CITY OF WESTFIELD, INDIANA

DRINKING WATER MASTER PLAN

FEBRUARY 2010

HNTB

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FEBRUARY 2010

ARCHITECTS ENGINEERS PLANNERS
111 MONUMENT CIRCLE, SUITE 1200
INDIANAPOLIS, INDIANA 46204
(317) 636-4682

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CHAPTER 1
INTRODUCTION

1.1 BACKGROUND

The City of Westfield is a quickly growing suburban community within Washington Township in Hamilton County, Indiana. As the number of residents and businesses in this community continues to grow, the importance of having a master plan in place for utilities in the area grows as well. Previous master planning efforts have been completed in 2001 and 2003 to help Westfield's distribution system grow effectively with their population. This report has been created to document the analysis and evaluations performed in the master planning process. The Drinking Water Master Plan evaluates needed improvements to the City of Westfield's raw water production, water treatment facilities, and finished water distribution system which would allow for potential full buildout of the study area shown in **Figure 1-1**. The timeframe and the possibility of realizing full development of this study area is difficult to predict since it depends on various demographic and economic factors both locally and regionally. Therefore, instead of a chronological approach, a geographical approach has been taken to evaluate development of this system.

1.2 PURPOSE AND SCOPE

The purpose of this master plan is to identify raw water sources, treatment capacity, water main routing, pumping capacity, and storage requirements that would be needed to fully buildout the study area. Since much of the area is undeveloped, the planning analysis included determining water main routes and finished water storage locations, and determining water main and storage tank sizes. The sizing of future infrastructure was determined by utilizing the Equivalent Dwelling Unit (EDU) assignments developed in the Wastewater Master Plan completed in September of 2006. These EDU assignments were developed based on existing land use of developed areas and projected land use in undeveloped areas. The Westfield-Washington Township Comprehensive Plan was used to assign the projected land use for undeveloped areas. These assignments were then used to determine how much capacity would be required if the study area is to fully develop.

A portion of the planning area for the City's drinking water system lies outside of Washington Township, and therefore outside of the scope of the Westfield-Washington Township Comprehensive Plan. This area, which resides within Noblesville Township in Hamilton County, is covered in the City of Noblesville's Comprehensive Plan. This document was referenced to determine the proposed land use for the distribution system area outside of Washington Township. The number of EDUs per acre was assigned in the same manner as for areas in Washington Township and Noblesville Township. The methodology used in assigning EDUs is described in **Chapter 2**.

The scope of this project includes the following:

1. Delineate specific planning areas for drinking water distribution system.

2. Utilize projected land use to predict probable future drinking water demands for each of the three pressure zones.

3. Develop drinking water treatment, pumping and storage needs for the planning area.

    a. Compare current flow conditions and capacities of the system to future needs based on probable future flows.
    b. Identify distribution system infrastructure to meet future needs of the planning area.

4. Create a distribution system service map showing future master plan water mains for the planning area.

1.3 STUDY AREA

The study area for this master plan is all of Washington Township and a small portion of Noblesville Township that includes the River Road and Cherry Tree Water Treatment Plant sites. This portion of Noblesville Township is already a part of the Westfield Drinking Water Utility's Service Area. The study area for the Drinking Water Master Plan is approximately 2,900 acres larger than the study area for the Wastewater Master Plan due to the inclusion of this portion of Noblesville Township. The total study area is approximately 62 square miles or 39,530 acres and is identical to the Public Works Department Service Area. **Figure 1-1** shows the study area boundaries.

