 MGD at the HU wellfield were simulated. These increases are over and above the average present-day rates presented in Table 2.2. These withdrawals represent an increase of approximately 33 and 80 percent over the current average HU withdrawal of approximately 6 MGD. Simulation results (Fig. 3.11-12) indicate a drawdown of up to 6 feet for the 2 MGD case and 10 feet with the additional 5 MGD withdrawal.

The final development scenario involved a combination of 10 percent increase in irrigation withdrawal (first scenario) along with the 5 MGD additional withdrawal at the HU wellfield (scenario 3) simulated above. This scenario may be assumed to represent an upper limit on groundwater development for agricultural and municipal purposes for the foreseeable future. The simulated drawdowns, from average present-day levels, is presented in Figure 3.13. As is evident from the figure, the groundwater development conditions simulated in this scenario will result in water levels to decline by up to 8 feet in the larger area, with up to 16 feet of drawdown expected near Hastings. It should be noted that the simulated results represent estimates of long-term average drawdowns from average existing conditions.



Simulated Drawdowns (ft) with
10% Increase in Irrigation Withdrawls

FIGURE: 3.10

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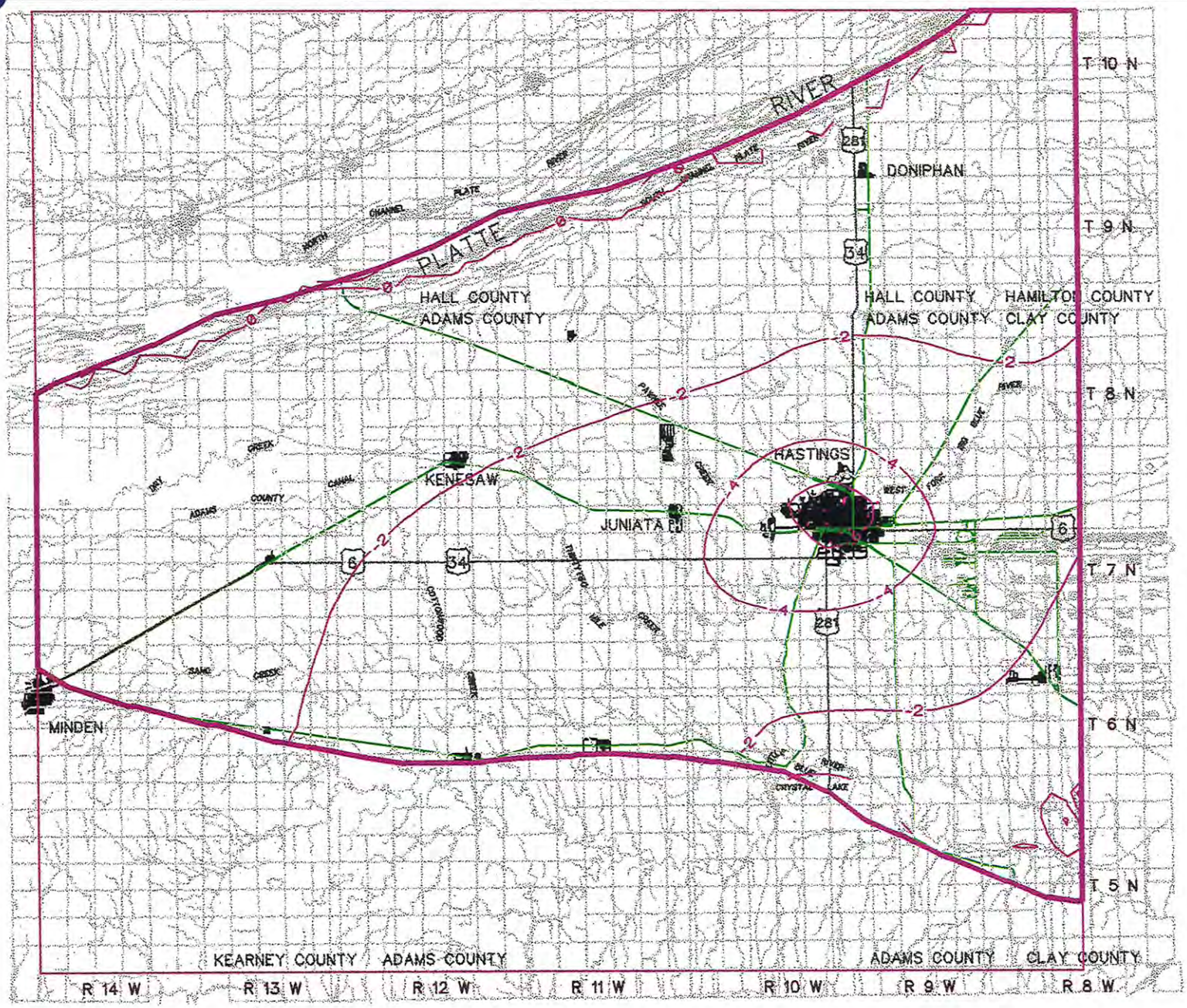

Simulated Drawdowns (ft) with
2 MGD Additional Withdrawls at HU Wells

FIGURE: 3.11

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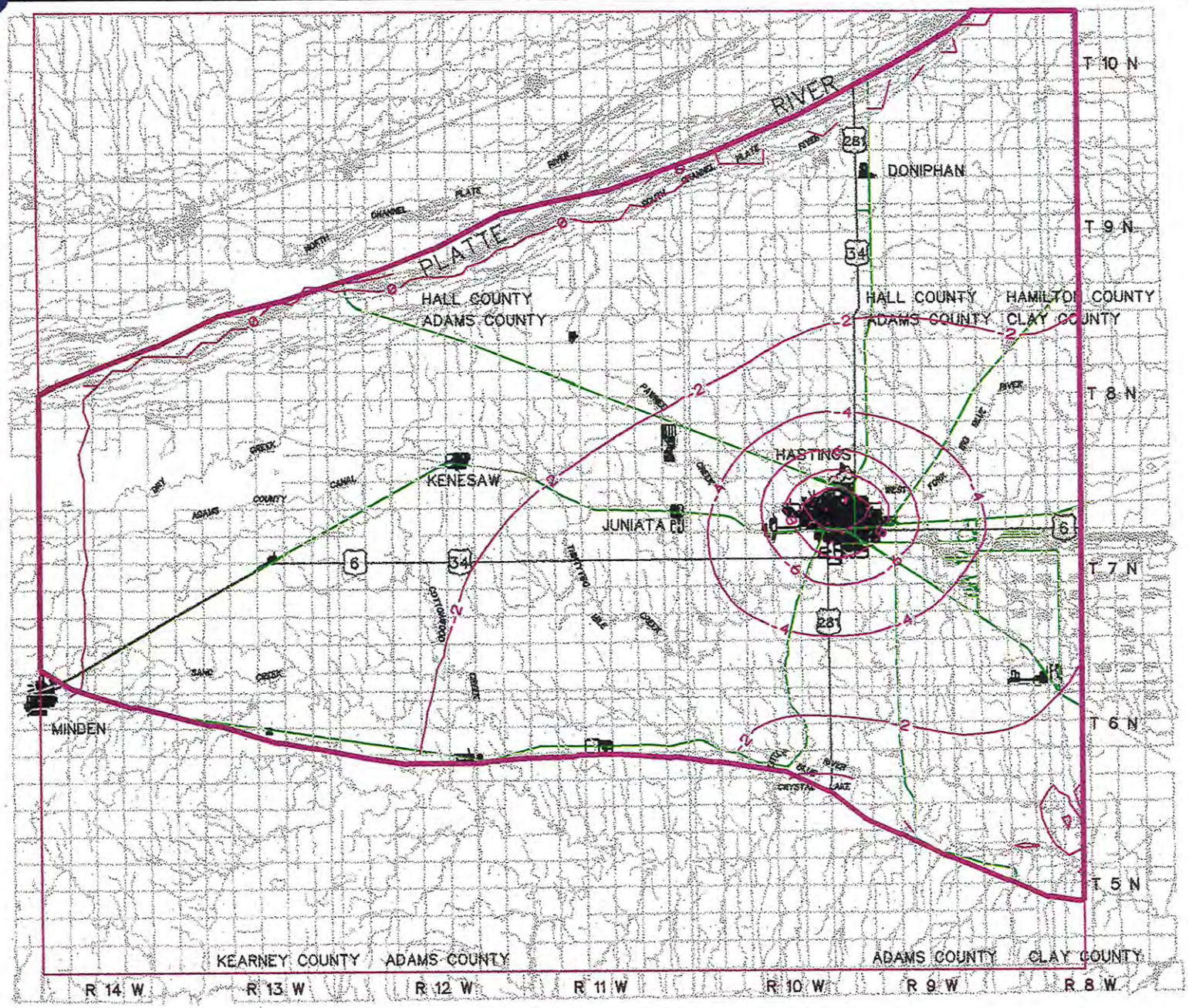

Simulated Drawdowns (ft) with
5 MGD Additional Withdrawls at HU Wells

FIGURE: 3.12

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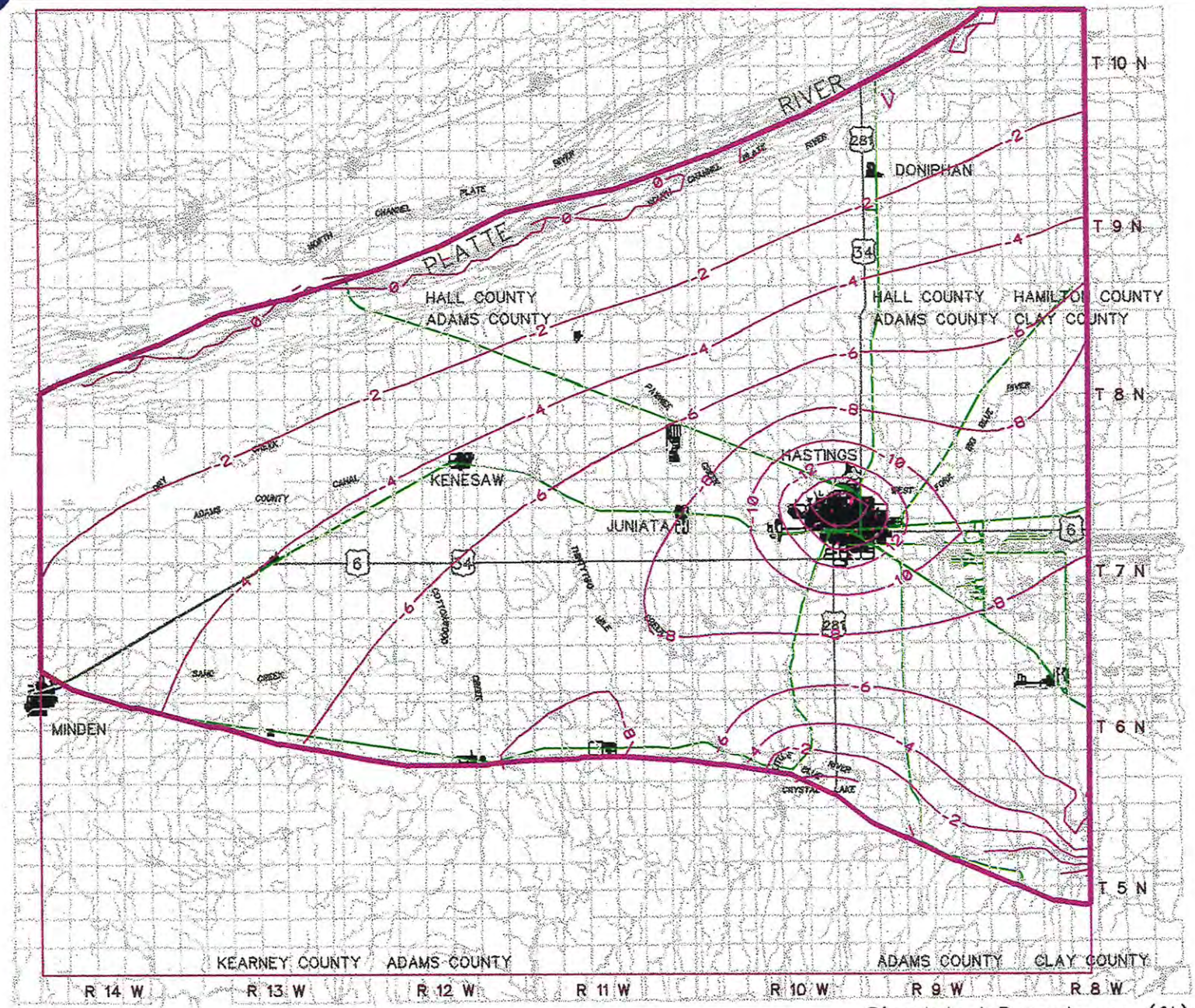

Simulated Drawdowns (ft) with
10 % Increase in Irrigation Withdrawls and
Additional 5 MGD Pumpage at HU Wells

FIGURE: 3.13

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Section 4

Well Head Protection Area Delineation

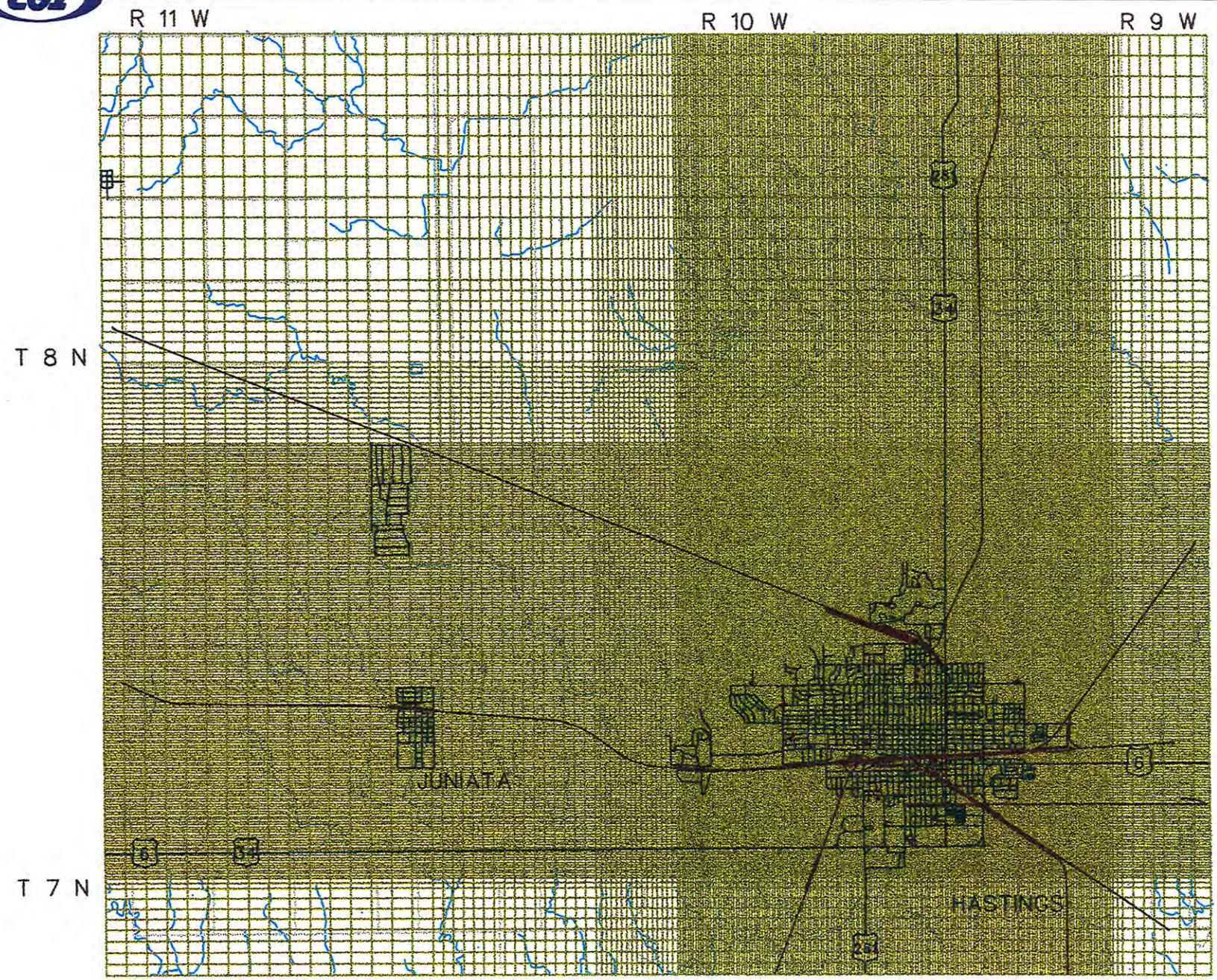
4.1 Introduction

The Nebraska Wellhead Protection Program (WHP), originating from the 1986 amendments to the Safe Drinking Water Act (SDWA), is intended to prevent groundwater pollution from entering public water supply wells in the state. WHP activities are centered around delineation of Wellhead Protection Areas (WHPA) on the surface for a specific well site. Well Head Protection Areas are areas in which contamination, if present, could reach public water supply wells within the useful lifetime of a well. Twenty years is defined as the useful life of a public water supply well by the Nebraska WHP program. The delineation of WHPA's enables monitoring of groundwater quality in order to provide early warning of encroaching contamination. It also provides a basis for conducting a contamination source inventory and ensuring proper land use within the WHPA.

Hastings Utilities (HU), on a voluntary and proactive basis, has taken the initiative to delineate WHPA at its municipal wellfield. Out of the several acceptable methods for delineating WHPA, HU chose numerical modeling, which is most sophisticated method of aquifer analysis and WHPA delineation. Other methods include Calculated Fixed Radii, Analytical Methods, and Hydrogeologic Mapping (WHPA, 1991). Numerical modeling is recognized as a superior method because it can account for field variability of hydrogeologic properties and hydrologic stresses, as well as the impact of well mutual interference on the delineated areas. This method was therefore used for the HU wellfield because of the heterogenieties in the aquifer parameters and the enhanced accuracy of numerical models as a predictive tool for delineation of the WHPA.

4.2 Capture Zone Model

The delineation of WHPA involved the construction of a groundwater flow model in the vicinity of Hastings and coupling of the flow model with a path and travel-time simulator. The flow model, referred to as the site model, was constructed from the regional model described in the previous chapter. A separate model was required for delineating WHPA because the grid resolution of the regional model was inadequate for precise delineation of the WHPA. The USGS numerical modeling code MODFLOW (McDonald and Harbaugh, 1988) was utilized for construction of the site model. A brief description of MODFLOW is provided in section 3.1. The location of the site model with respect to the regional model is shown in Figure 1.1. The site model grid is presented in Figure 4.1. The grid spacing varies from 165 feet near the well sites to 1320 feet near the model boundaries. The relatively fine spatial dicretization in the site model enabled not only precise specification of the municipal wells in the study area, but also resulted in a more accurate representation the groundwater flow dynamics in the area. This is important since delineation of the WHPA is highly dependent on the simulated flowfield.



Site Model Grid

• EXISTING HU MUNICIPAL WELLS

FIGURE: 4.1

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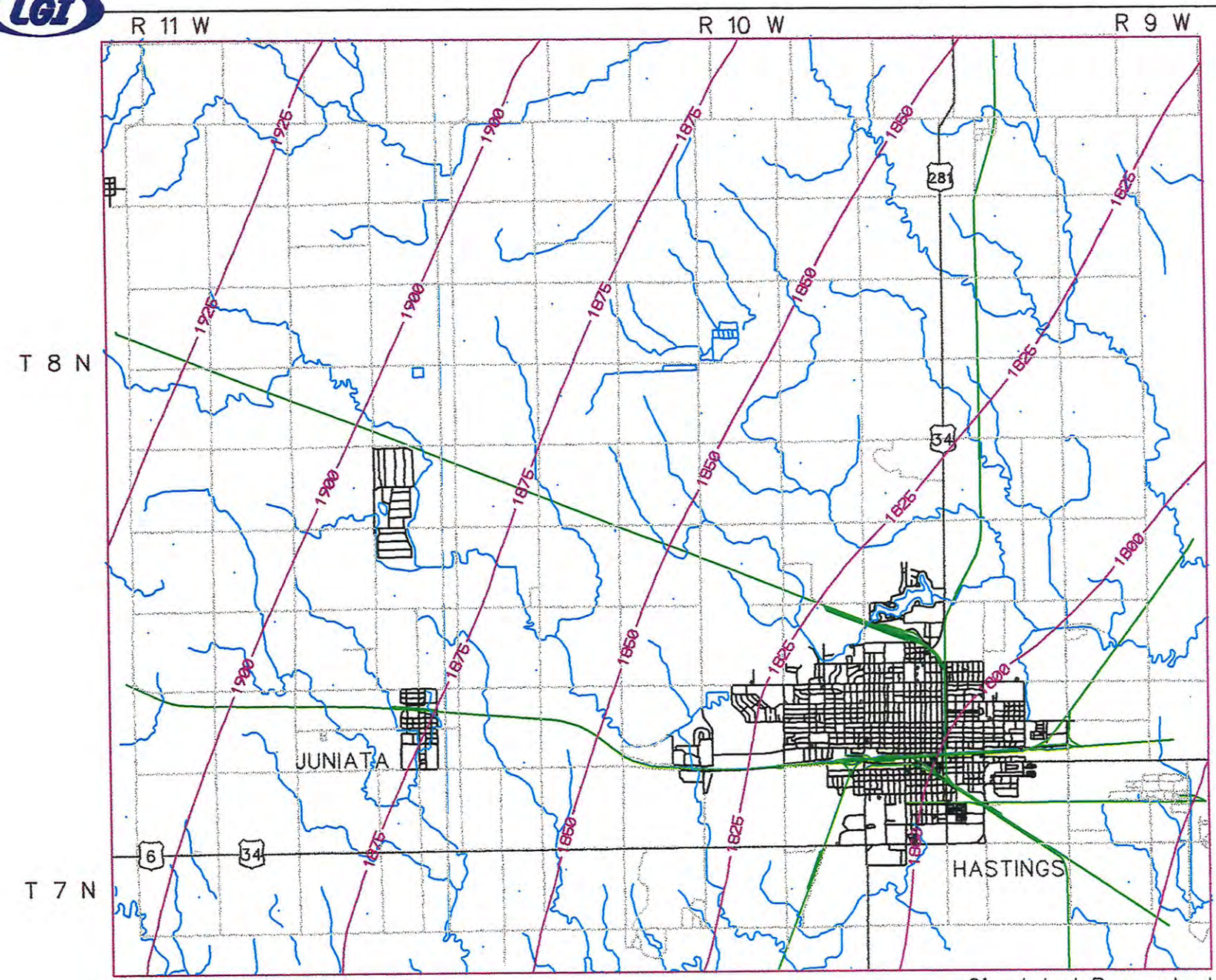
The hydrogeolgic parameters and the (post-development) hydrologic stresses were specified to the site model by interpolation from the regional model. Constant head conditions were specified along the model boundaries. The boundary heads were obtained from the post-development state simulated by the regional model. Average 1993-95 pumpage rates at the municipal wells (Table 2.2) were specified to represent average withdrawal rates in the wellfield.

The proprietary groundwater path and travel-time simulator PATH3D (Papadopulus, 1989) was coupled with the site model for delineating the WHPA. PATH3D is a general particle tracking program for calculating groundwater pathlines and travel times. It is designed specifically to utilize the head field simulated by MODFLOW. The velocity interpolator in PATH3D derives a velocity field from the hydraulic heads calculated by MODFLOW. A numerical solver, capable of automatic stepsize adjustment to achieve pre-determined accuracy, is then employed by PATH3D to track the movement of fluid particles in the groundwater flow system.

The post-development groundwater flow field simulated by the site model is presented in Fig. 4.2a and 4.2b. This flow field was input to PATH3D for calculating groundwater velocities. The velocity calculated from the flow field is the average velocity through the soil matrix, termed the Darcy Velocity. In reality, water flows in a tortuous path through voids in the porous medium (Bear, 1972). In order to calculate the actual flow velocity, PATH3D requires specification of the effective porosity of the medium. The effective porosity is generally lower than the porosity or specific yield of the aquifer. This is because all voids are not interconnected for transmission of water. Previous studies have estimated the specific yield and/or effective porosity to range from 0.20 to 0.278 in the study area (Boyle; 1983, Conestoga-Rovers; 1991, Woodward-Clyde; 1987). In general, lower the effective porosity, higher the resulting pore velocity. In order to obtain conservative results, an effective porosity of 0.18 was assumed in this study for the (base-case) WHPA simulations. As discussed below, sensitivity simulations were conducted to estimate the impact of variability in this parameter on the delineated WHPA.

The WHPA corresponding to a 20 year time-of-travel (TOT) for the existing HU wells is presented in Figure 4.3. The protection zones are fairly long and narrow owing to the relatively high transmissivity of the aquifer. The WHPA's are aligned in a northwest direction along the groundwater flow direction. In general, the zone for each well extends approximately five miles upstream. Due to the likelihood of slight variation in the flow direction seasonally and annually, it is recommended that the WHPA be widened by approximately five degrees. The resulting composite WHPA zone is presented in Figure 4.4, which represents the entire surface area within which a contaminant, if present, can reach a municipal well within twenty years assuming no degradation. Since the capture zone model does not extend till the Platte River, it was not possible to calculate the groundwater travel time between the municipal wellfield and the Platte River. However, from the potentiometric surface of the aquifer, it is estimated that the travel time is approximately 50-75 years.



Simulated Present-day
Water Levels (ft, amsl)

FIGURE: 4.2a

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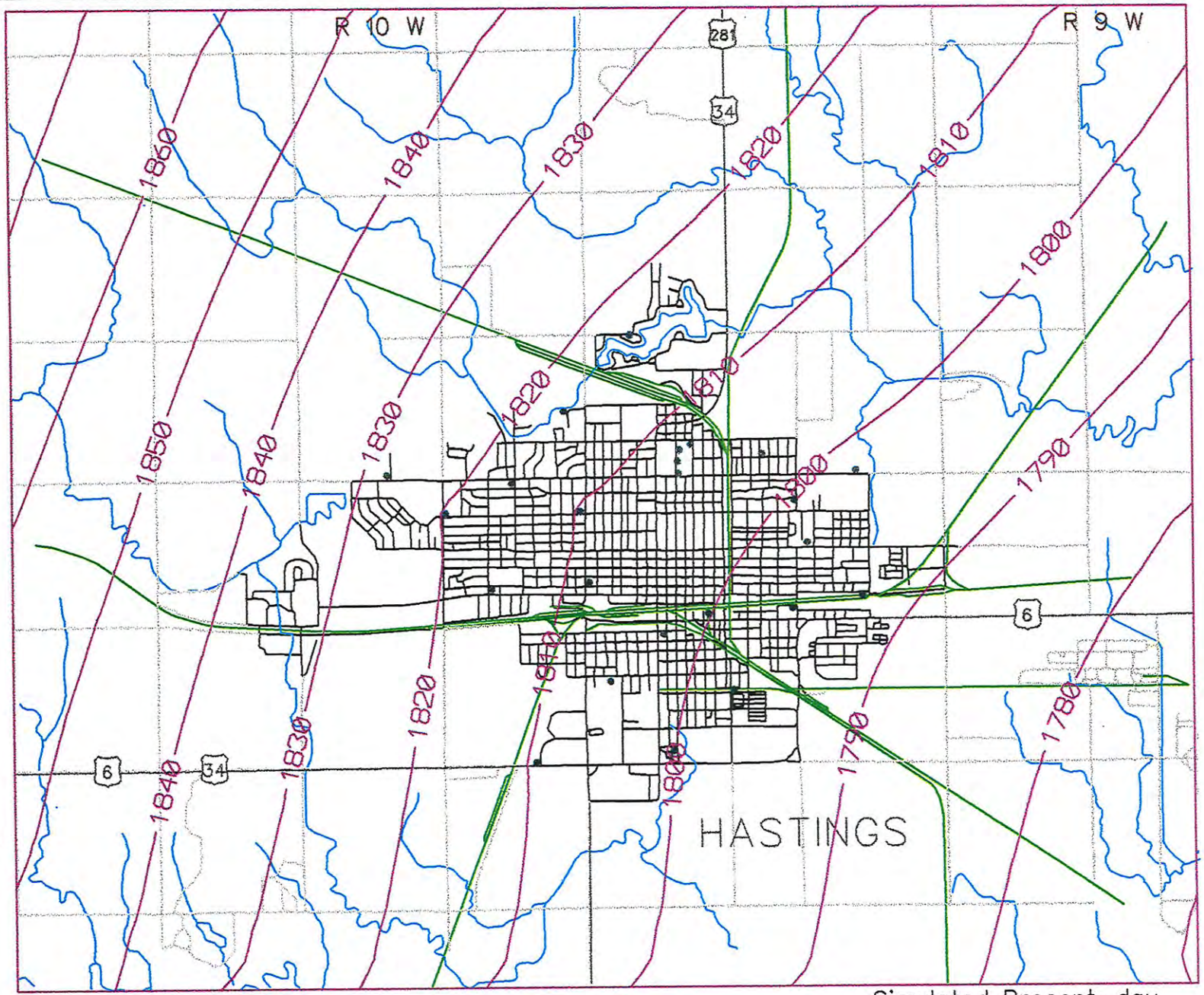

Simulated Present-day Water Levels (ft, amsl) in the Vicinity of Hastings

FIGURE: 4.2b

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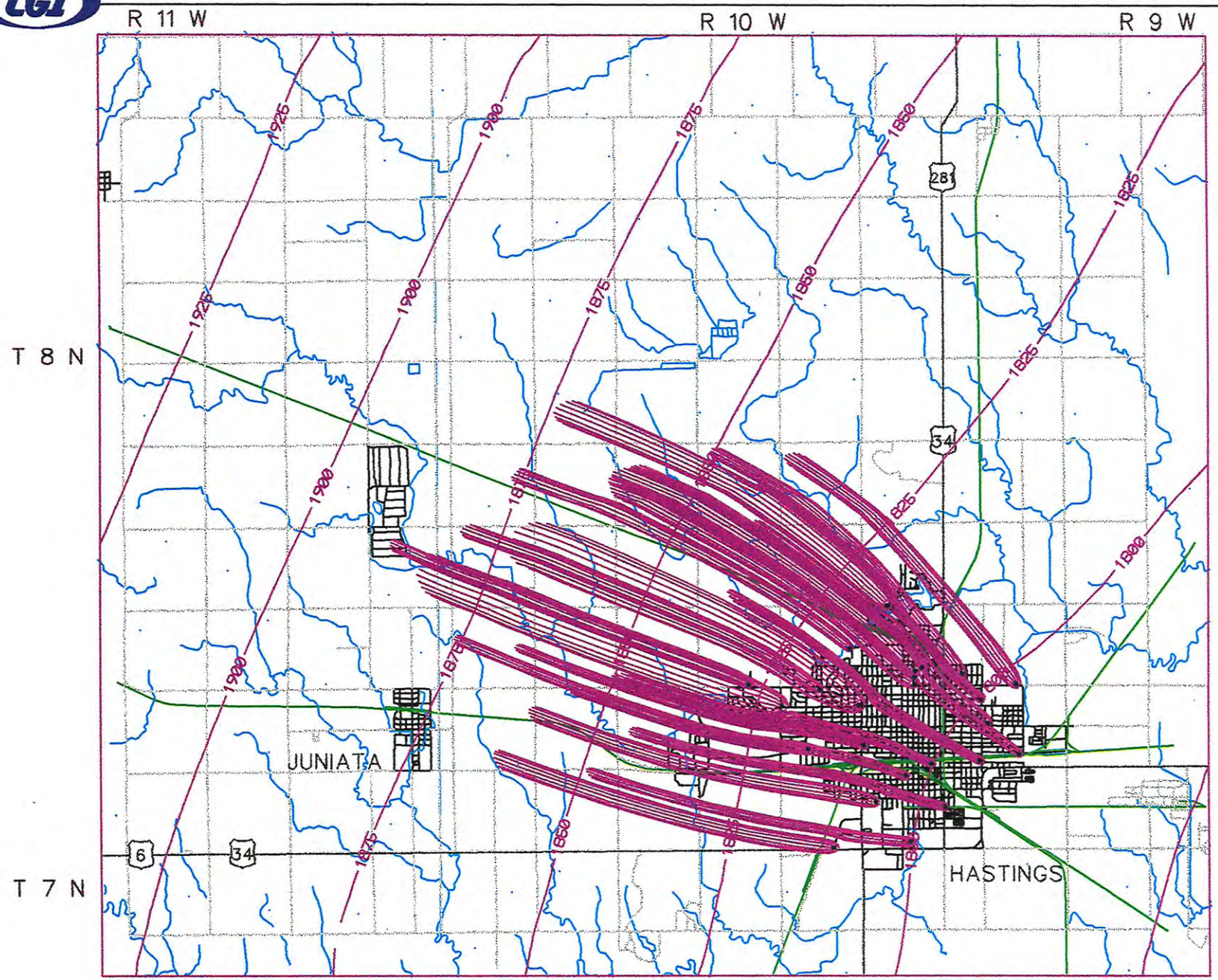

Capture Zones at Hastings Utilities Wells for 20 Year Time-of-Travel

FIGURE: 4.3

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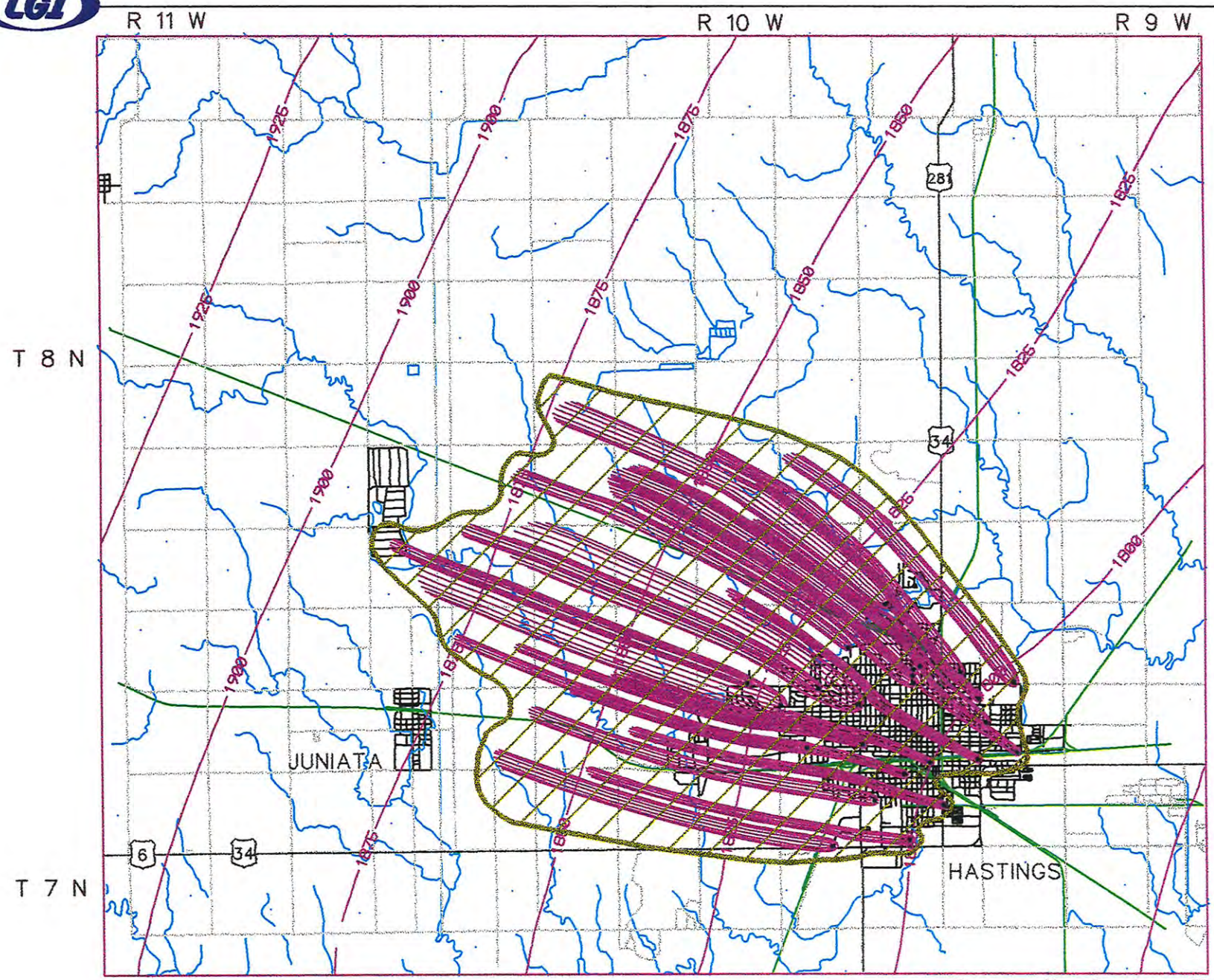

Composite Capture Zone
for 20 Year Time-of-Travel

FIGURE: 4.4

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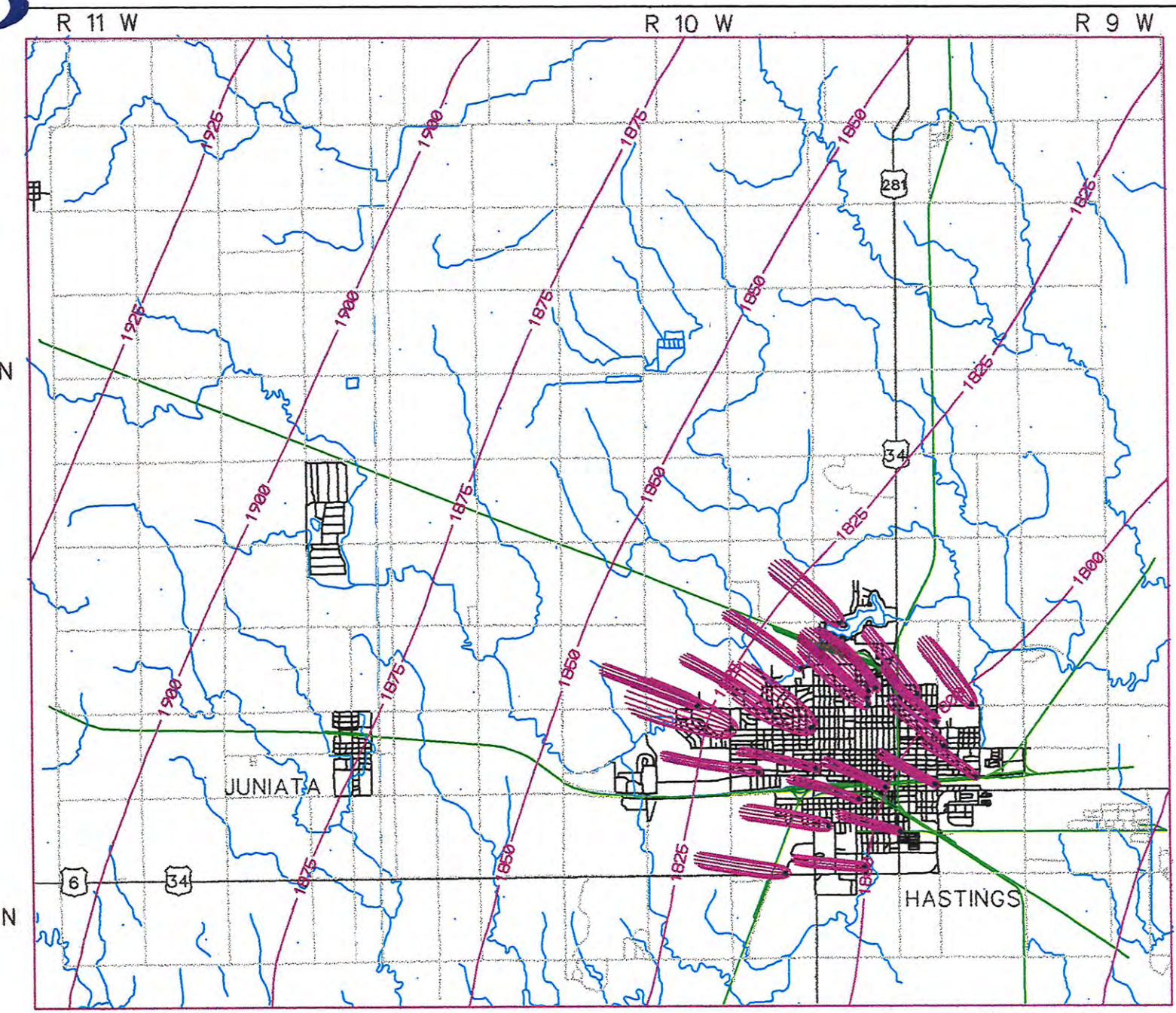
The WHPA corresponding to 5 and 10 year time-of-travel are presented in Figs. 4.5 and 4.6. In general, the length of the delineated areas vary directly with the travel time. The delineated areas for different travel periods are useful for placing monitoring wells at appropriate locations in order to provide adequate warning of encroaching contamination, as well as allow for mobilization and implementation of remedial action if needed. They are also useful for implementing a contaminant-specific wellhead protection strategy because different contaminants have different degradation and retardation rates. Existing water-level monitoring wells (Figure 2.5) could be utilized for monitoring water quality as well.

As mentioned earlier, there may be some uncertainty and natural variation in the specified effective porosity. Based on published information, an effective porosity of 0.18 was assumed for delineating the WHPA. However, to account for any variation in the magnitude of this parameter, the 20 year TOT WHPA zones corresponding to an effective porosity of 0.13 and 0.23, the expected range of this parameter, are presented in Figs. 4.7-8. As expected, the length of the capture zones increase with decreasing effective porosity and decrease with increasing porosity.

4.3 Examination of the Process of Nitrate Migration in Groundwater

Nitrates in groundwater pose a threat of contamination to the HU wellfield. Several wells have already been taken out of service due to high nitrate levels. Nitrate levels in most other wells have risen from less than 1 ppm in the late sixties to approximately 5 ppm presently. Wellhead Protection Area delineation is a useful management strategy, but it cannot protect wells from contamination originating from outside the protected zone and eventually migrating towards the wells. In addition, it is often difficult to modify existing land use within WHPA's for political and economic reasons. The impact of various agricultural practices within the WHPA on actual contamination levels at the HU wells cannot be estimated by particle tracking simulations, since it cannot account for concentration levels. Questions related to the source or sources of contamination at HU wells, expected nitrate levels in the future, the likelihood of nitrate levels to exceed the EPA recommended limit of 10 ppm, impacts of localized plumes from accidental or improper handling practices passing through the well sites, are all vital questions which cannot be addressed through particle tracking simulations. More advanced analyses procedures are required to predict fate and transport of nitrates.

The transport of nitrate within the saturated zone is primarily governed by advective-dispersive processes (Canter; 1997, Cambareri; 1989). Several hypothetical scenarios were simulated with an advection-dispersion model to gain an understanding of the impact of these processes on nitrate migration in groundwater. Representative values of the aquifer parameters and the groundwater flow field were specified. It should be noted that the simulations were conducted only to obtain an estimate of the influence of advective-dispersive process on the transport of nitrates in groundwater. Precise estimate of nitrate levels requires calibration the model to field data.



Capture Zone
for 5 Year Time-of-Travel

FIGURE: 4.5

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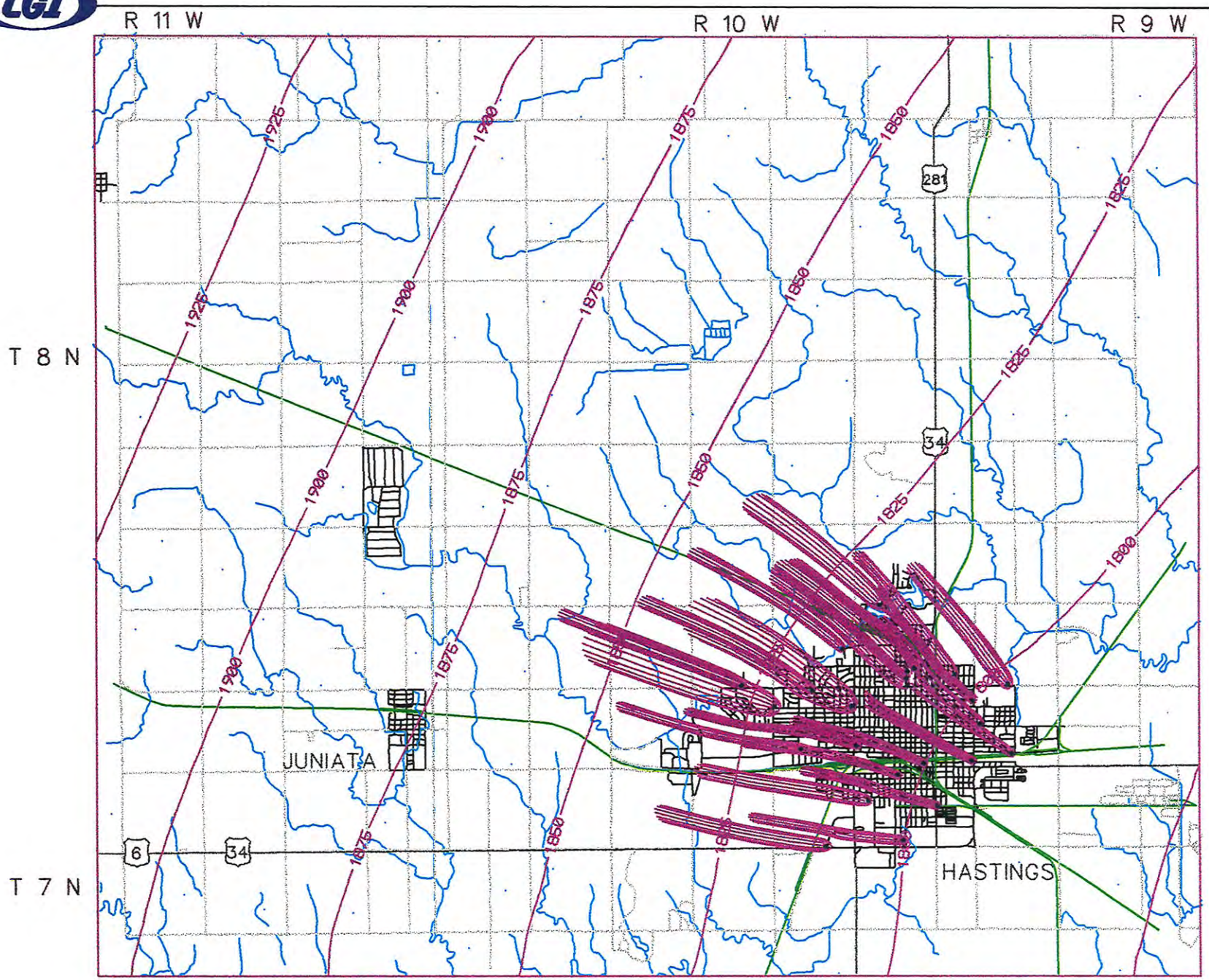

Capture Zone
for 10 Year Time-of-Travel

FIGURE: 4.6

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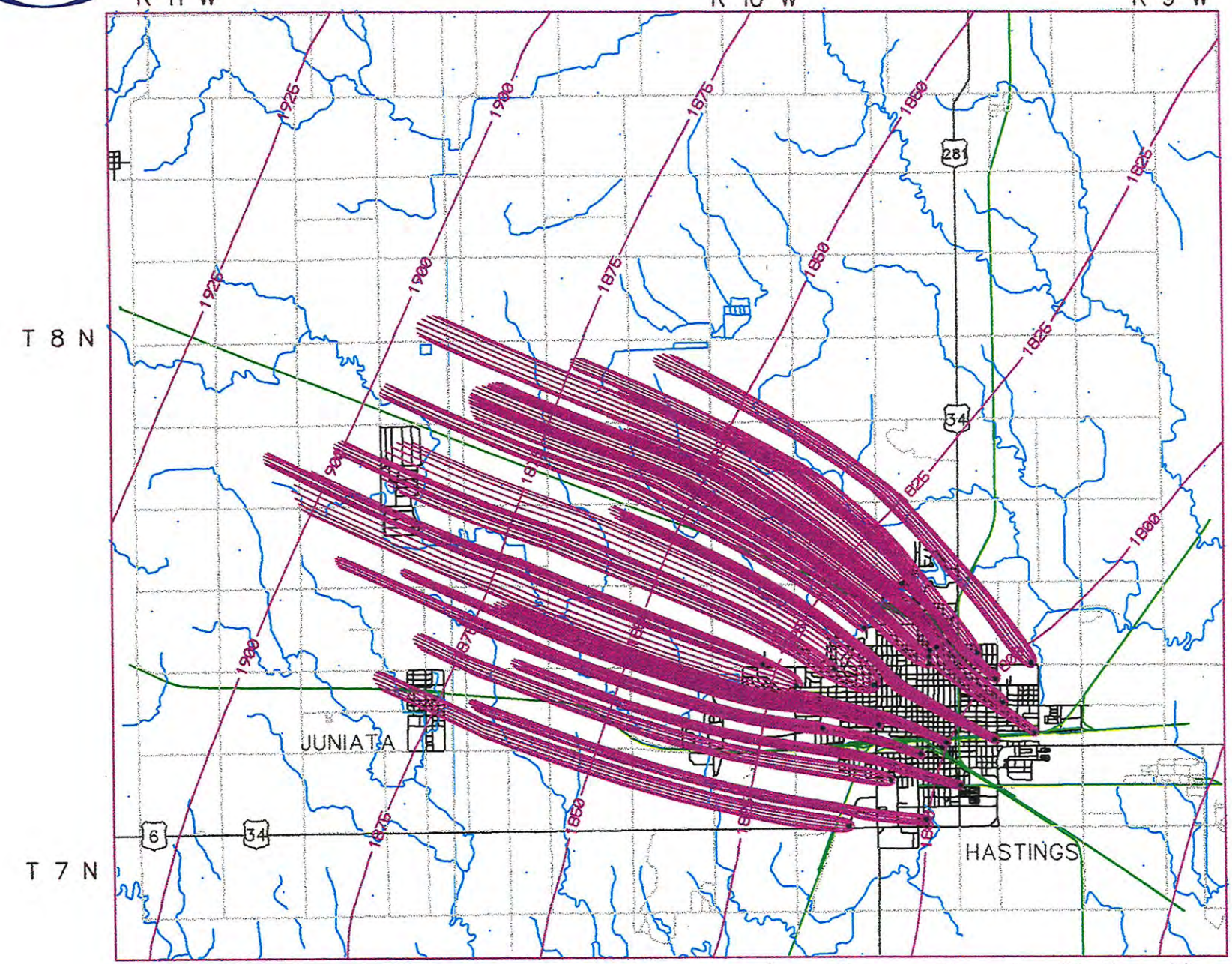

Capture Zone for 20 Year Time-of-Travel
(Effective Porosity – 0.13)

FIGURE: 4.7

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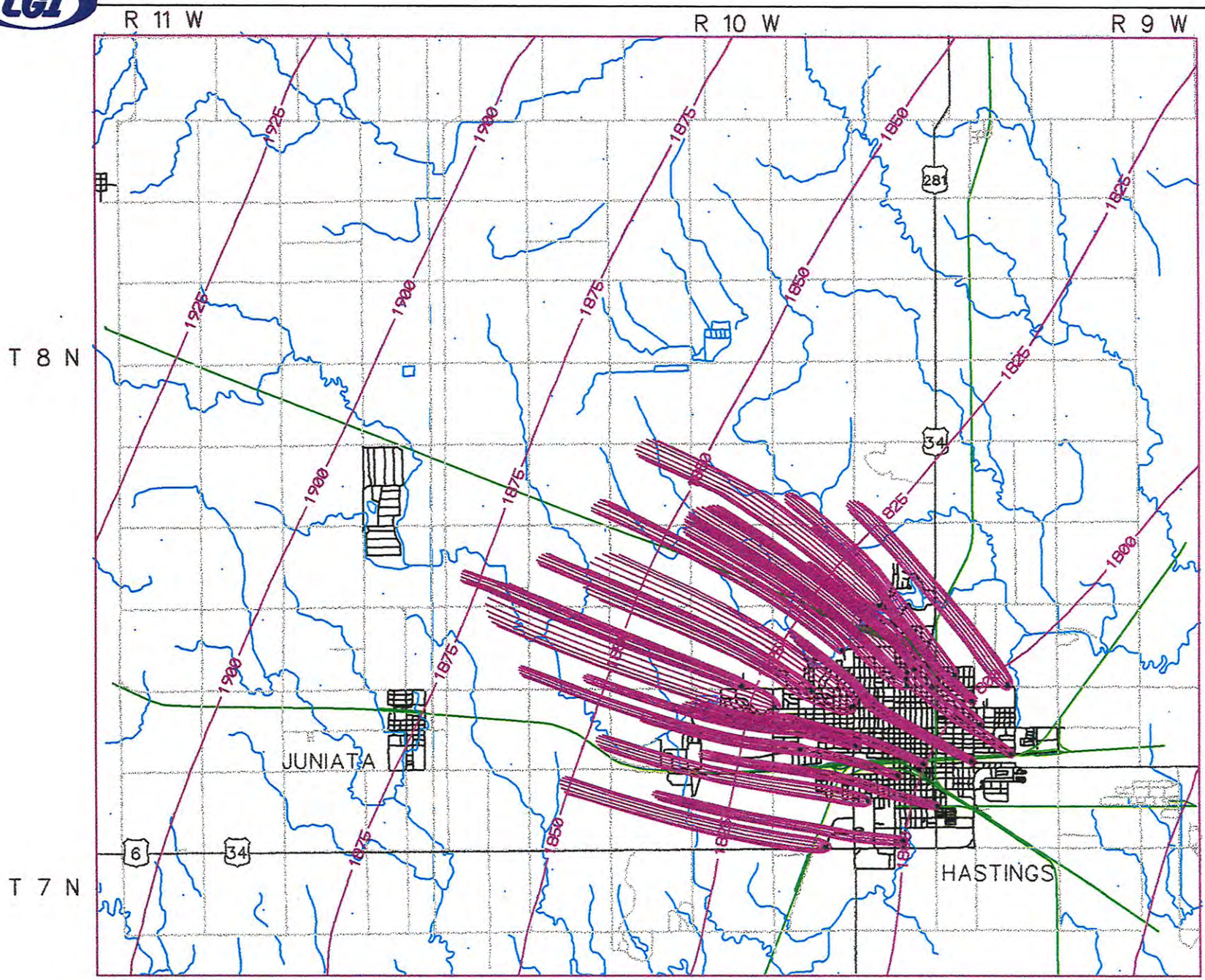

Capture Zone for 20 Year Time-of-Travel
(Effective Porosity – 0.23)

FIGURE: 4.8

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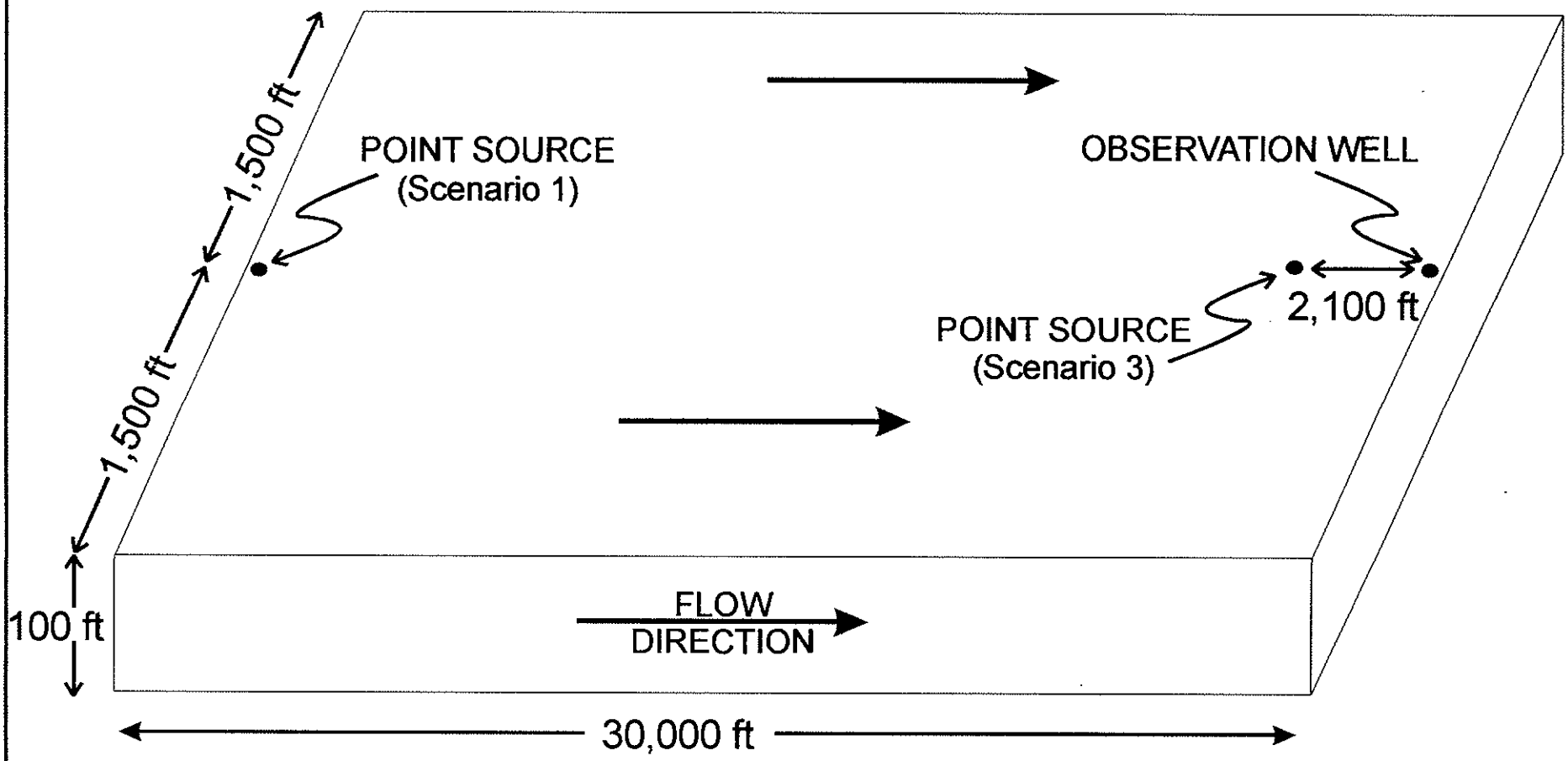
The model configuration is presented in Figure 4.9. For all simulations, a uniform flow field of 0.6 feet/day was assumed. This is a representative value of this parameter in the area. The aquifer was simulated by a single model layer. For more precise results, a multilayer model is required to account for gradation of nitrates with depth. Longitudinal and transverse dispersivities were specified as 50 and 5 feet respectively. For all simulations, fresh water initial conditions were assumed.

For the first scenario, a constant rate nitrate point-source at the upstream boundary (Fig. 4.9), resulting in a concentration of 20 ppm in the groundwater, was simulated. This scenario represents conditions developed under a feed-lot. The areal distribution of nitrates after 10, 25, and 50 years are presented in Figures 4.10a,b, and c. The breakthrough curve at an observation well approximately 5 miles from the source (Fig. 4.9) is presented in Figure 4.11. The simulation results highlight the development and spreading of a nitrate plume, with contamination reaching the well in approximately 15 years and rising to 3.5 ppm in 50 years. It appears that nitrate levels will eventually stabilize at about 5 ppm. The results of this scenario suggests that feed-lots in the area could contribute to nitrate contamination at the HU wells. In order to obtain more precise estimates of contamination from feed-lots, model calibration is required.

For the second scenario, nitrate loading from a non-point source was simulated. A uniform recharge rate of approximately four inches per year was specified. Nitrate concentration of 5.0 ppm at the water table was assumed. This was estimated from a previous research study (Spalding and Kitchen, 1988) on the impact of nitrate contamination in agricultural areas of Nebraska. A concentration of between 4-5 ppm at a depth of 60 feet under fertility plots loaded with 400 lbs of nitrogen per acre was noted in the study. Since the nitrate loading is uniformly distributed in this scenario, only advective process was considered for this scenario. The breakthrough curve at the observation well along the downstream boundary (Fig. 4.9), assuming mixing along the entire thickness of the aquifer, is presented in Figure 4.12. Nitrate levels increase with time, eventually stabilizing at 1.5 ppm. The attenuated nitrate levels appear to be rather low. It is possible that that the estimated loading rate is low. Furthermore, the attenuated levels are a function of the contributing area. For the present simulations, the upstream boundary was approximately five miles from the observation well. For a larger contributing area, as would exist in the actual study area, the attenuated concentration will be higher. In order to construct a representative nitrate model, spatial and temporal variation of the loading rate would need to be calibrated to the observed distribution on nitrates in the study area. The objective of the present simulations is merely to assess the impact of advective-dispersive processes on migration rates



Problem Description
for Nitrate Simulations

FIGURE: 4.9

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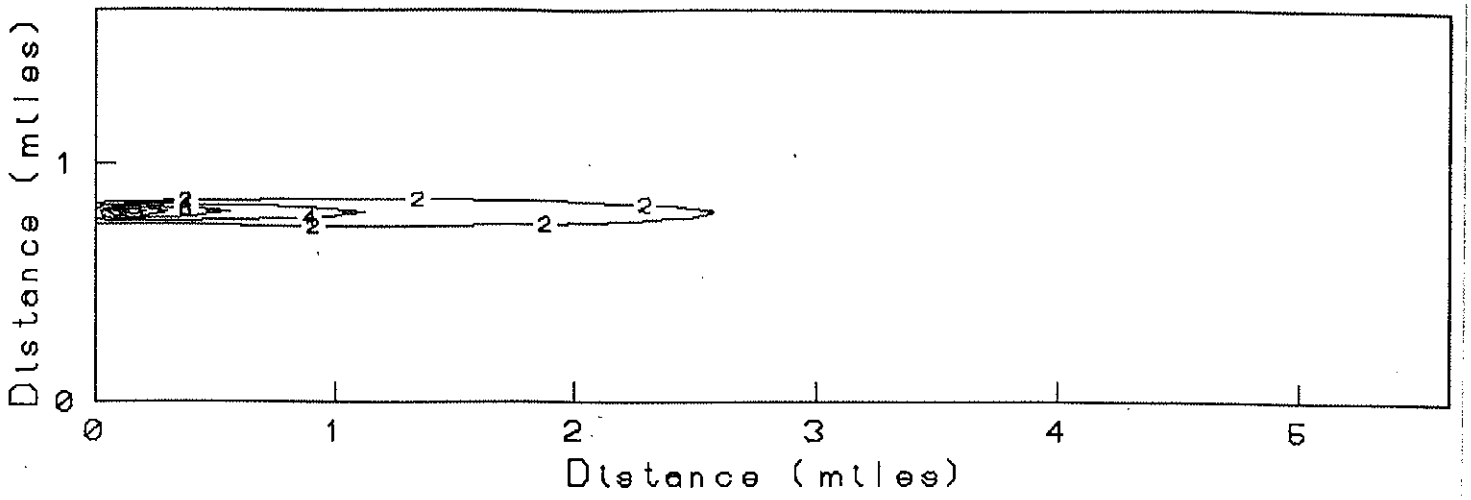


Figure 4.10a Areal Distribution of Nitrates after 10 Years of Loading

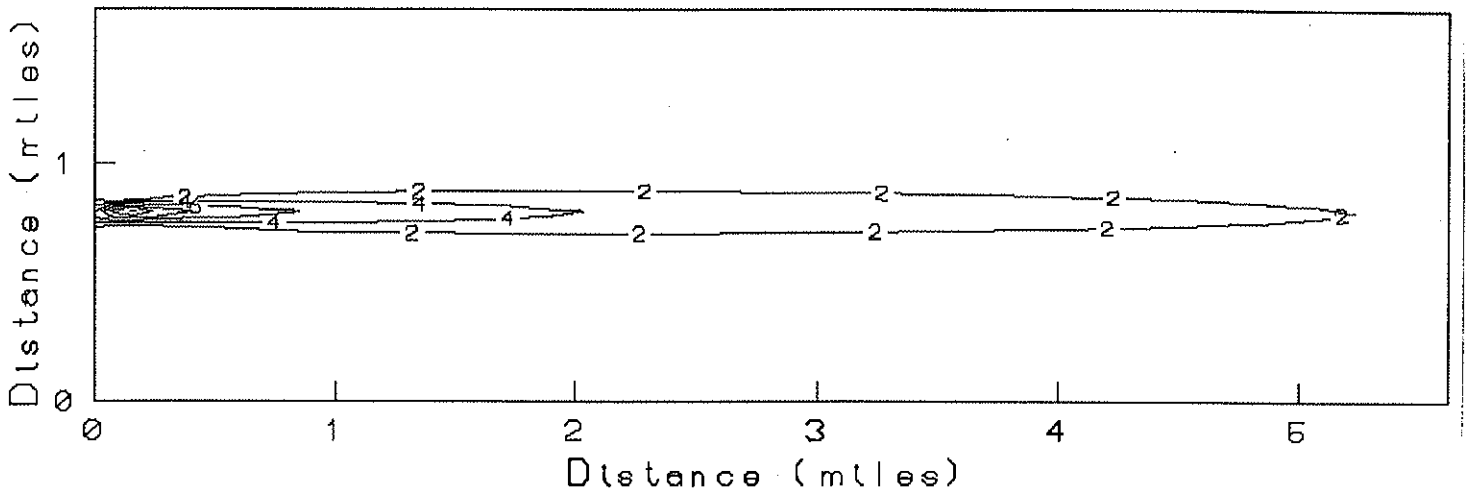


Figure 4.10b Areal Distribution of Nitrates after 25 Years of Loading

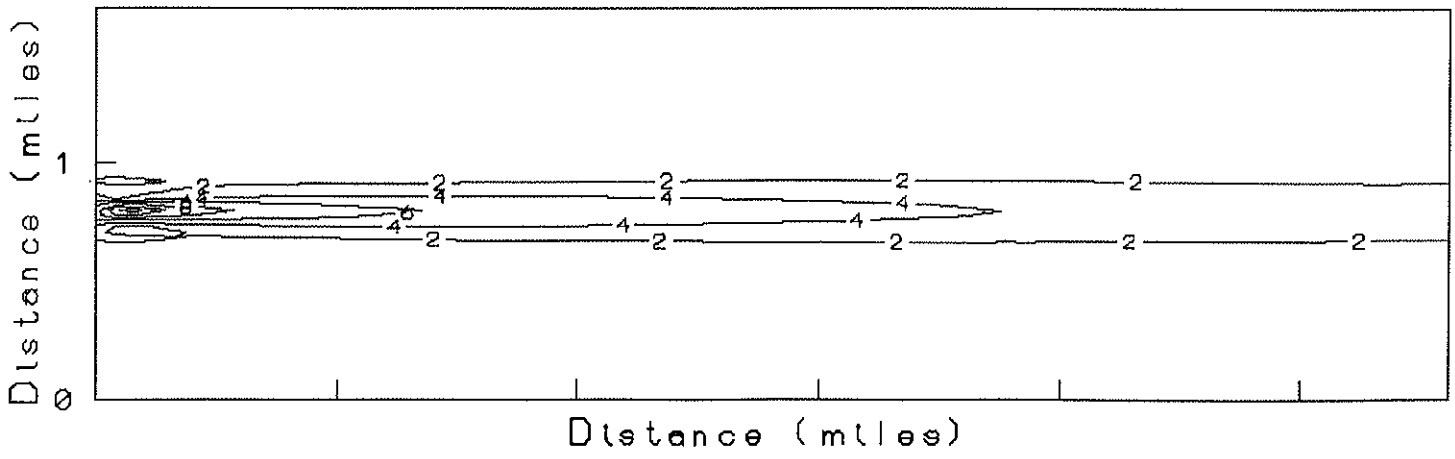


Figure 4.10c Areal Distribution of Nitrates after 50 Years of Loading

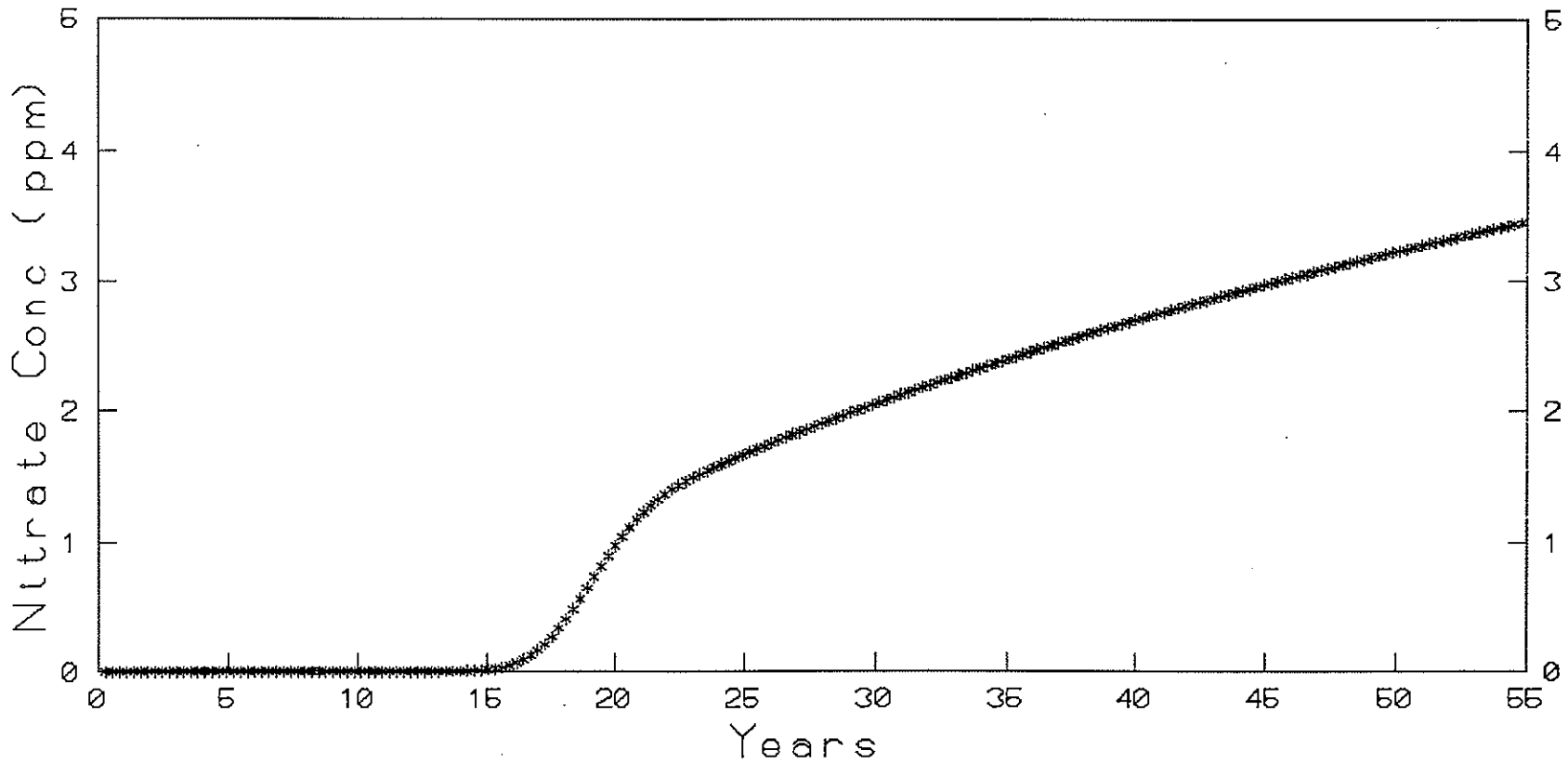


Figure 4.11 Nitrate Concentration at Observation Well for Scenario 1

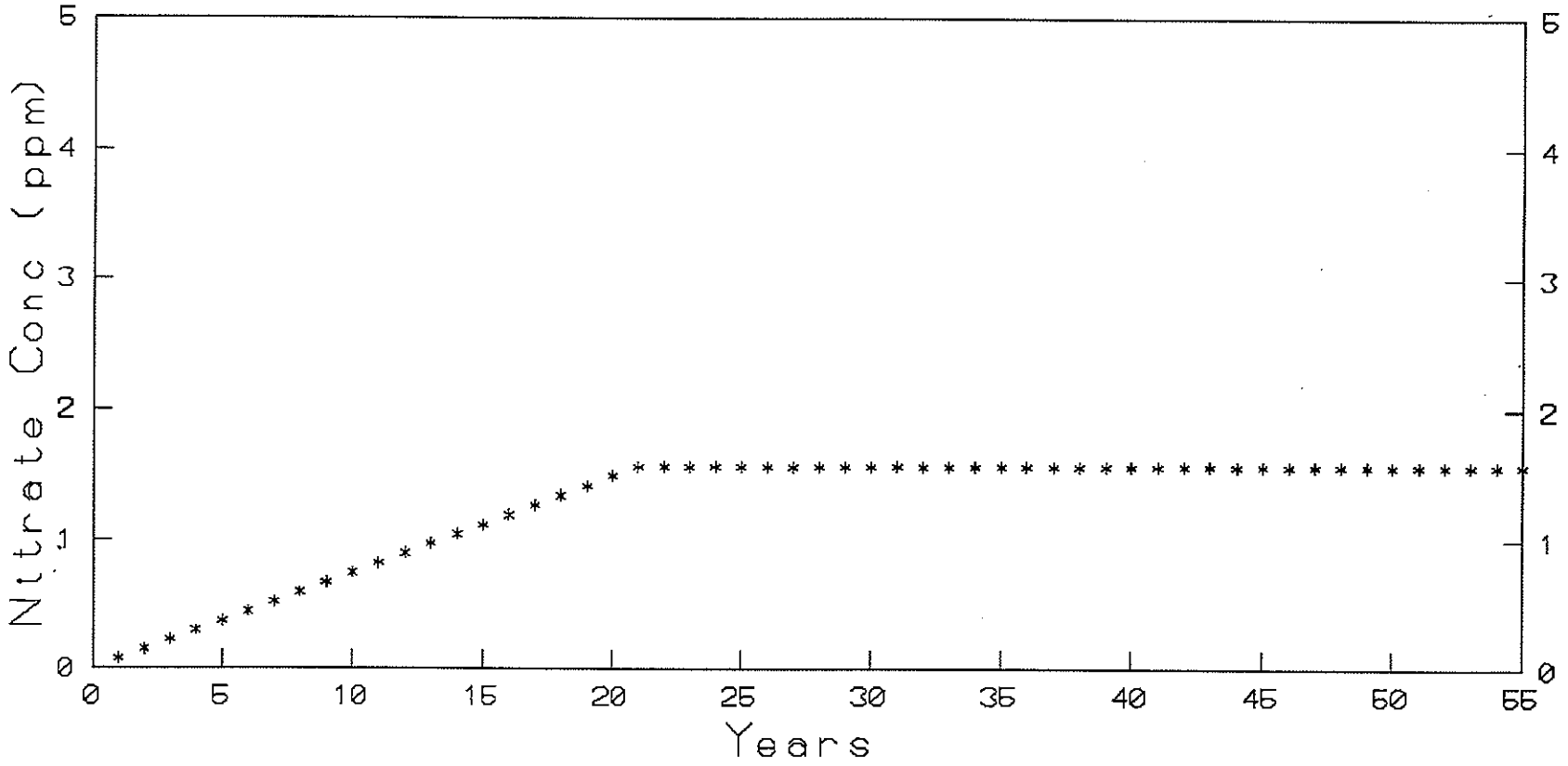


Figure 4.12 Nitrate Concentration at Observation Well for Scenario 2

and spreading of nitrates, and not the prediction of nitration concentration itself.

For the final scenario, the case of a plume developing and migrating from a one-time leaching of nitrate was simulated. The loading rate is presented in Figure 4.13a. This scenario may represent conditions developing under a storage facility due to improper handling of nitrate sources. The location of the source and observation well are presented in Figure 4.9. The breakthrough curve at the observation well is presented in Figure 4.13b. As expected, nitrate levels rise precipitously as the plume passes through the observation site and then subside as the plume moves downstream. This condition is reflected in nitrate concentrations at HU well no. 11.

4.4 Wellfield Development in Northwest Hastings

In order to replace lost wellfield capacity due to contamination and aging of existing wells, wellfield development in northwest Hastings is under consideration by HU (HDR, 1996). The locations of the proposed wells, in northwest Hastings, are presented in Figure 2.8. Model runs were conducted in order to assess the capability of the aquifer to support withdrawals from a cluster of proposed and existing wells in northwest Hastings.

For the first scenario, 6 MGD was withdrawn from wells 8, 16, 17, 22, 23, 25-35 in the northwest. The resulting steady-state (long term average) water levels are presented in Figure 4.14. Comparing the simulation results with water levels resulting from the 6 MGD pumpage in existing wells (Fig. 4.2b), a slight recovery is noted in the original wellfield and a drawdown of less than 3 feet is noted in the northwest. It is therefore obvious that the aquifer can support existing demand of approximately 6 MGD from pumpage in the northwest.

A simulation run was also conducted with 10 MGD withdrawal from the same cluster of wells simulated in the first scenario. The increased withdrawal may be considered to represent withdrawals for a dry period. The simulated water levels for this case are presented in Figure 4.15. A drawdown of over 5 feet can be noted for this case. Since the drawdown is not excessive, it can be concluded that withdrawals from the wells in the northwest to meet municipal demands is feasible.

Although the average municipal withdrawal is 6 MGD, peak daily demand may be substantially higher. Peak demand in recent years has been as high as 19 MGD. In order to determine the capability of the well cluster in northwest Hastings to meet peak demand, transient simulation are required. These however are beyond the present scope of work. In order to estimate the transient impact of peak withdrawal, a Theis (Driscoll, 1986) drawdown analyses was conducted. Theoretical drawdowns at a distance of 1000 feet from a well pumping 1200 gallon per minute (gpm) in a confined aquifer of 200,000 gpd/ft transmissivity and storage coefficient of 0.2 were calculated. The drawdowns after 30, 60, and 120 days were 1.5, 2.0, and 2.4 feet respectively (Table 4.1). The corresponding drawdowns at a distance of 2000 feet were 0.7, 1.2, and 1.5 feet respectively. For the layout of the proposed wells presented in Figure 2.8, each well is to be influenced by no more than 3 wells at distance of 1000 feet and two wells at a distance of 2000

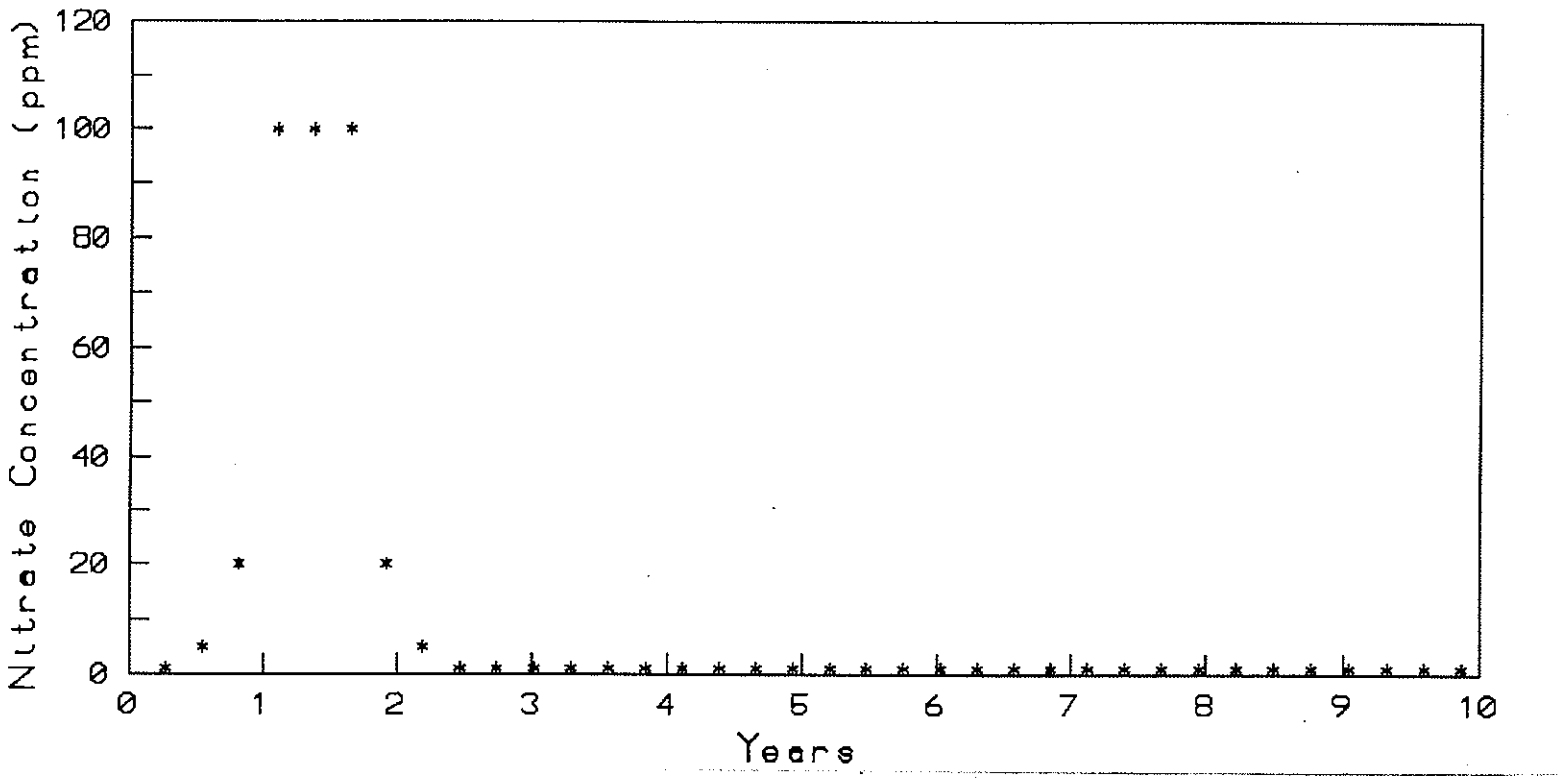


Figure 4.13a Nitrate Loading in Groundwater for Scenario 3

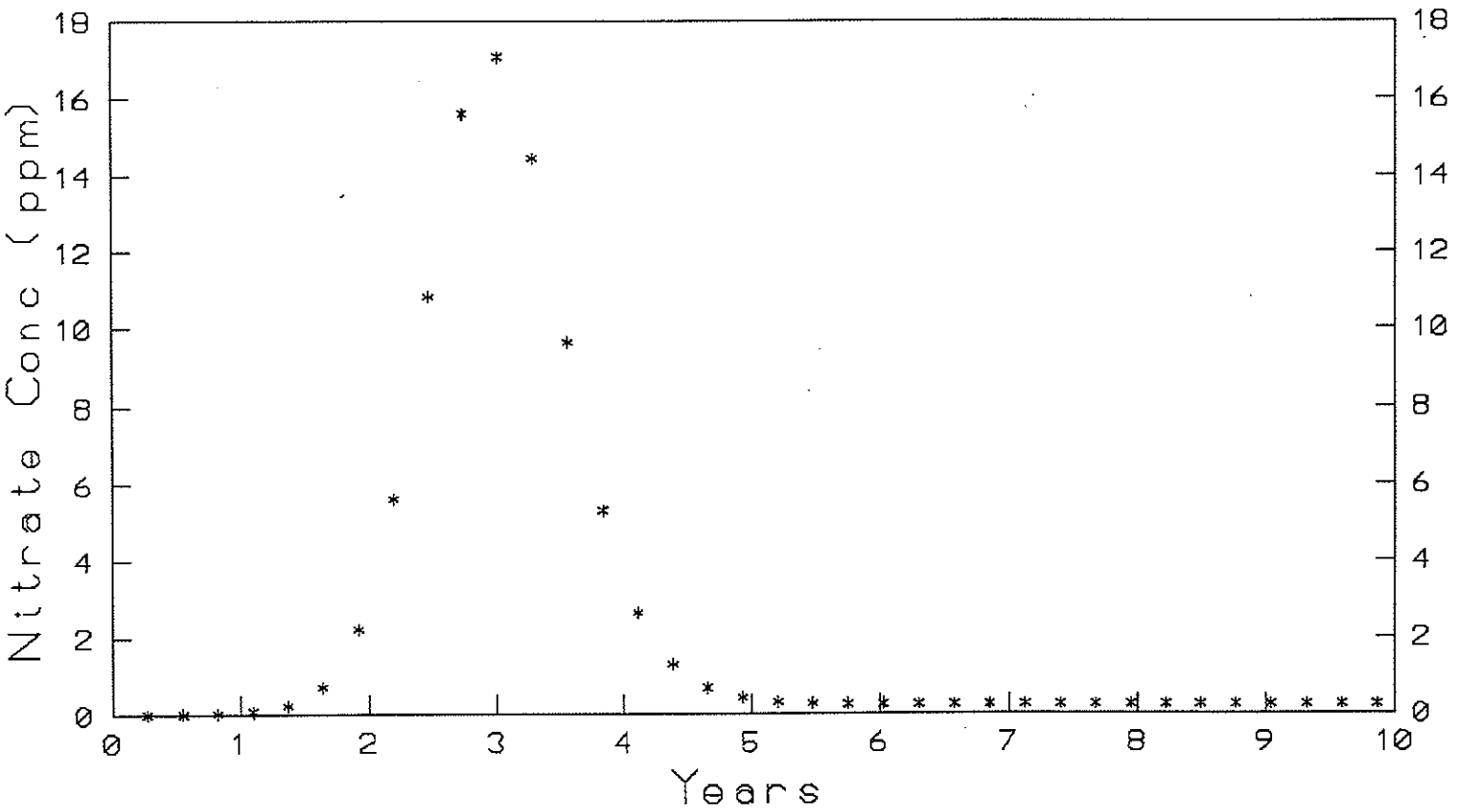
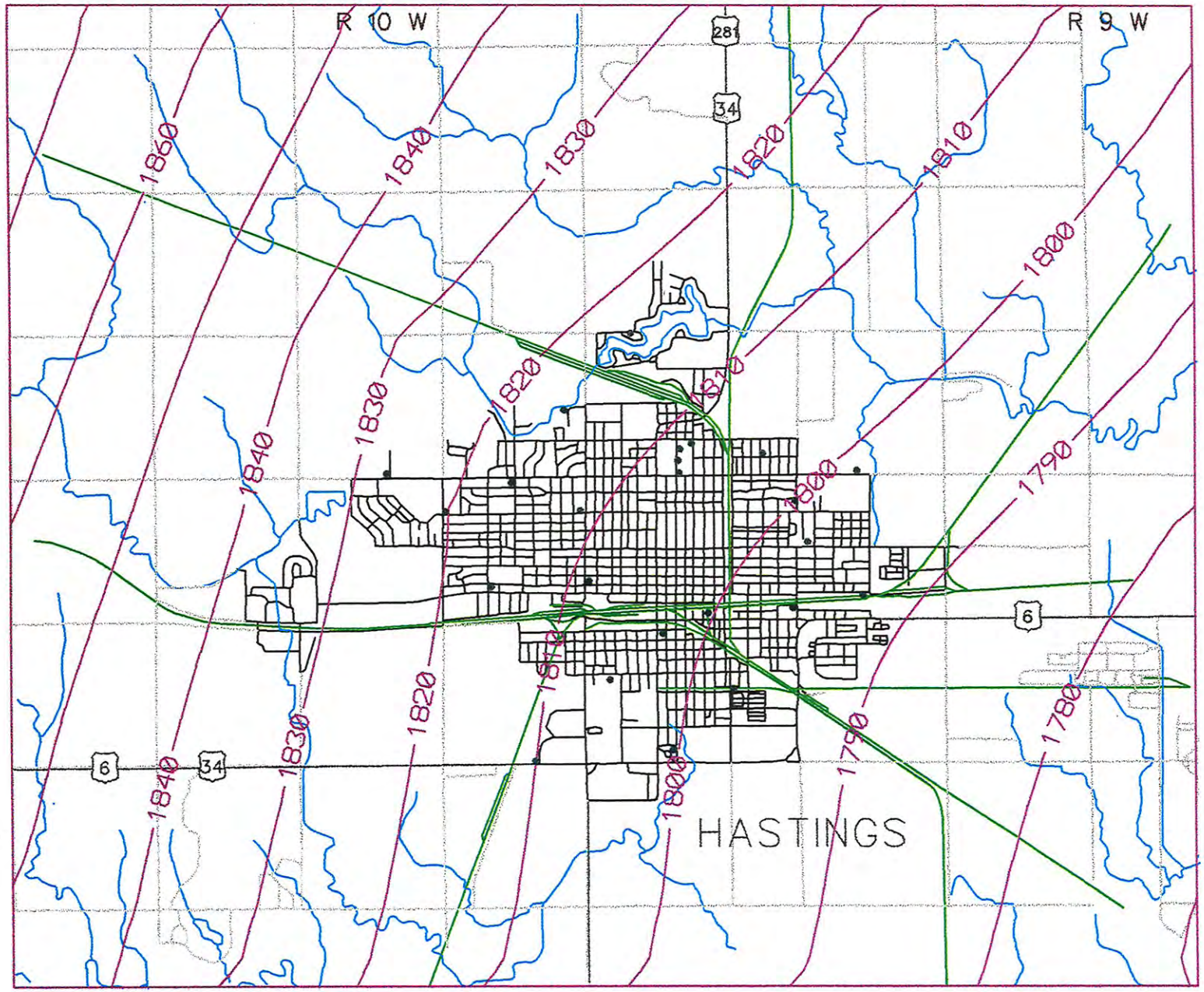


Figure 4.13b Nitrate Concentration at Observation Well for Scenario 3



Simulated Water Levels with 6 MGD
Withdrawl from Wells in Northwest Hastings

FIGURE: 4.14

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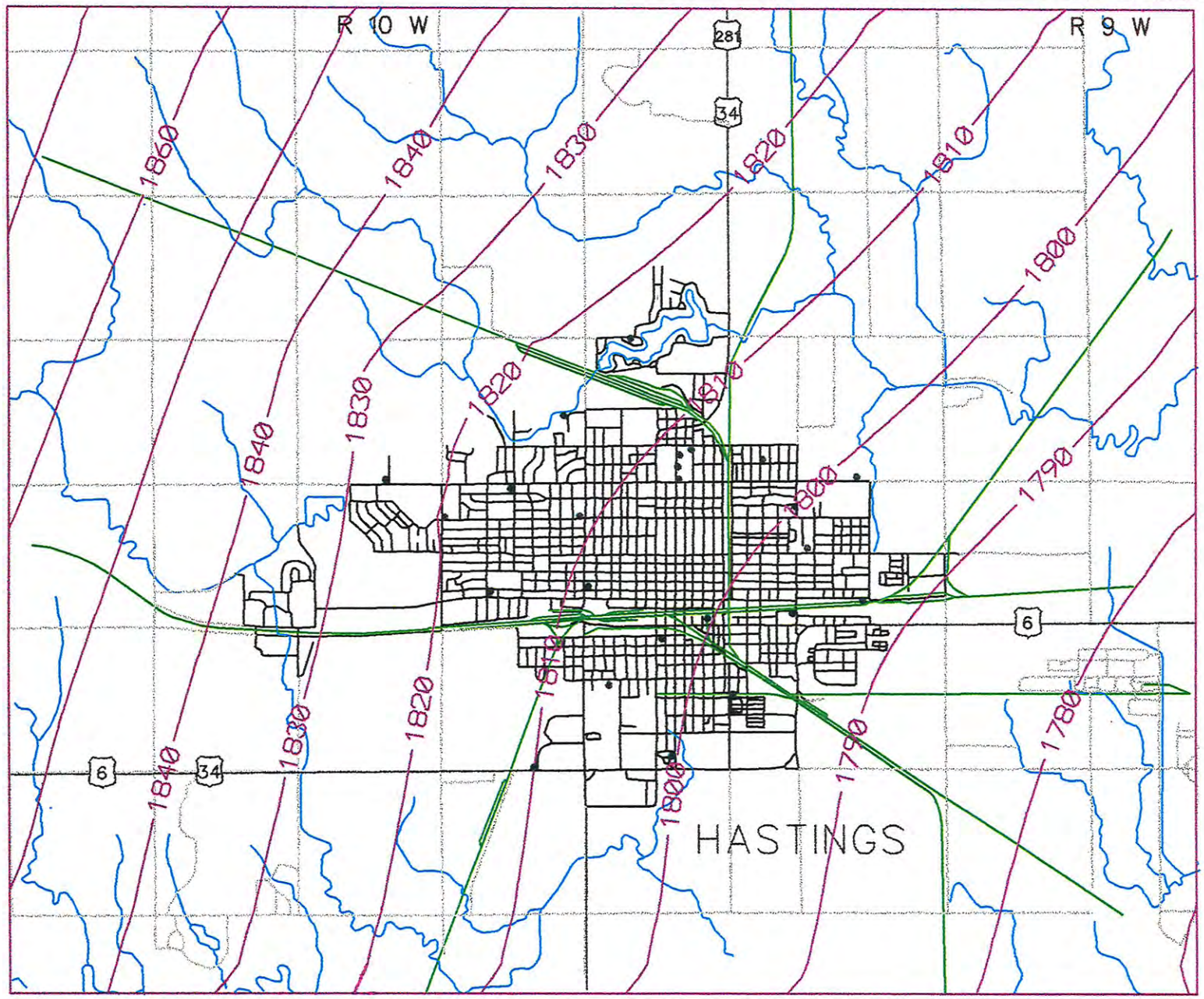

Simulated Water Levels with 10 MGD Withdrawl from Wells in Northwest Hastings

FIGURE: 4.15

Layne GeoSciences, Inc.

feet. Rest of the wells are generally at sufficient distances so as not to cause significant mutual interference. The combined maximum mutual interference is therefore estimated to be less than 6 feet after 30 days of pumpage. This is 8 percent of the estimated saturated thickness of approximately 80 feet in the area. A well spacing of 1,000 feet therefore appears adequate to accommodate mutual well interference and drawdown in the pumped well. In order to account for nonuniform hydrogeologic conditions at the site, it may be prudent to maintain a spacing of 1,500 feet if possible.

Table 4.1

Distance versus Drawdown Estimates from Theis Analysis

Distance from Pumped Well (ft)	Time (days)	Drawdown (ft)
1000	30	1.5
	60	2
	120	2.4
2000	30	0.7
	60	1.2
	120	1.5

Summary and Conclusions

A groundwater flow model of the aquifer underlying Hastings and vicinity was developed for the study. The model was first calibrated to post-development water levels and then validated to pre-development conditions. The model provides useful information of the various hydrologic components in the subsurface. Simulation results indicate that groundwater withdrawals for municipal and irrigation purposes have been supplemented with increased recharge from the Platte River, infiltration from the Phelps Canal, and irrigation return flow. Development scenarios suggests that additional withdrawals to meet future municipal demands can be supported by the aquifer. Additionally, model simulations indicate that meeting entire municipal demand from proposed and existing wells in northwest Hastings is feasible.

Wellhead Protection Areas (WHPA) were delineated by coupling the groundwater model with a particle tracking simulator. Five, ten, and twenty year capture zones were delineated for existing municipal wells. The delineated zones will enable Hastings Utilities to monitor groundwater quality, thereby providing early warning of encroaching contamination. WHPA delineations also provides a basis for conducting a contamination source inventory and ensuring proper land use within the delineated zones.

Hypothetical simulations were conducted with an advection-dispersion model in order to assess the importance of these processes on migration of nitrates in the subsurface. Simulation results indicate that given adequate historical data for calibration, it is possible to construct a fate and transport model to obtain estimates of future nitrate levels at Hastings Utilities wells.

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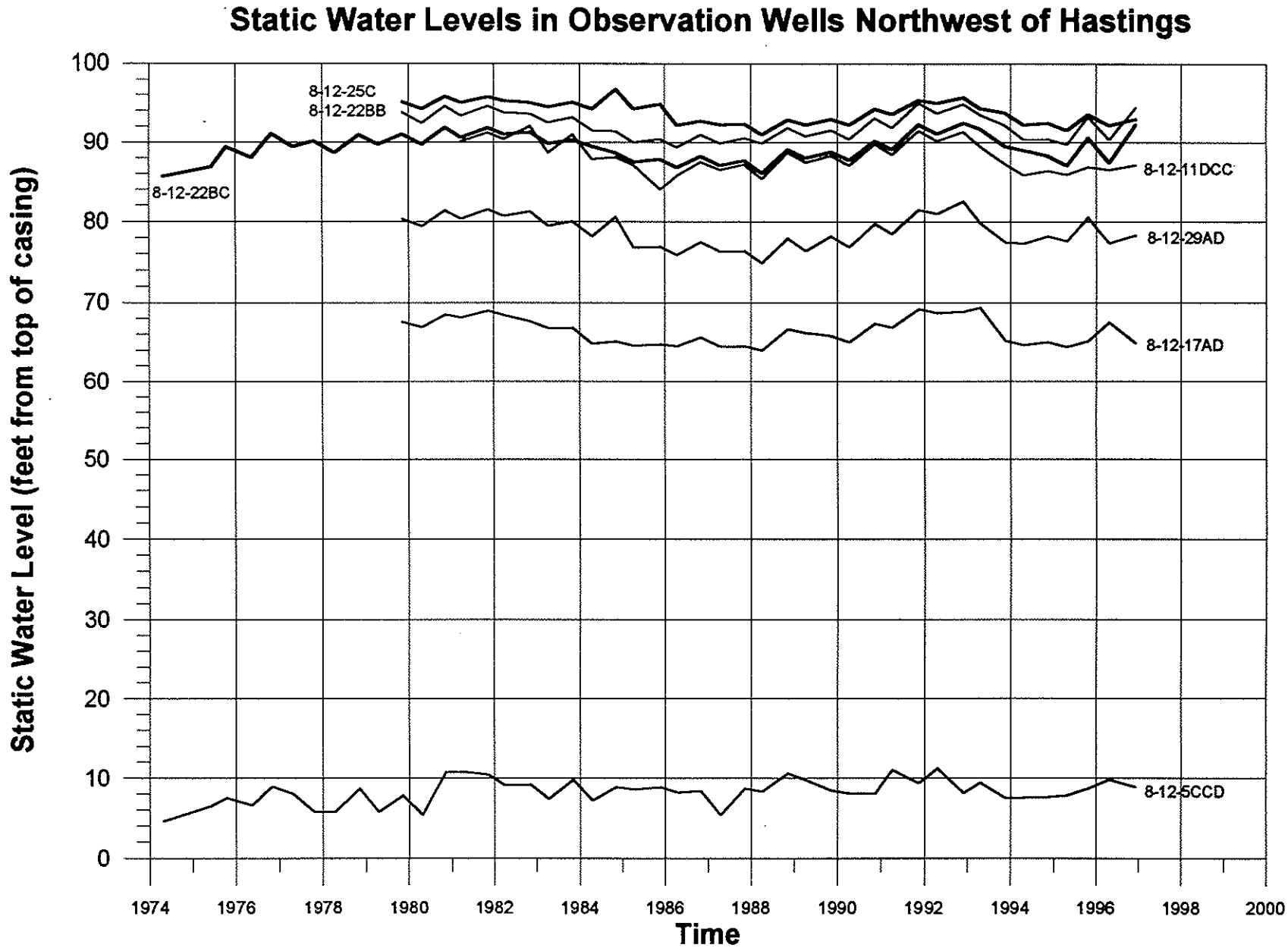
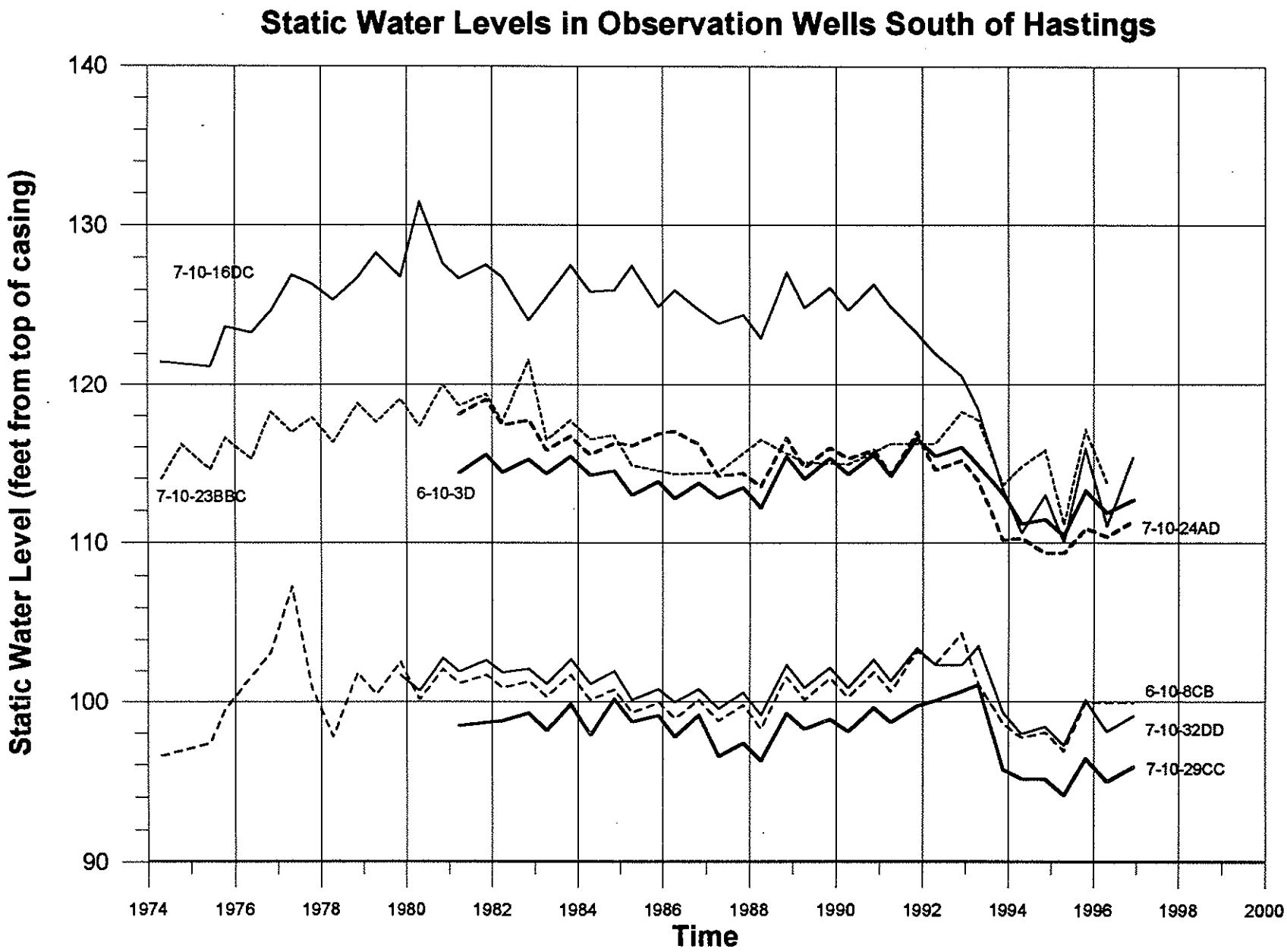
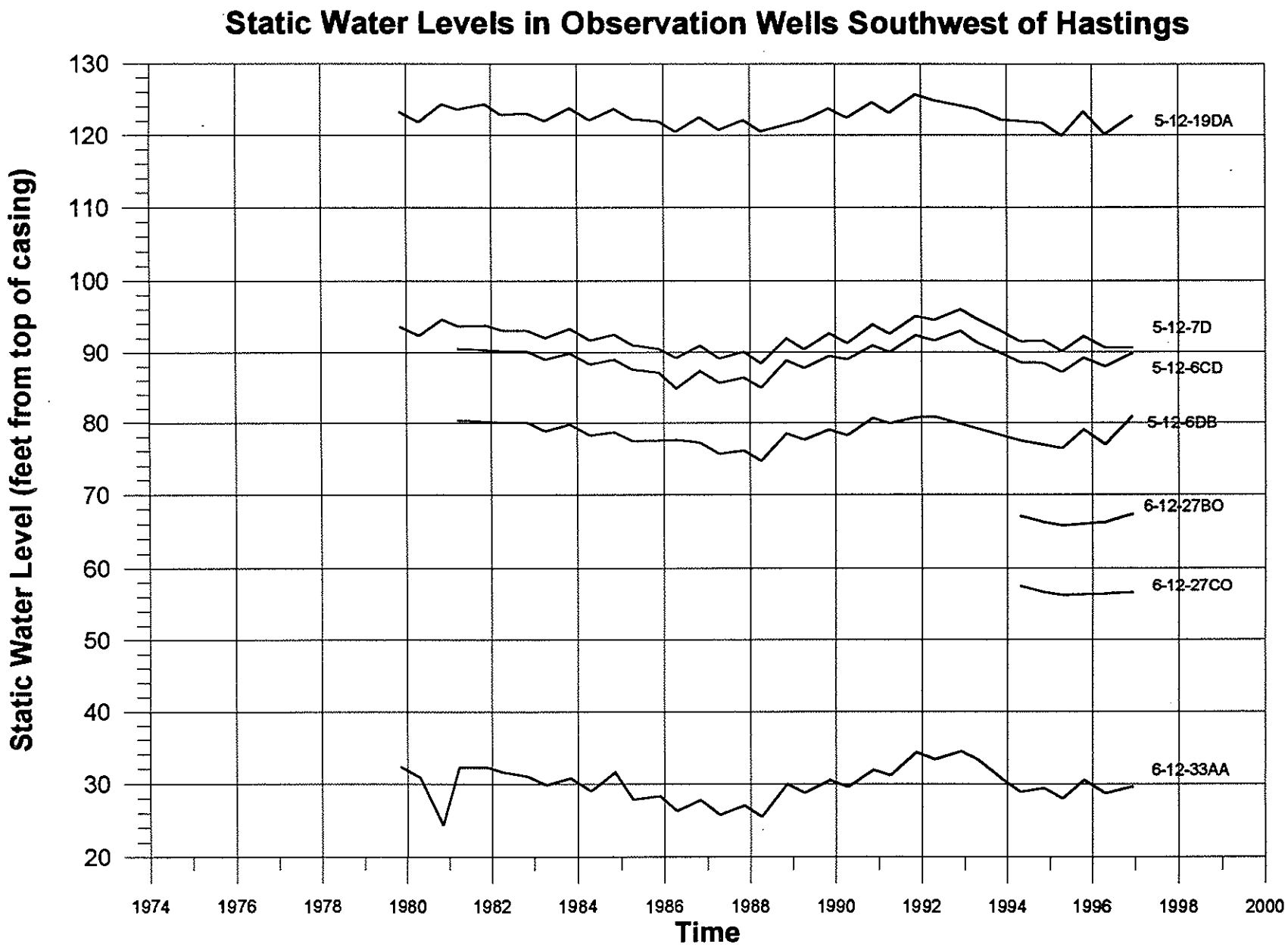
Appendix A

Water level hydrographs at monitoring wells in Adams County. For location of monitoring wells, refer to Figure 2.5. Additional hydrographs presented in Figures 2.6a-e.


Static Water Levels in Observation Wells Northwest of Hastings
Static Water Level (feet from top of casing)
100
90
80
70
60
50
40
30
20
10
0
8-12-25C
8-12-22BB
8-12-22BC
8-12-11DCC
8-12-29AD
8-12-17AD
8-12-5CCD
1974
1976
1978
1980
1982
1984
1986
1988
1990
1992
1994
1996
1998
2000
Time


Static Water Levels in Observation Wells South of Hastings
Static Water Level (feet from top of casing)
140
130
120
110
100
90
1974
1976
1978
1980
1982
1984
1986
1988
1990
1992
1994
1996
1998
2000
Time
7-10-16DC
7-10-23BBC
6-10-3D
7-10-24AD
6-10-8CB
7-10-32DD
7-10-29CC

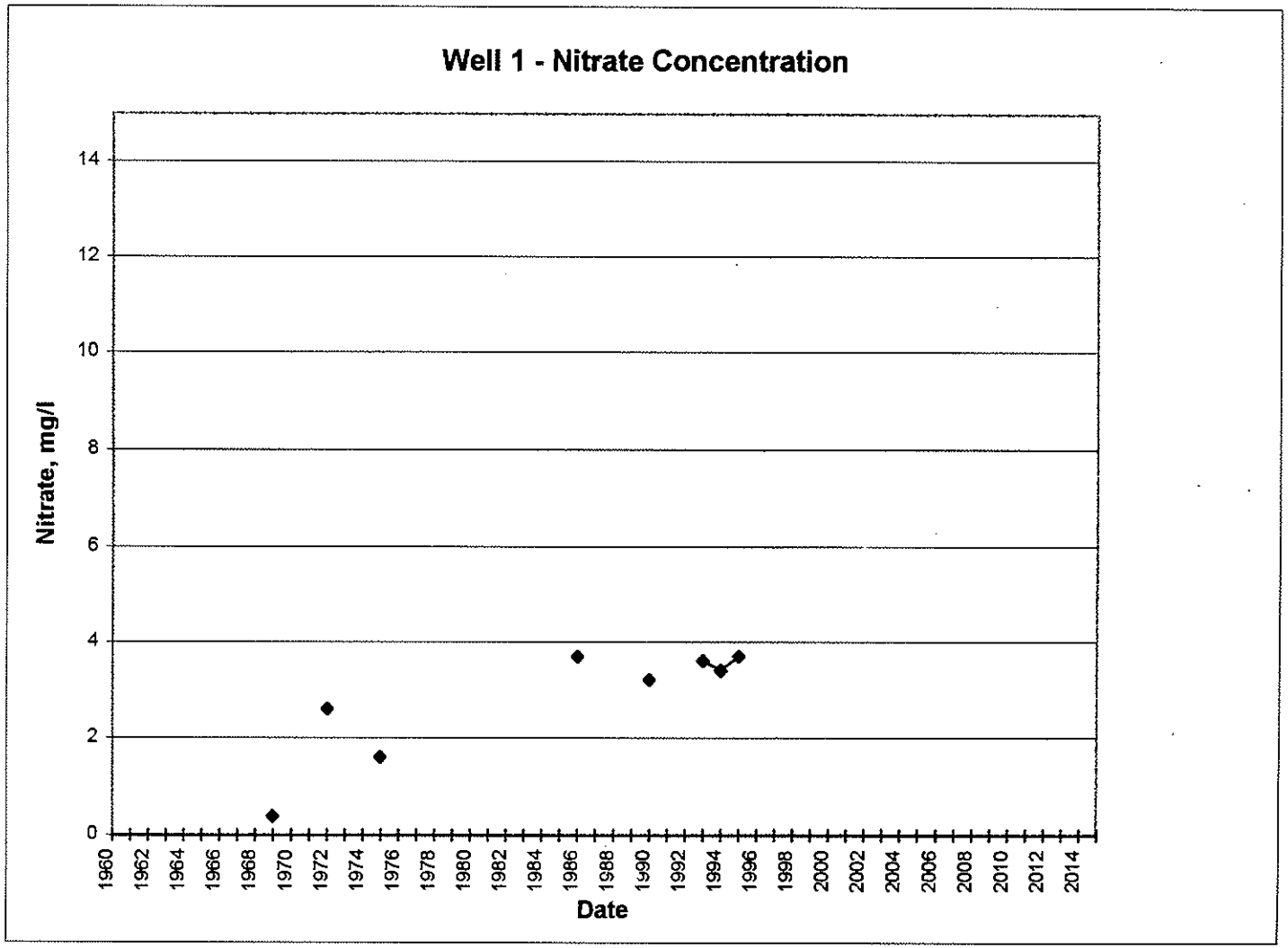

Static Water Levels in Observation Wells Southwest of Hastings
Static Water Level (feet from top of casing)
130
120
110
100
90
80
70
60
50
40
30
20
1974
1976
1978
1980
1982
1984
1986
1988
1990
1992
1994
1996
1998
2000
Time
5-12-19DA
5-12-7D
5-12-6CD
5-12-6DB
6-12-27BO
6-12-27CO
6-12-33AA

Appendix B

Nitrate Concentration in HU wells.

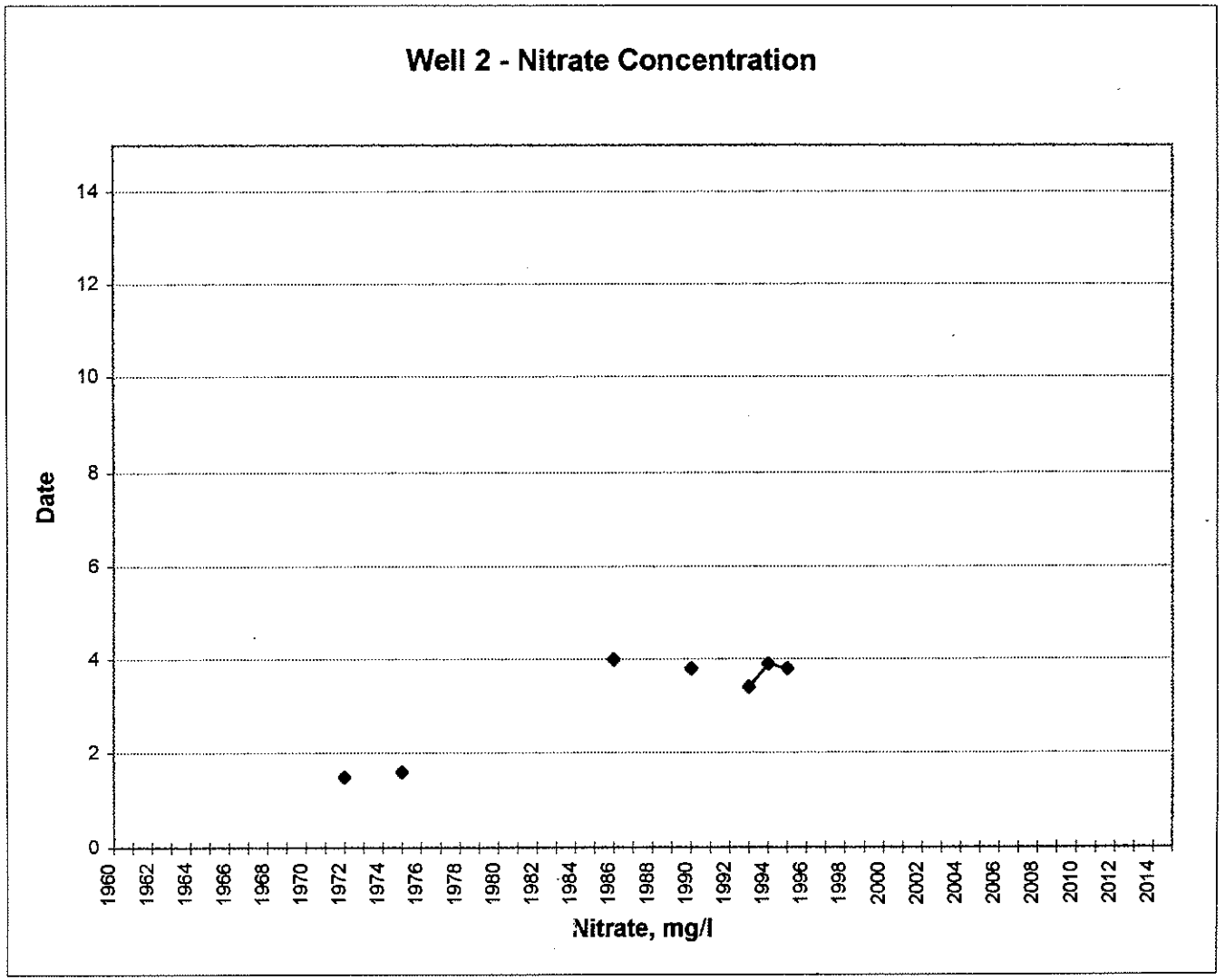
Well 1 - Nitrate Concentration

1969	0.4
4/12/72	2.6
1/18/75	1.6
5/7/86	3.7
6/12/90	3.2
6/8/93	3.6
4/5/94	3.4
5/2/95	3.6
5/4/95	3.7



Well 2 - Nitrate Concentration

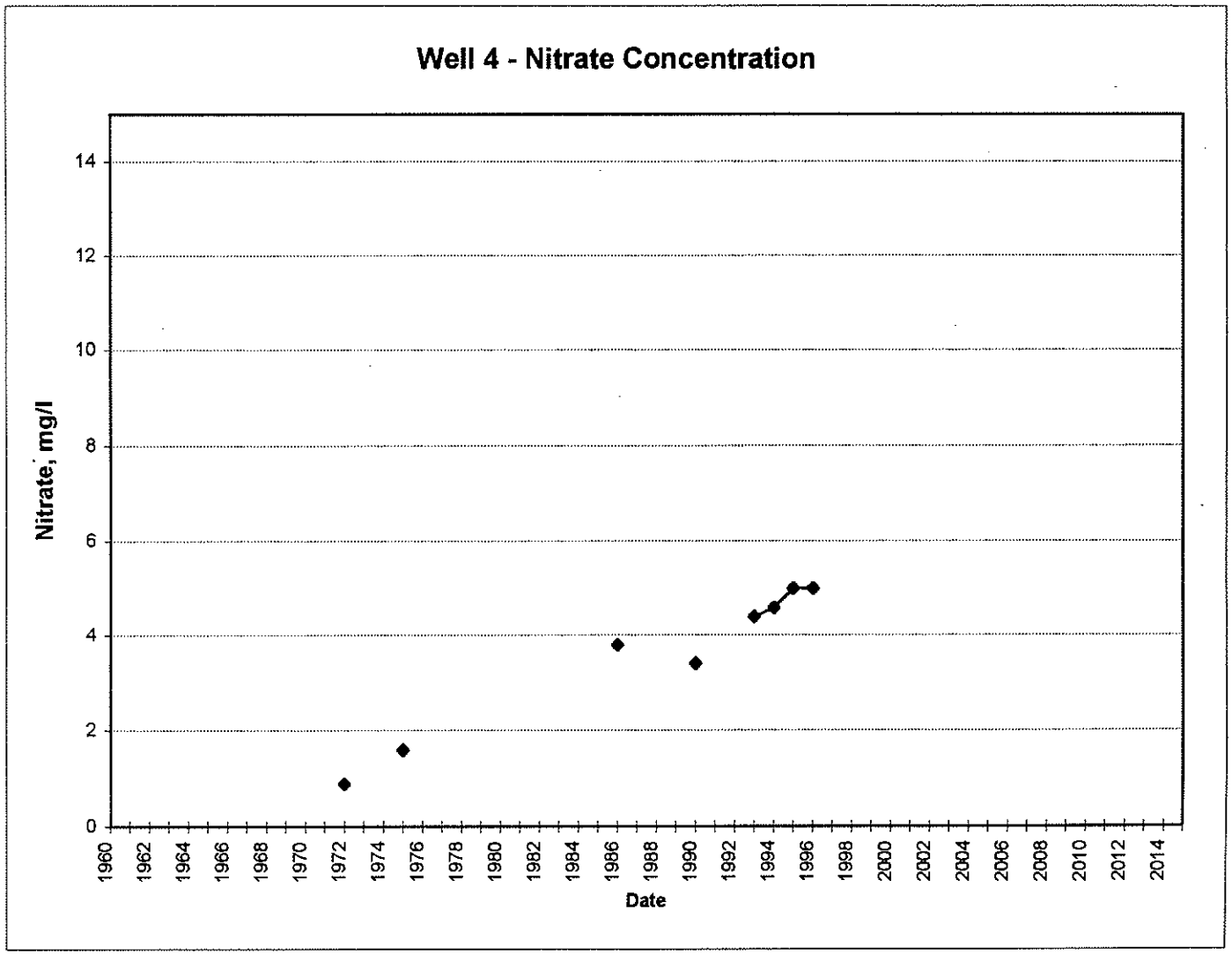
4/12/72	1.5
11/18/75	1.6
5/7/86	4
6/12/90	3.8
6/8/93	3.4
4/5/94	3.9
5/2/95	3.6
5/4/95	3.8



89 (2)

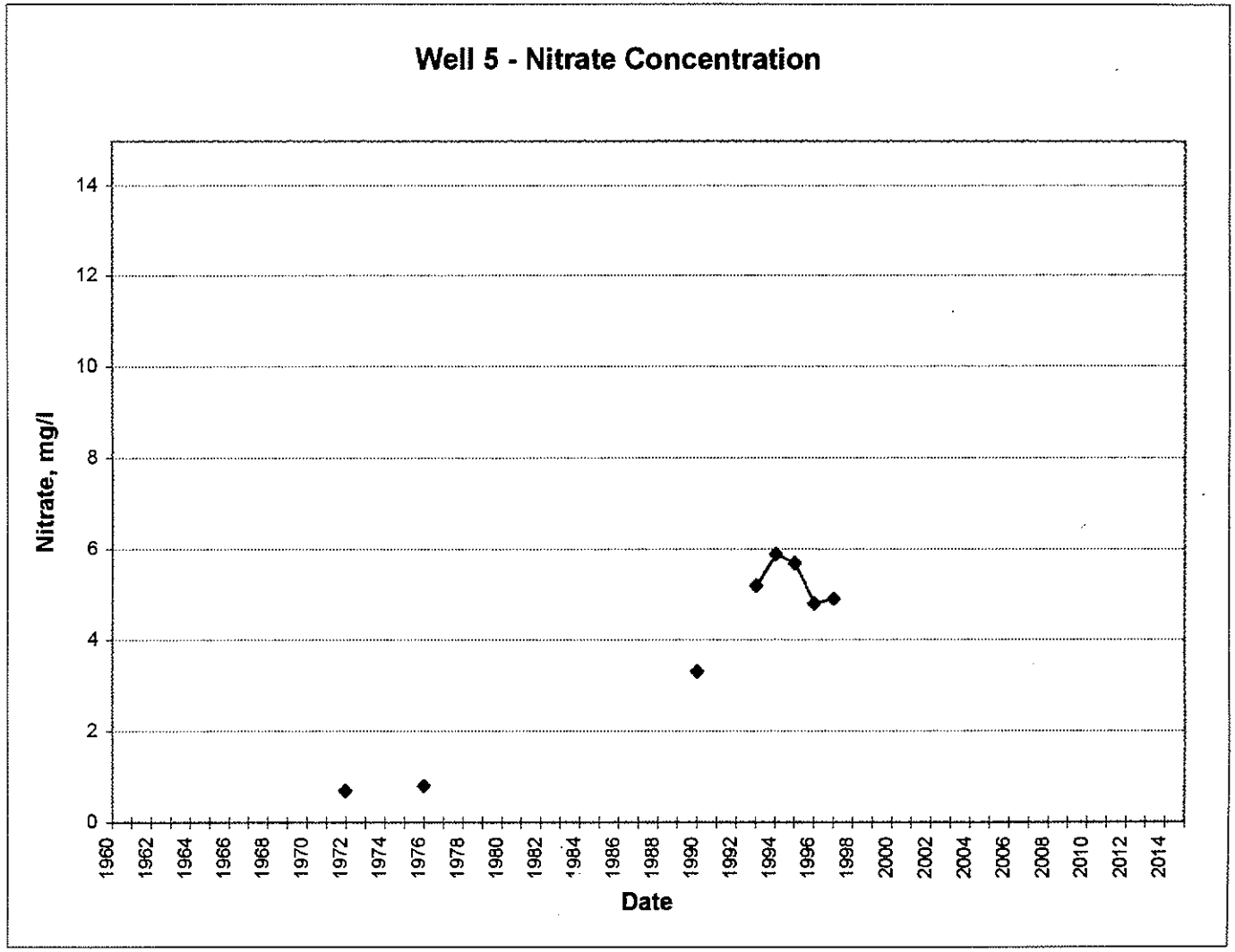
Well 4 - Nitrate Concentration

4/12/72	0.9
11/18/75	1.6
5/7/86	3.8
6/12/90	3.4
6/8/93	4.4
4/5/94	4.6
5/2/95	4.8
5/4/95	5
2/7/96	4.8
8/28/96	5



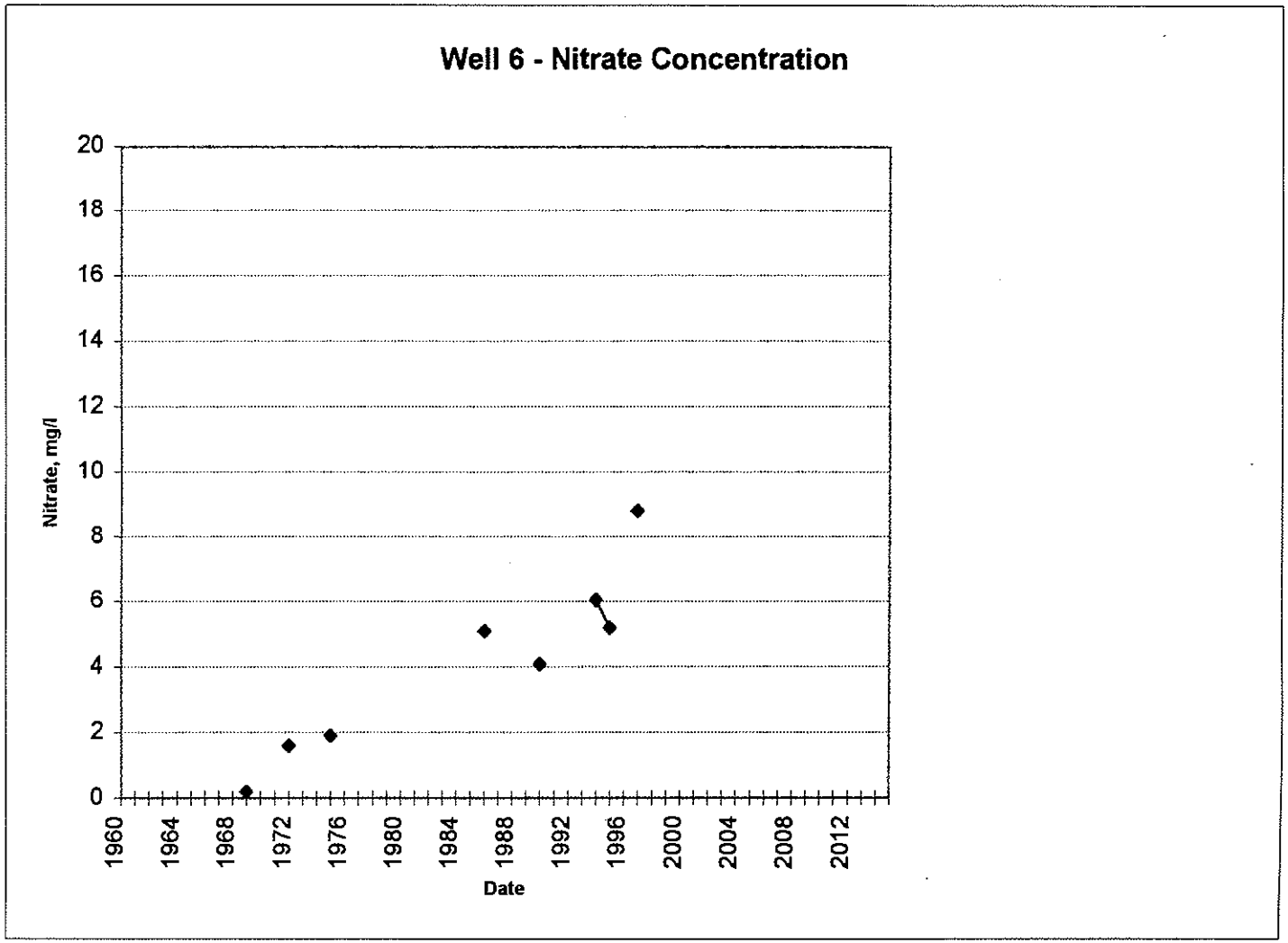
Well 5 - Nitrate Concentration

4/12/72	0.7
8/11/76	0.8
6/11/90	3.3
6/8/93	5.2
11/29/93	3.8
2/15/94	3.8
7/26/94	3.5
11/1/94	5.9
12/20/94	3.2
1/17/95	3.7
5/2/95	3.7
10/26/95	4.8
12/13/95	5.7
2/7/96	3.8
6/25/96	4.8
8/28/96	4.2
4/28/97	4.6
7/21/97	4.9



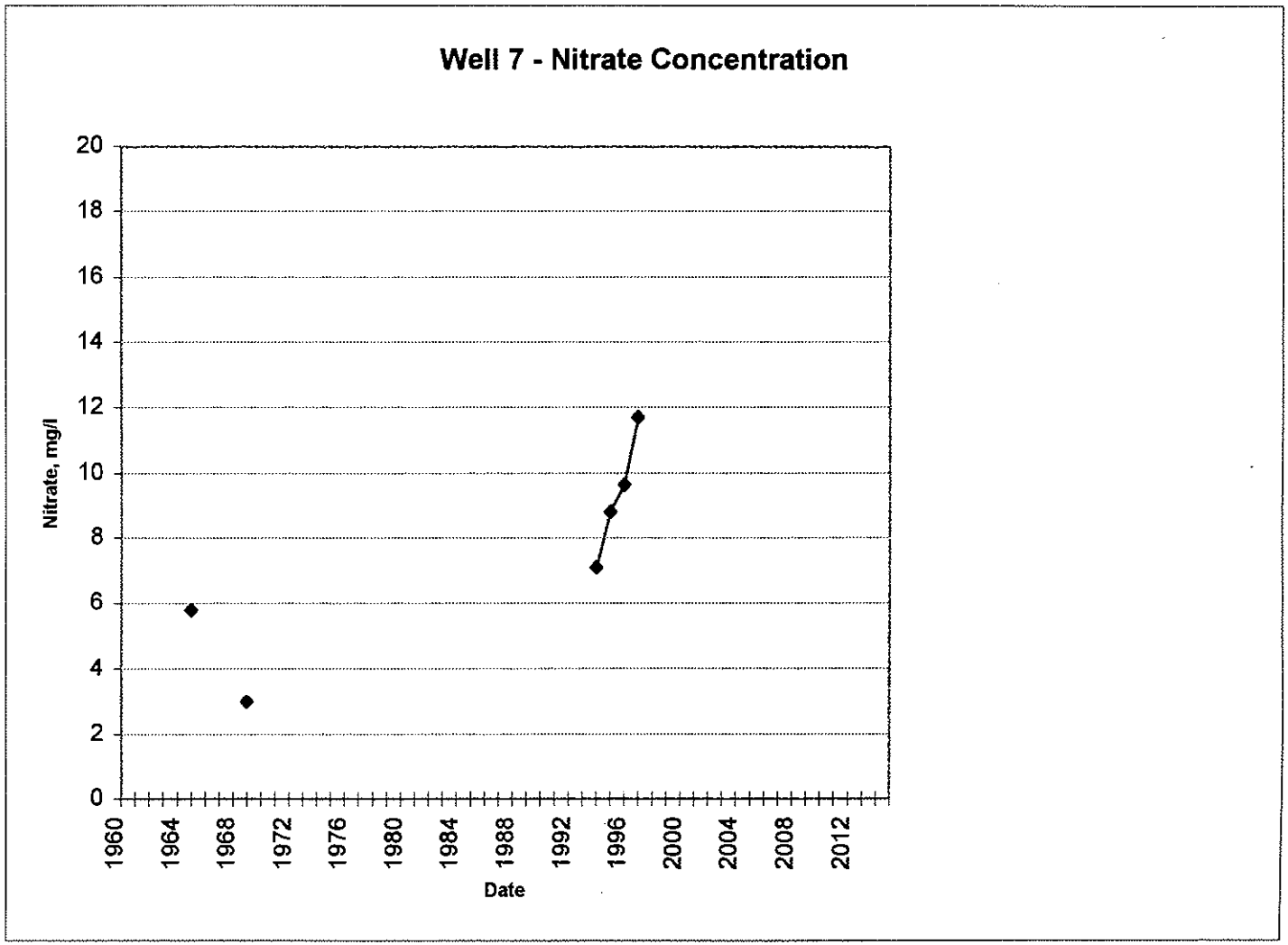
Well 6 - Nitrate Concentration

Date	Nitrate, mg/l
1969	0.2
1972	1.6
1975	1.9
1986	5.1
1990	4.1
7/26/94	5.8
8/9/94	6.3
5/3/95	5.3
5/9/95	5.9
7/28/95	4.3
4/28/97	7.9
7/21/97	8.8



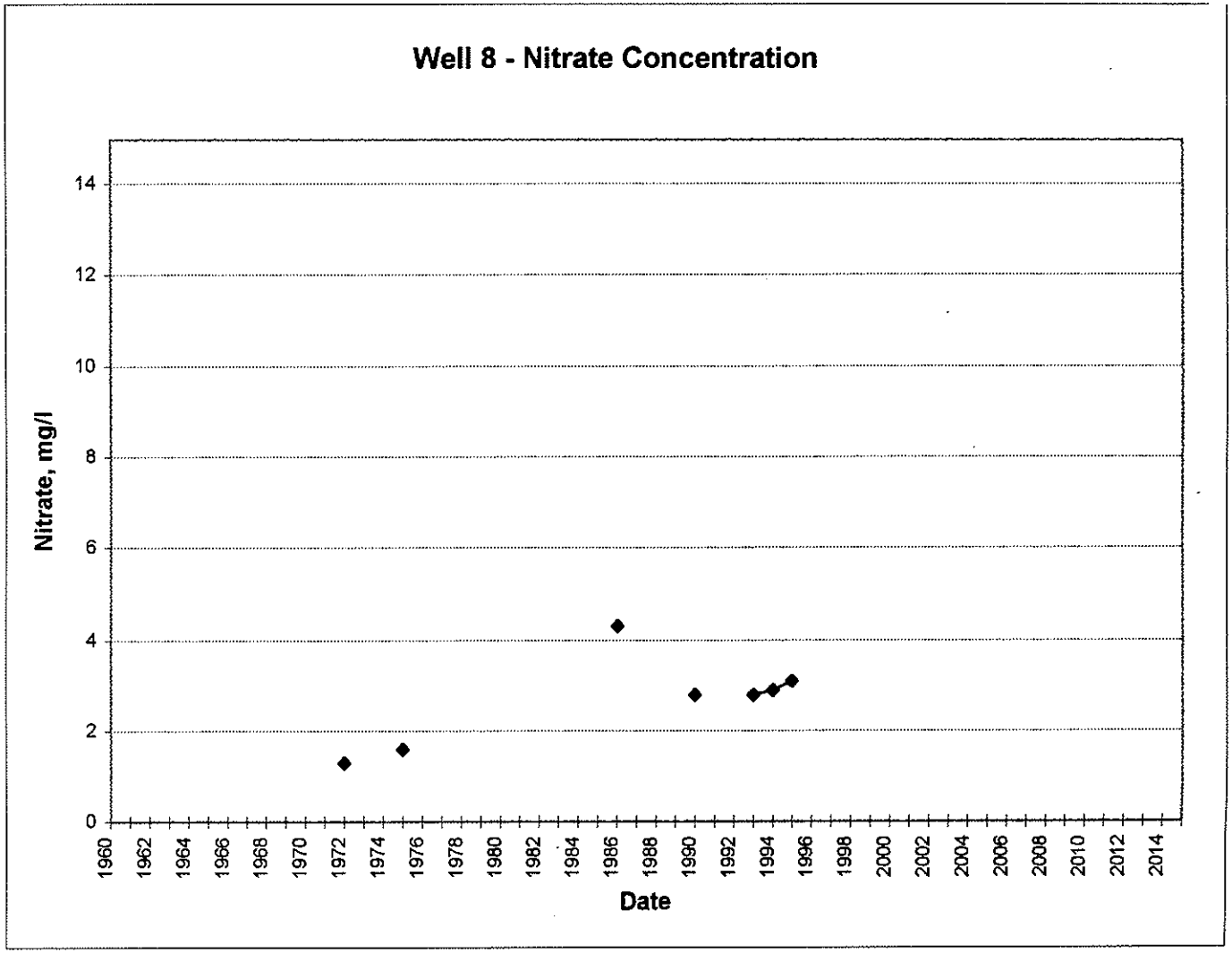
Well 7 - Nitrate Concentration

Date	Nitrate, mg/l
5/13/65	5.8
4/21/69	3.0
2/15/94	8.9
7/26/94	5.1
11/1/94	7.6
12/20/94	6.8
1/17/95	8.2
5/2/95	8.5
8/30/95	9.6
10/23/95	8.7
12/13/95	9.2
2/7/96	9.1
8/28/96	10.2
4/28/97	10.8
5/5/97	10.4
7/21/97	11.7



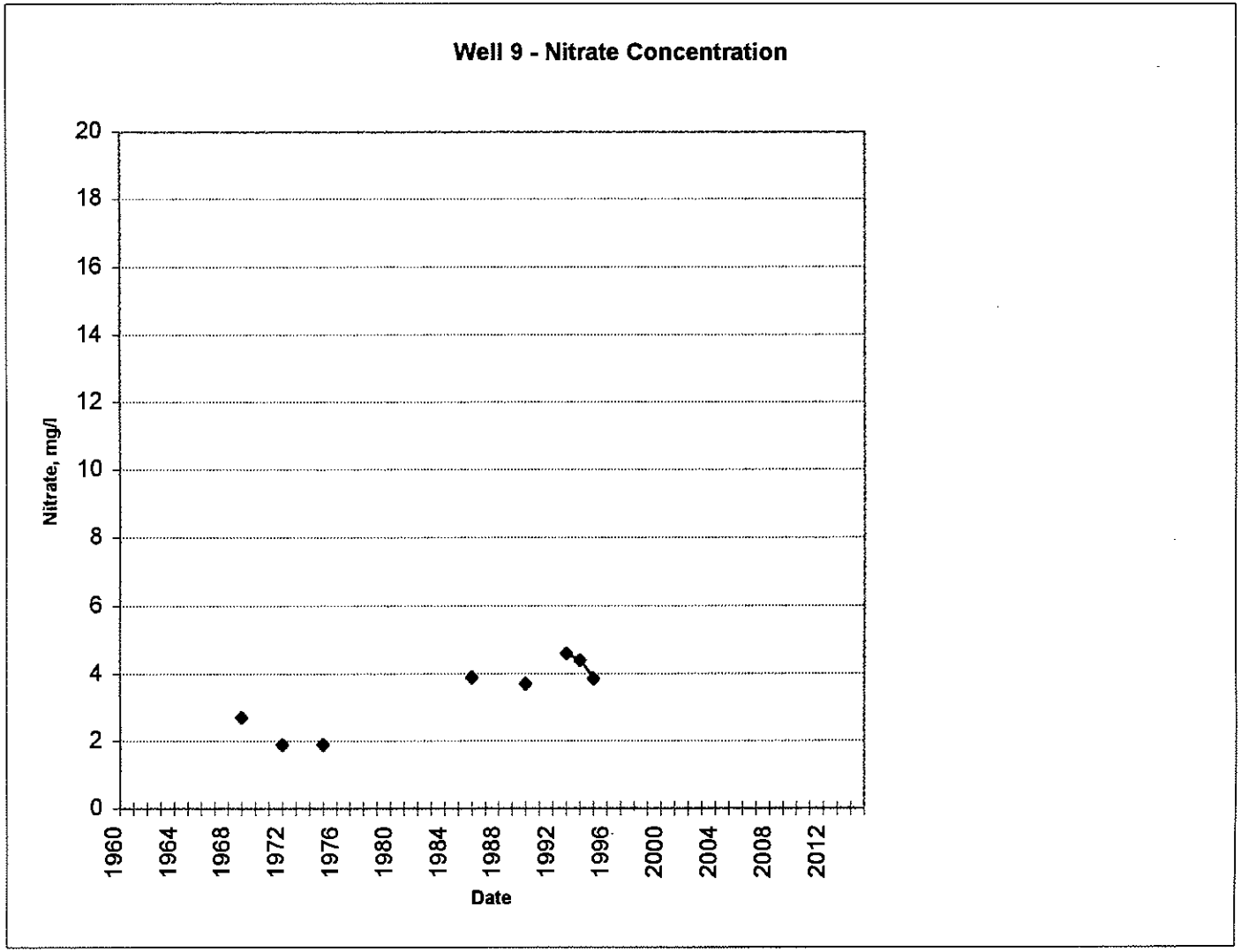
Well 8 - Nitrate Concentration

4/12/72	1.3
11/18/75	1.6
5/7/86	4.3
6/13/90	2.8
6/8/93	2.8
4/5/94	2.9
5/3/95	3.1
5/3/95	2.8



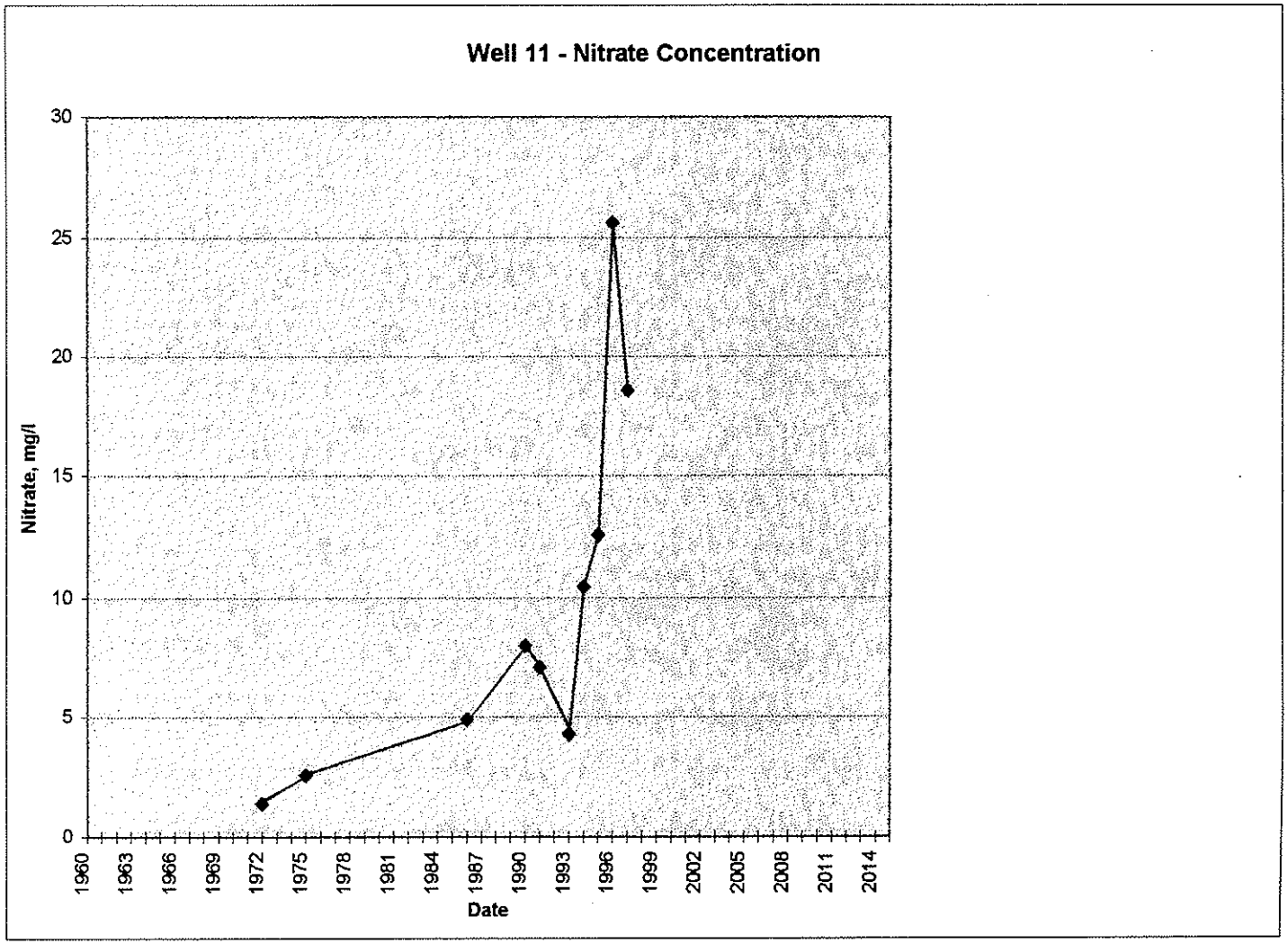
Well 9 - Nitrate Concentration

Date	Nitrate, mg/l
1969	2.7
1972	1.9
1975	1.9
1986	3.9
1990	3.7
1993	4.6
4/5/94	4.4
5/2/95	3.7
5/4/95	4



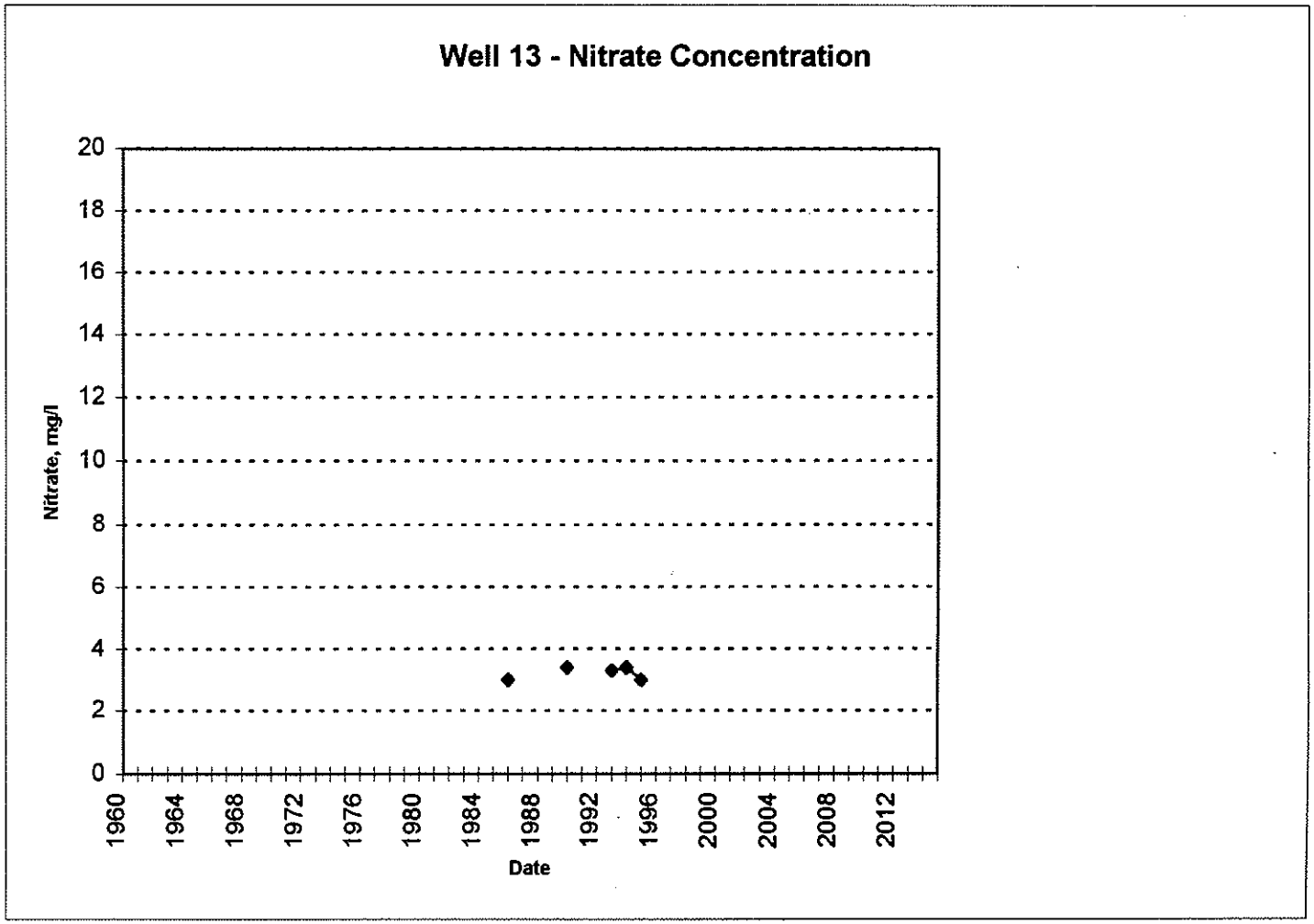
Well 11 - Nitrate concentration

DATE	Nitrate, mg/l
1972	1.4
1975	2.6
1986	4.9
1990	8
1991	7.1
6/8/93	4.3
4/6/94	9
12/19/94	11.9
1/17/95	12.7
1/30/95	9.8
2/27/95	13.8
5/2/95	14.2
2/7/96	5.8
6/25/96	25.6
4/28/97	18.6
7/21/97	11.8



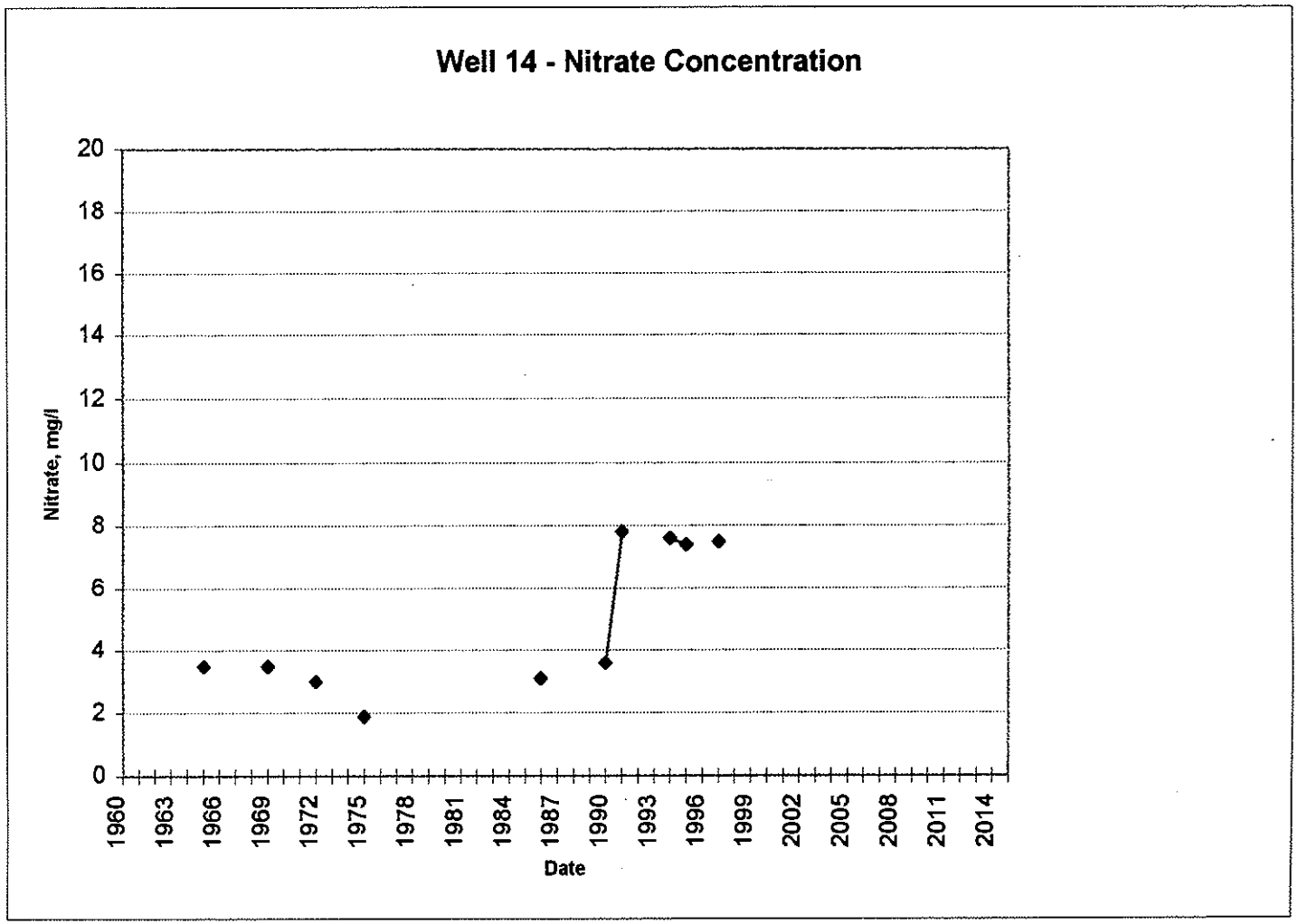
Well 13 - Nitrate Concentration

Date	Nitrate, mg/l
1986	3
1990	3.4
6/8/93	3.3
4/5/94	3.4
5/2/95	2.9
5/4/95	3



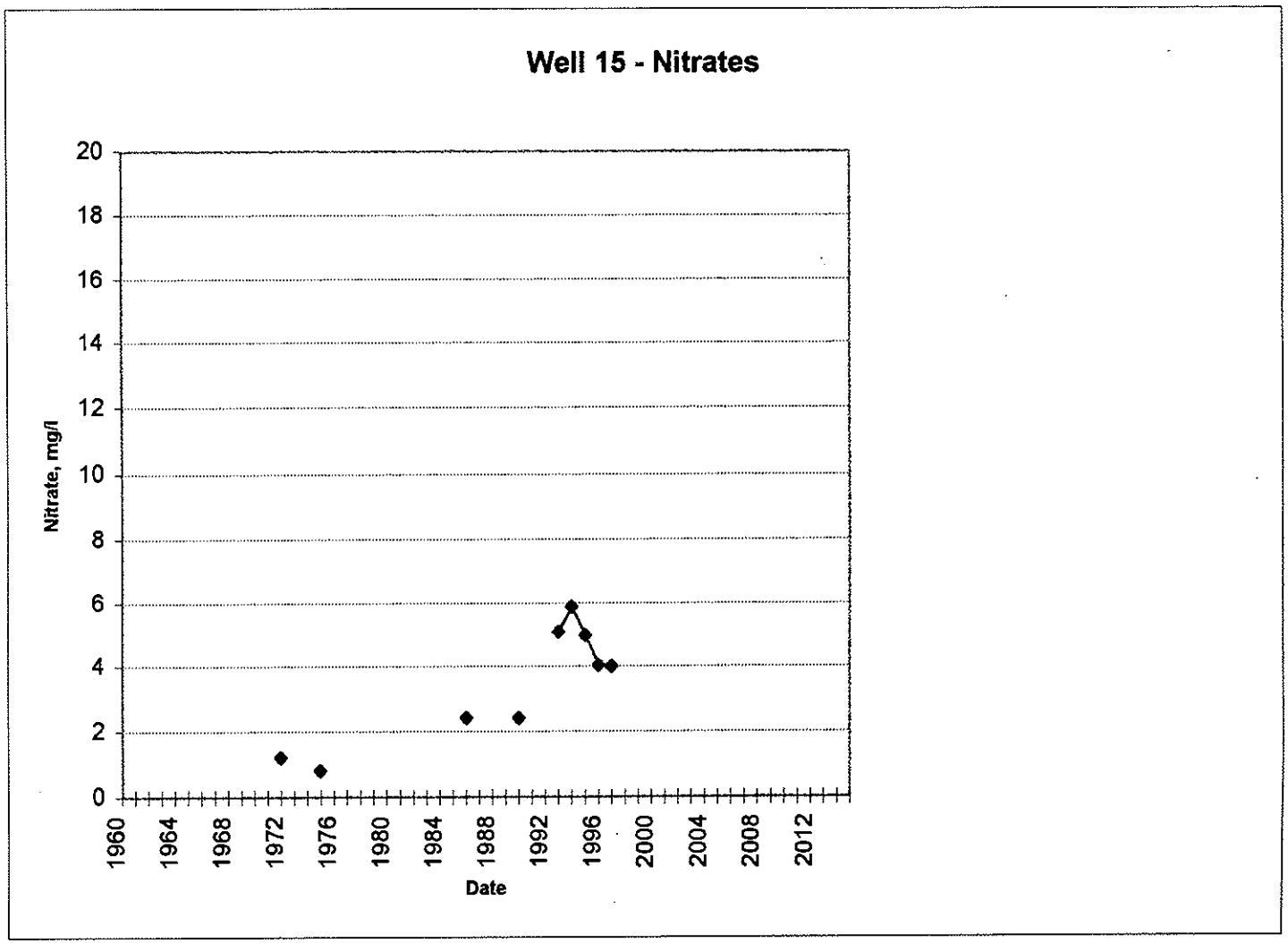
Well 14 - Nitrate Concentration

Date	Nitrate, mg/l
5/13/65	35
1969	3.5
1972	3
1975	1.9
1986	3.1
1990	3.6
1991	7.8
6/22/94	4.8
7/26/94	7.6
5/3/95	7
5/4/95	7.8
7/28/95	3.9
4/28/97	7.5
7/21/97	5.6



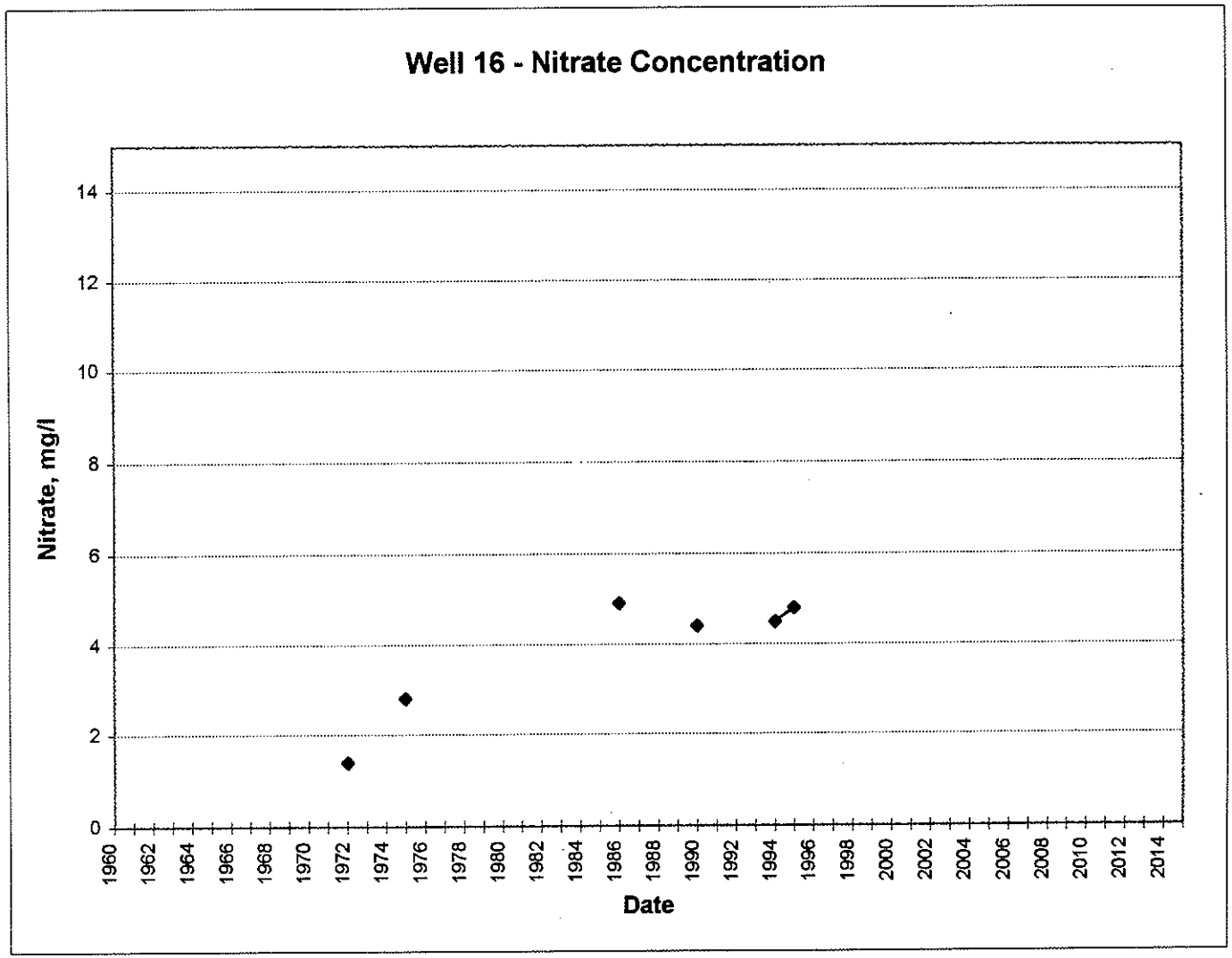
Well 15 - Nitrate Concentraion

Date	Nitrate, mg/l
1972	1.2
1975	0.8
1986	2.4
1990	2.4
6/8/93	5.1
2/15/94	7
7/26/94	5.8
11/1/94	6
12/20/94	4.9
1/17/95	5.7
5/2/95	5.4
8/29/95	5.2
10/23/95	4.4
12/13/95	4.2
2/7/96	4.3
8/28/96	3.8
4/28/97	4
7/21/97	3.9



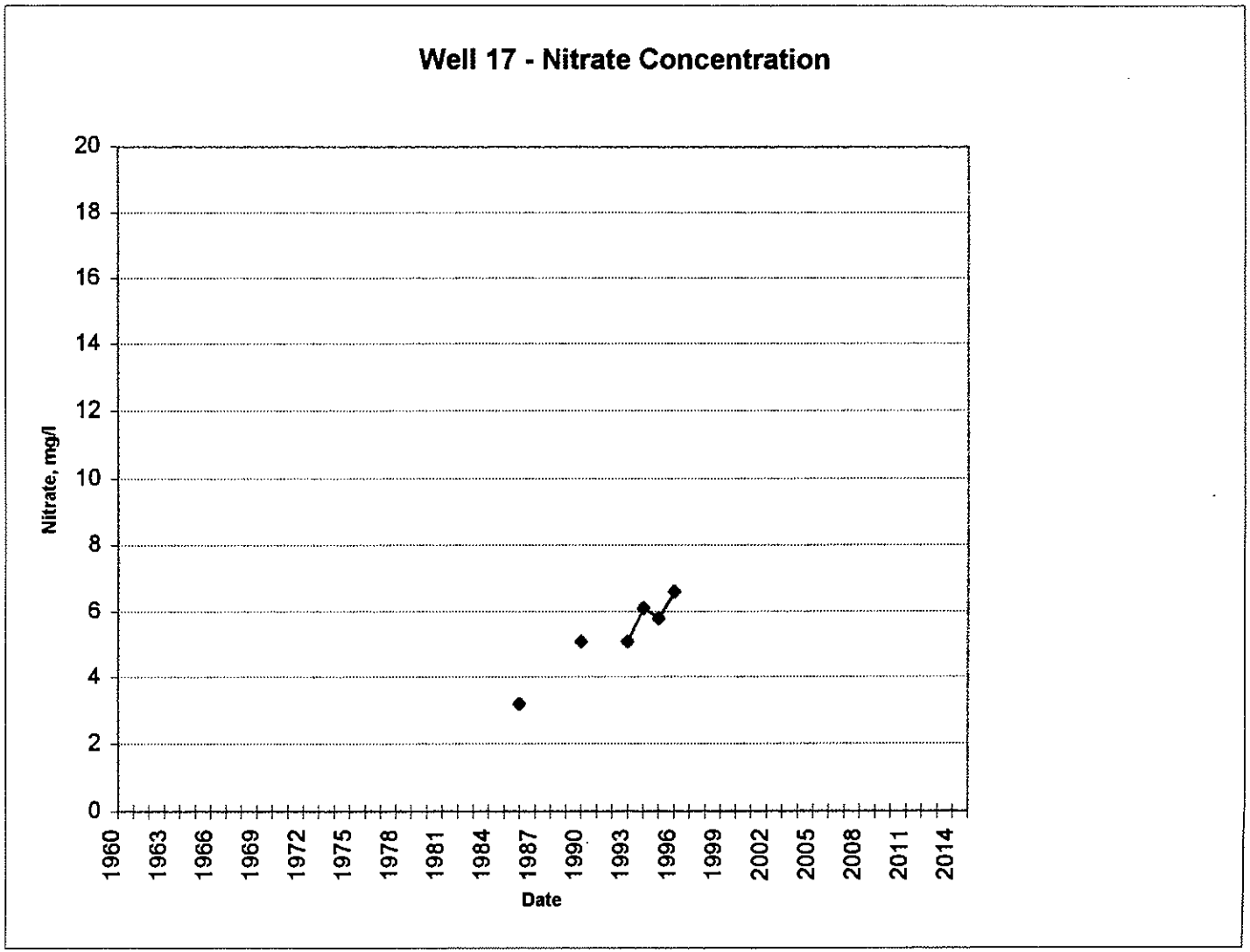
Well 16 - Nitrate Concentration

4/12/72	1.4
11/18/75	2.8
5/7/86	4.9
6/11/90	4.4
4/5/94	4.4
6/22/95	4.5
5/2/95	4.6
5/4/95	4.8



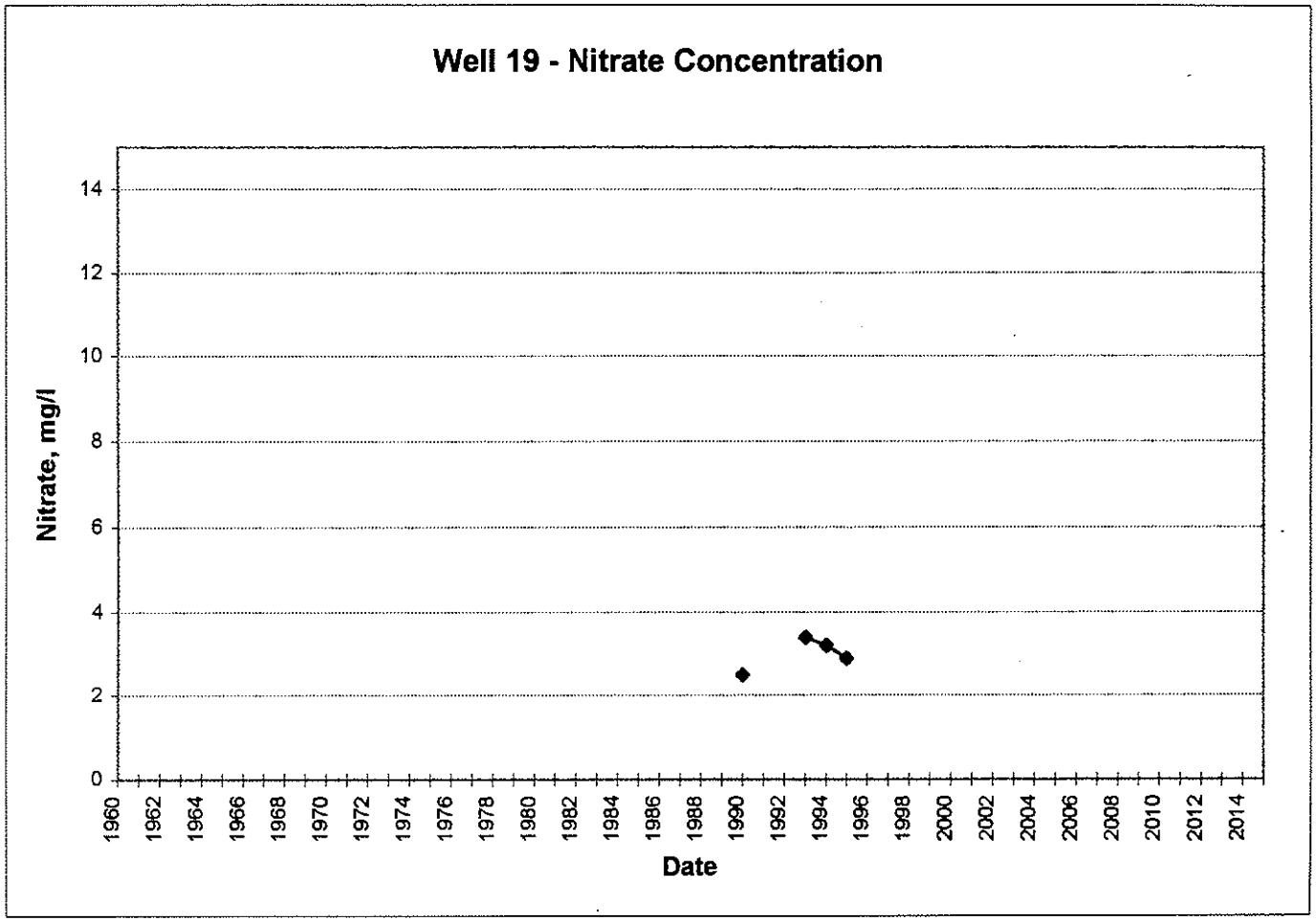
Well 17 - Nitrate Concentration

Date	Nitrate, mg/l
1986	3.2
1990	5.1
6/2/93	5.1
2/15/94	6.1
7/26/94	6.2
11/1/94	4.6
12/20/94	6.1
1/17/95	5.9
5/2/95	5.8
8/31/95	5.6
10/23/95	5.8
12/13/95	6
2/7/96	6.4
8/28/96	6.8
7/21/97	7



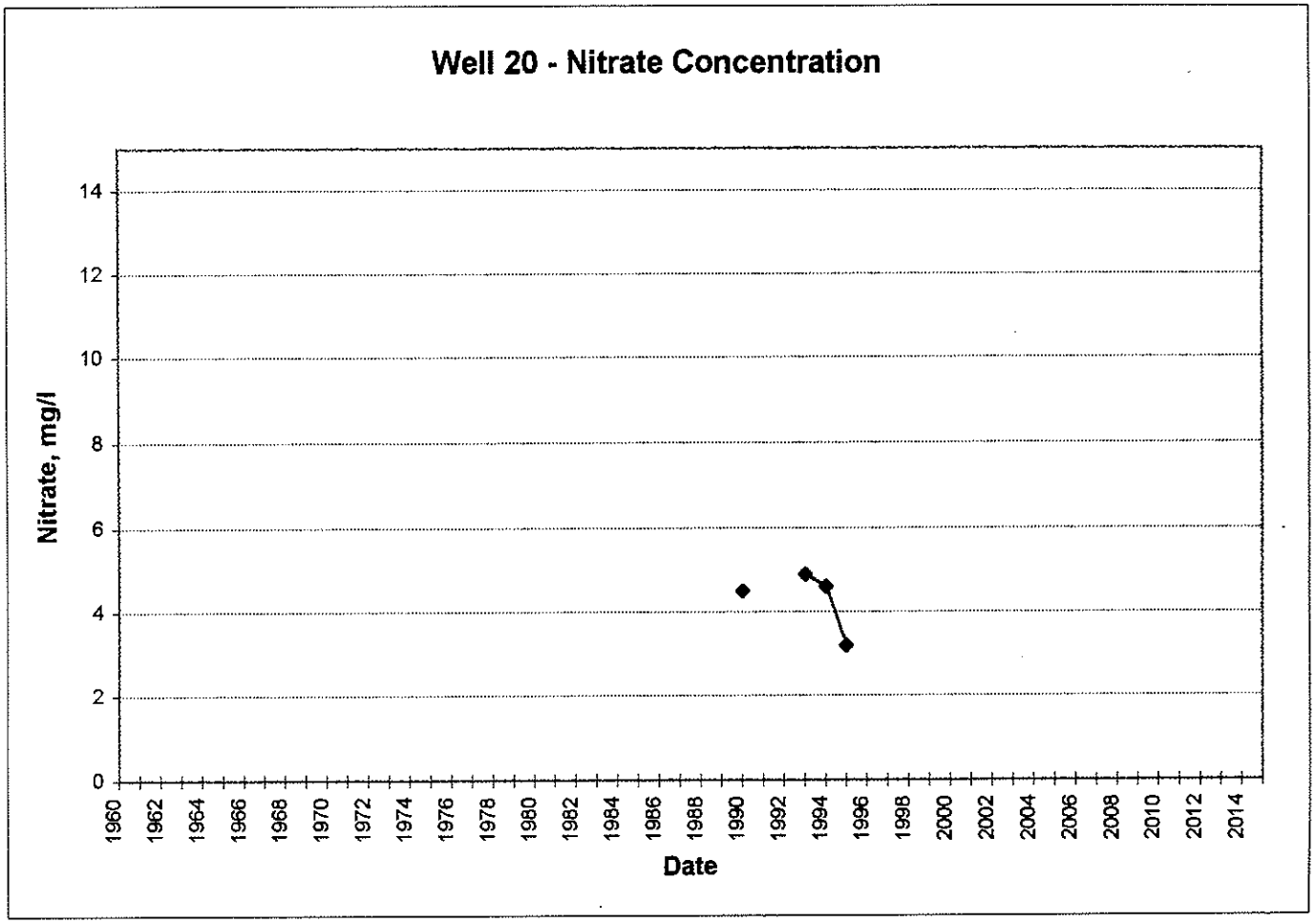
Well 19 - Nitrate Concentration

6/11/90	2.5
6/8/93	3.4
4/5/94	3.2
5/2/95	2.9
5/4/95	2.9



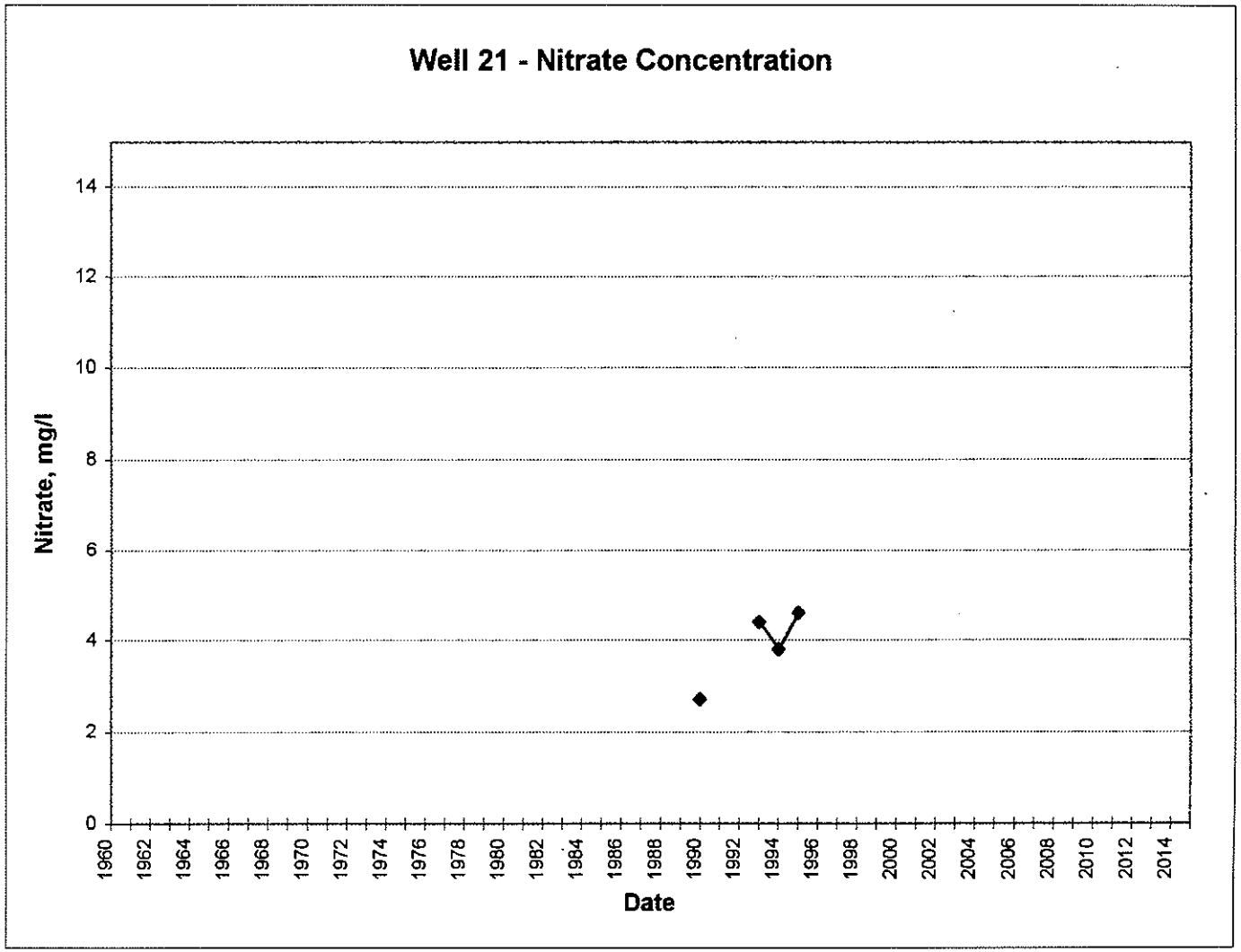
Well 20 - Nitrate Concentration

6/13/90	4.5
6/8/93	4.9
6/8/94	4.6
5/3/95	3
5/3/95	3.2



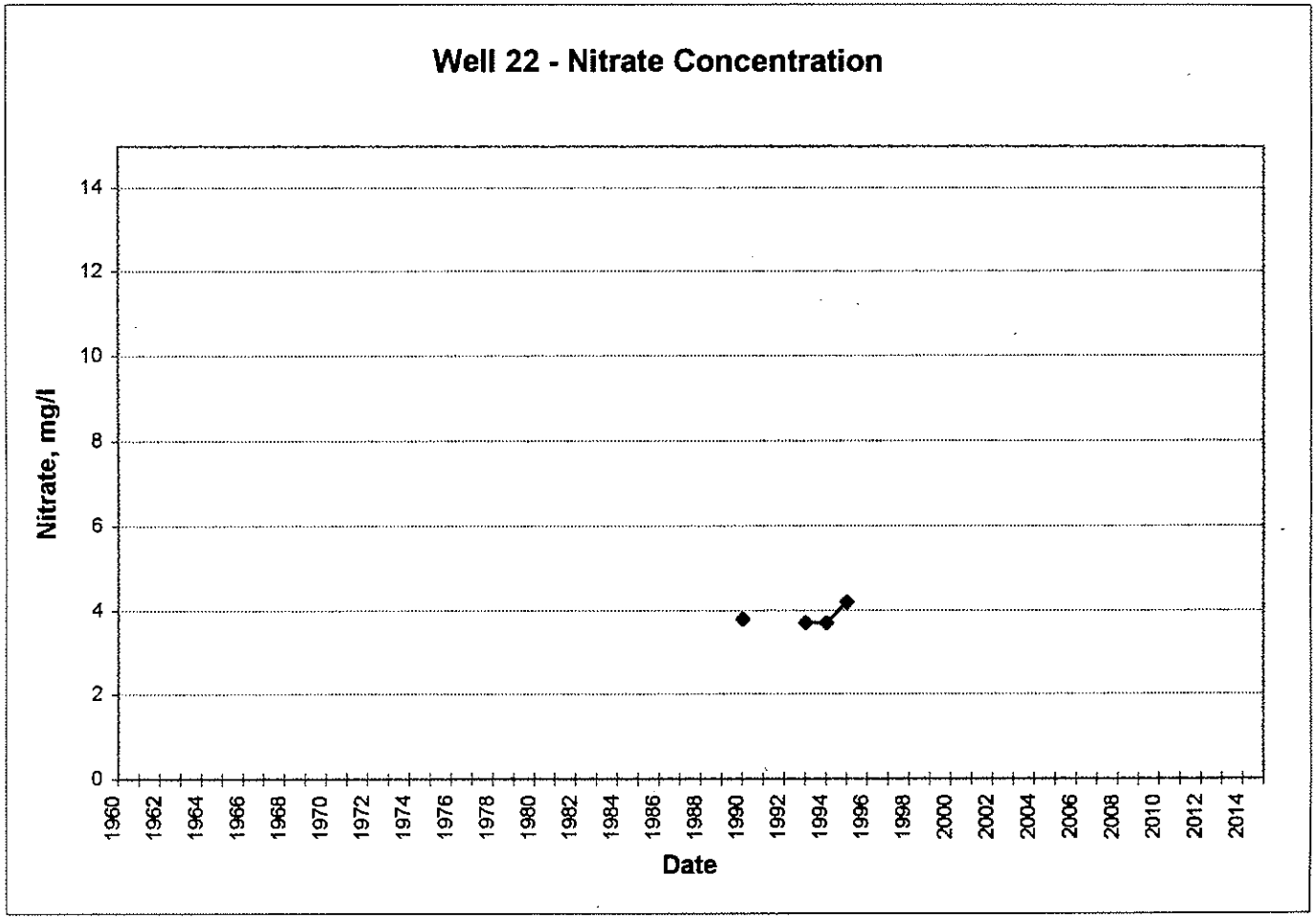
Well 21 - Nitrate Concentration

6/13/90	2.7
6/18/93	4.4
4/5/94	3.8
5/2/95	4.1
5/4/95	4.6



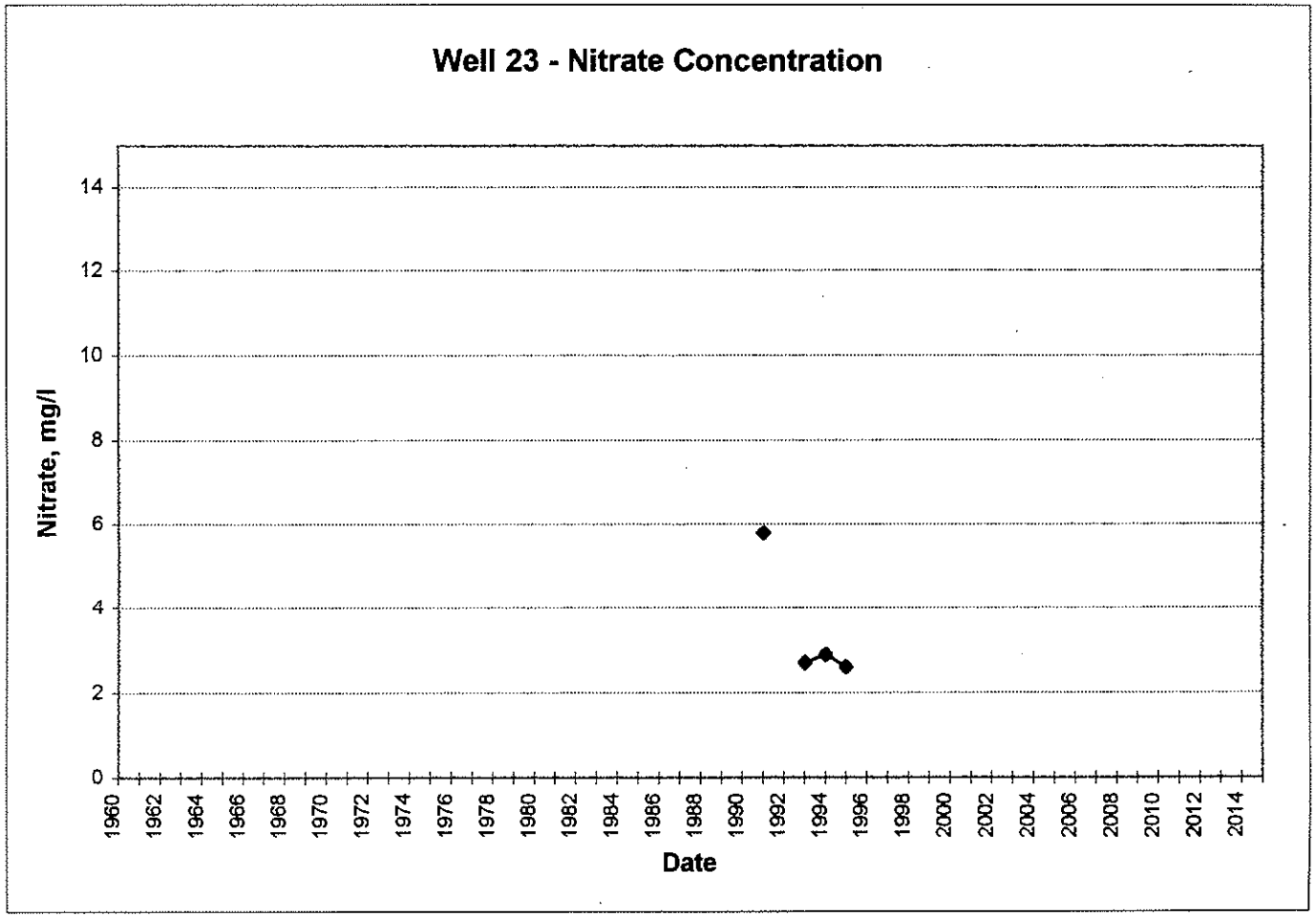
Well 22 - Nitrate Concentration

2/13/90	3.8
6/8/93	3.7
4/5/94	3.5
8/9/95	3.7
5/2/95	4
5/4/95	4.2



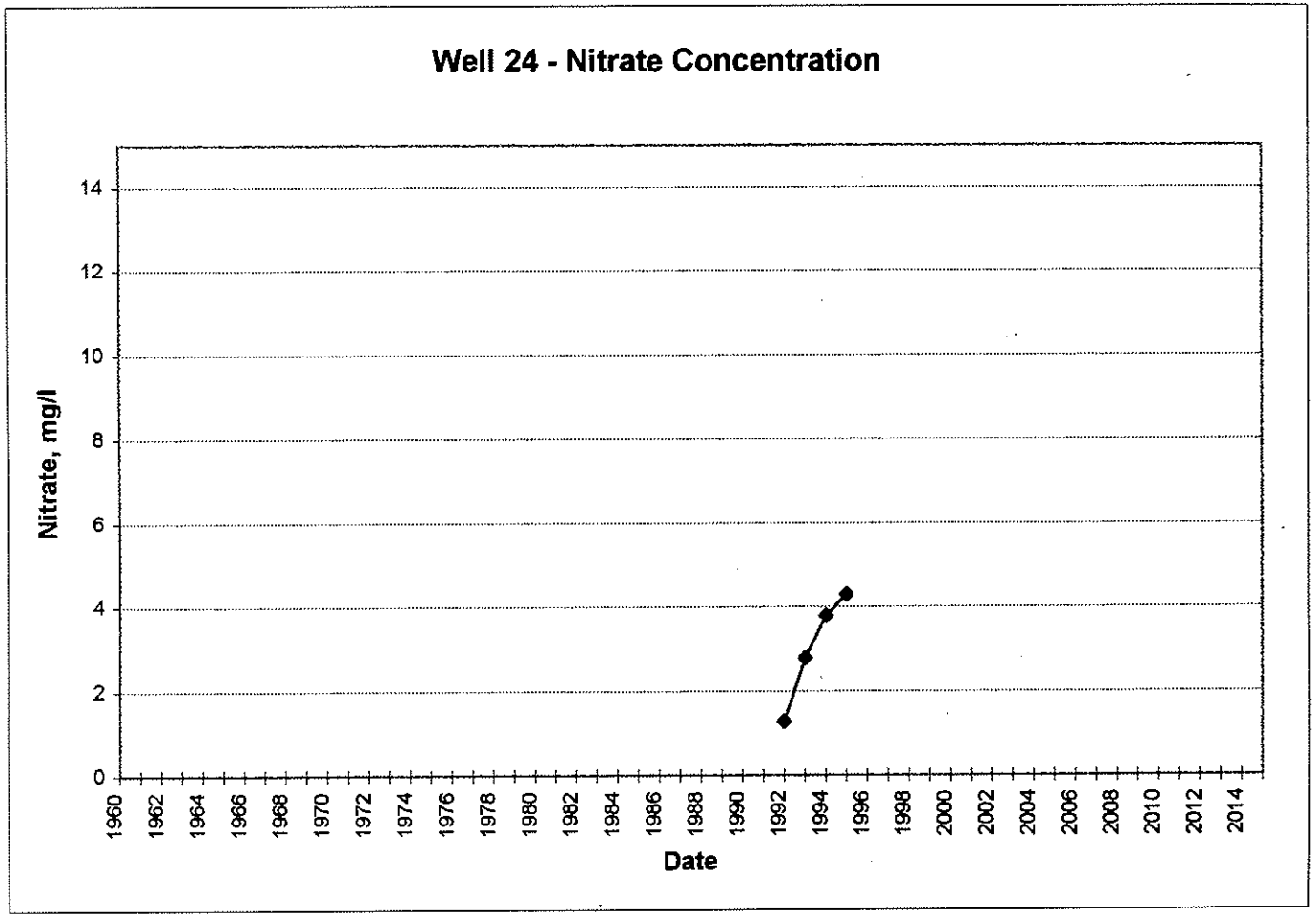
Well 23 - Nitrate Concentration

3/20/91	5.8
6/8/93	2.7
4/5/94	2.4
8/9/95	2.9
5/2/95	2.6
5/2/95	2.5



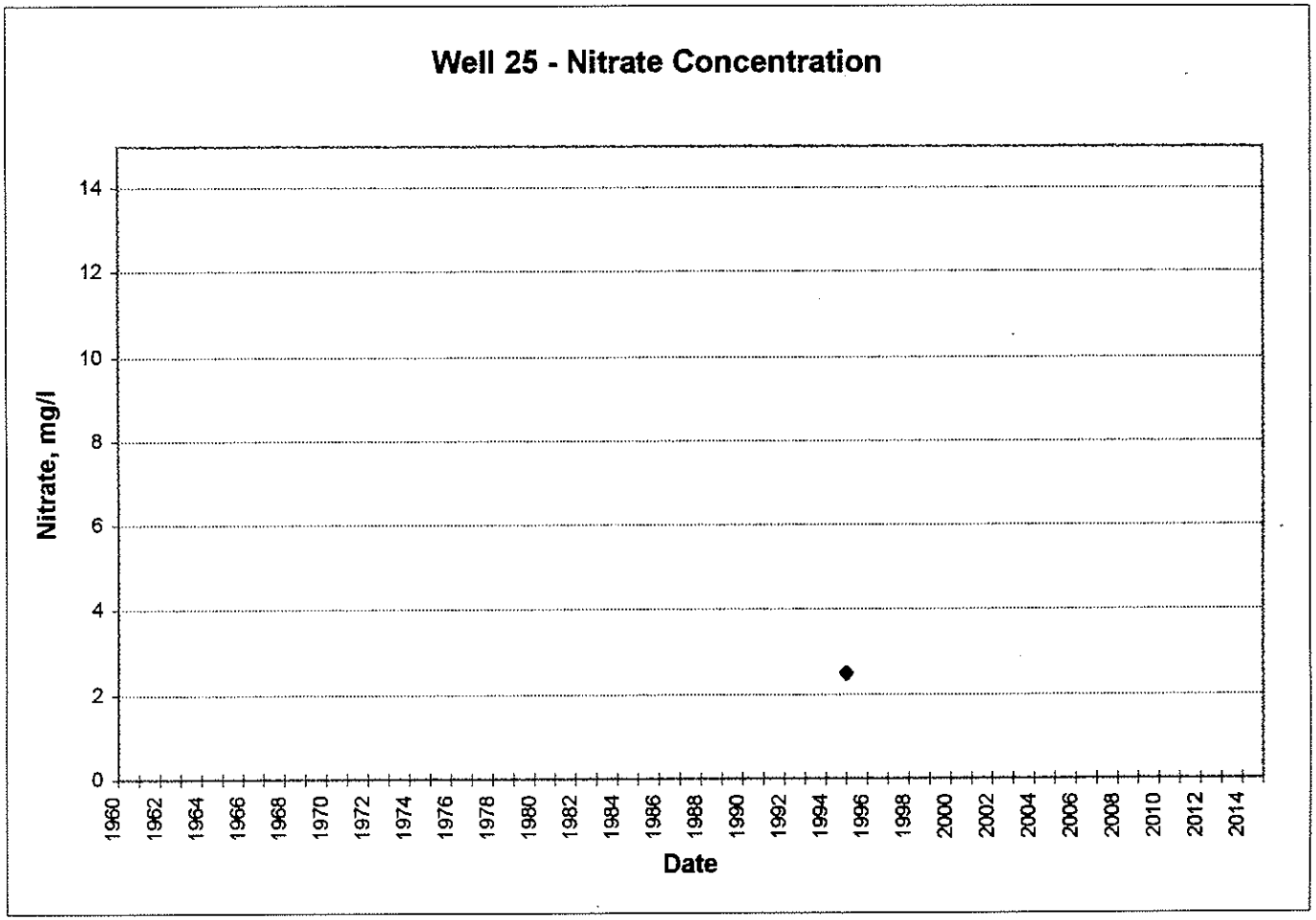
Well 24 - Nitrate Concentration

6/11/92	1.3
1/19/93	1.7
6/8/93	2.8
4/5/94	3
8/9/94	3.8
5/2/95	4
5/4/95	4.3



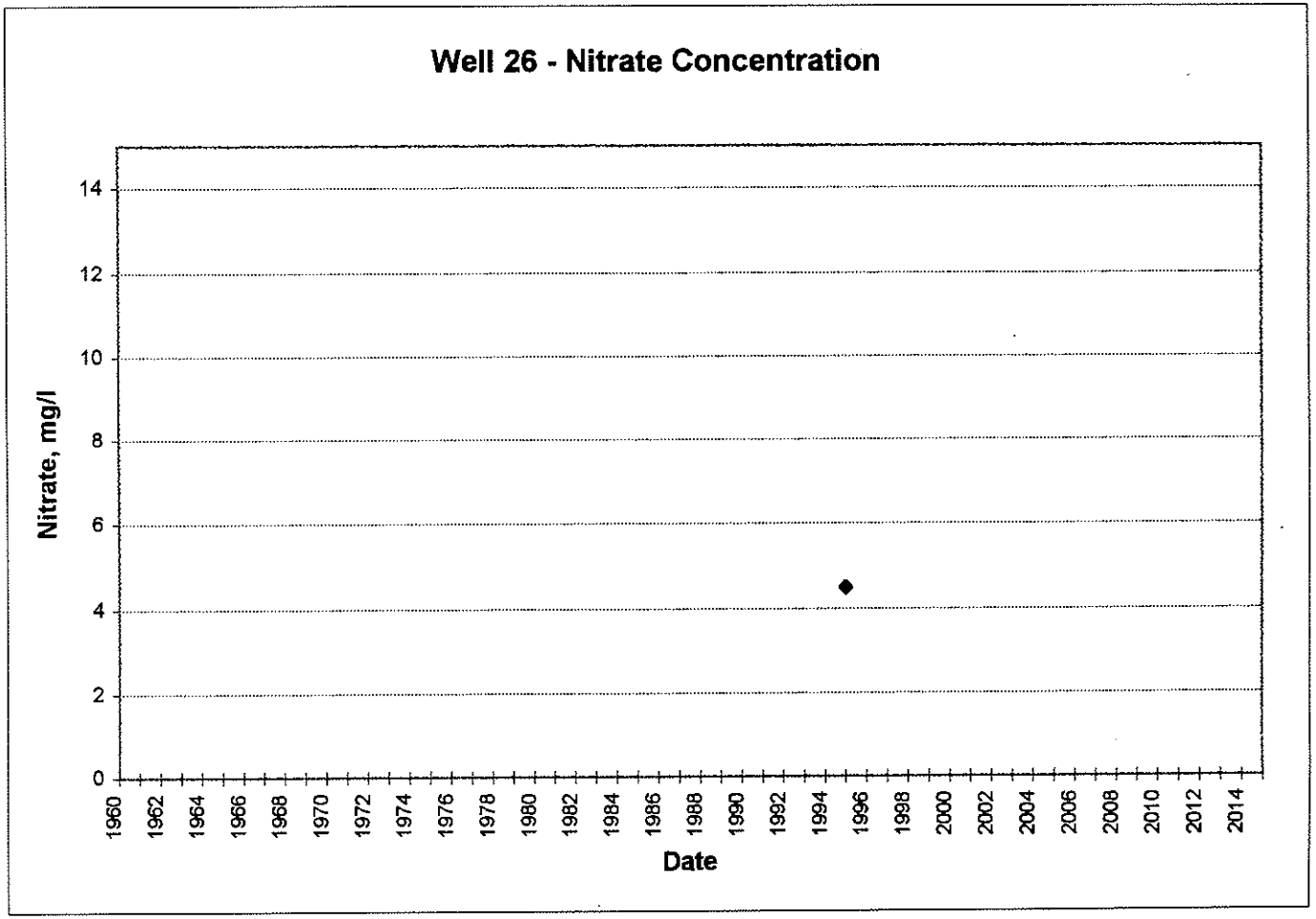
Well 25 - Nitrate Concentration

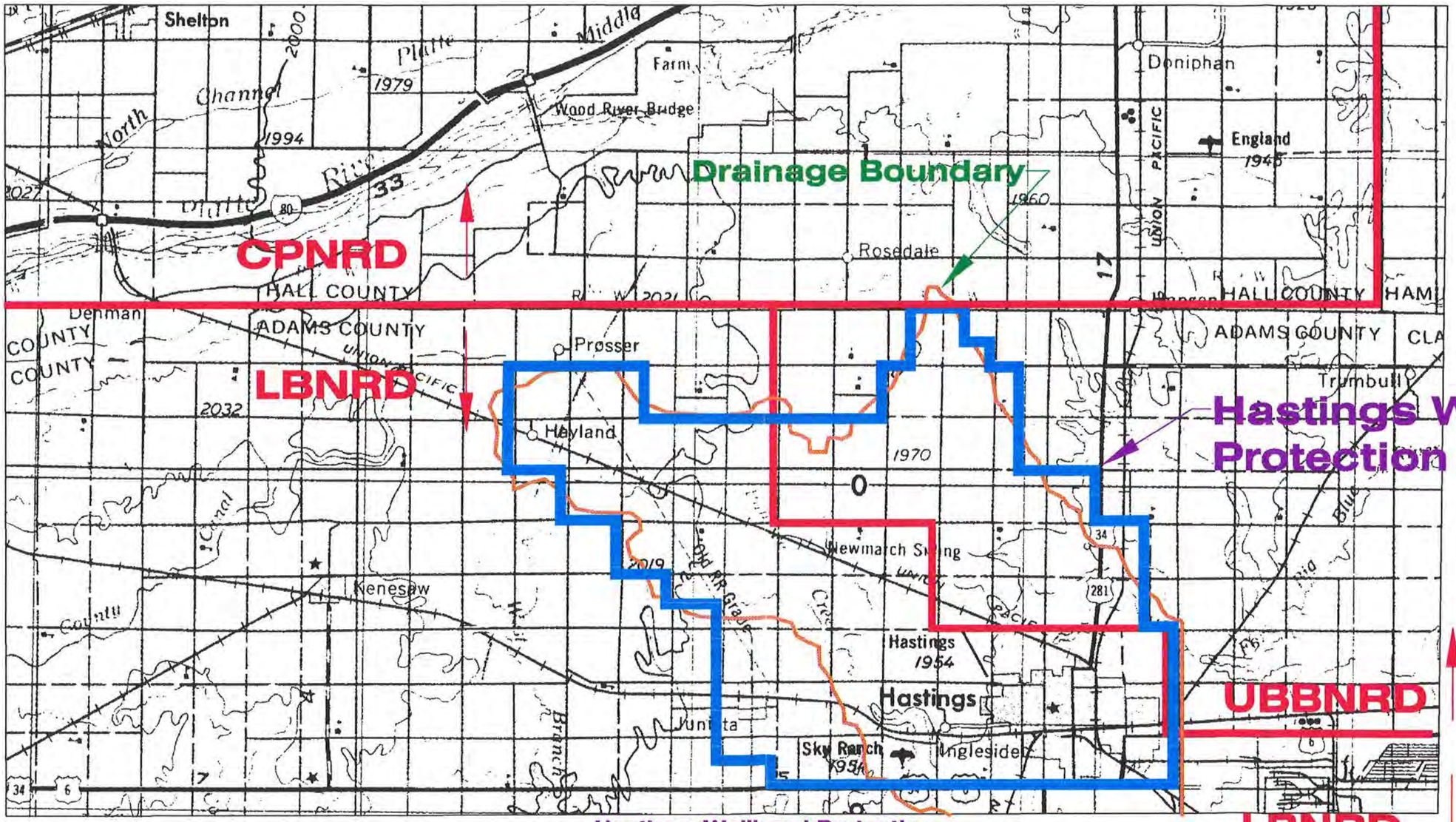
6/27/95	2.5



Well 26 - Nitrate Concentration

5/31/95	2.5





Hastings Wellhead Protection Area Level 2

Note:
Total WHPA 68.25 sq. mi.
WHPA within LBNRD 46.0 sq. mi.
WHPA within UBBNRD 22.25 sq. mi.

HASTINGS UTILITIES
HASTINGS, NEBRASKA

Hastings Wellhead Protection Area
c:\wellhead

DSN: M. Stange	SUP'T	CKD:	No. 1 of 1
DRN: T. Waite	W.O. No.	CKD:	
DATE: 1-21-98	RET. No.	APP:	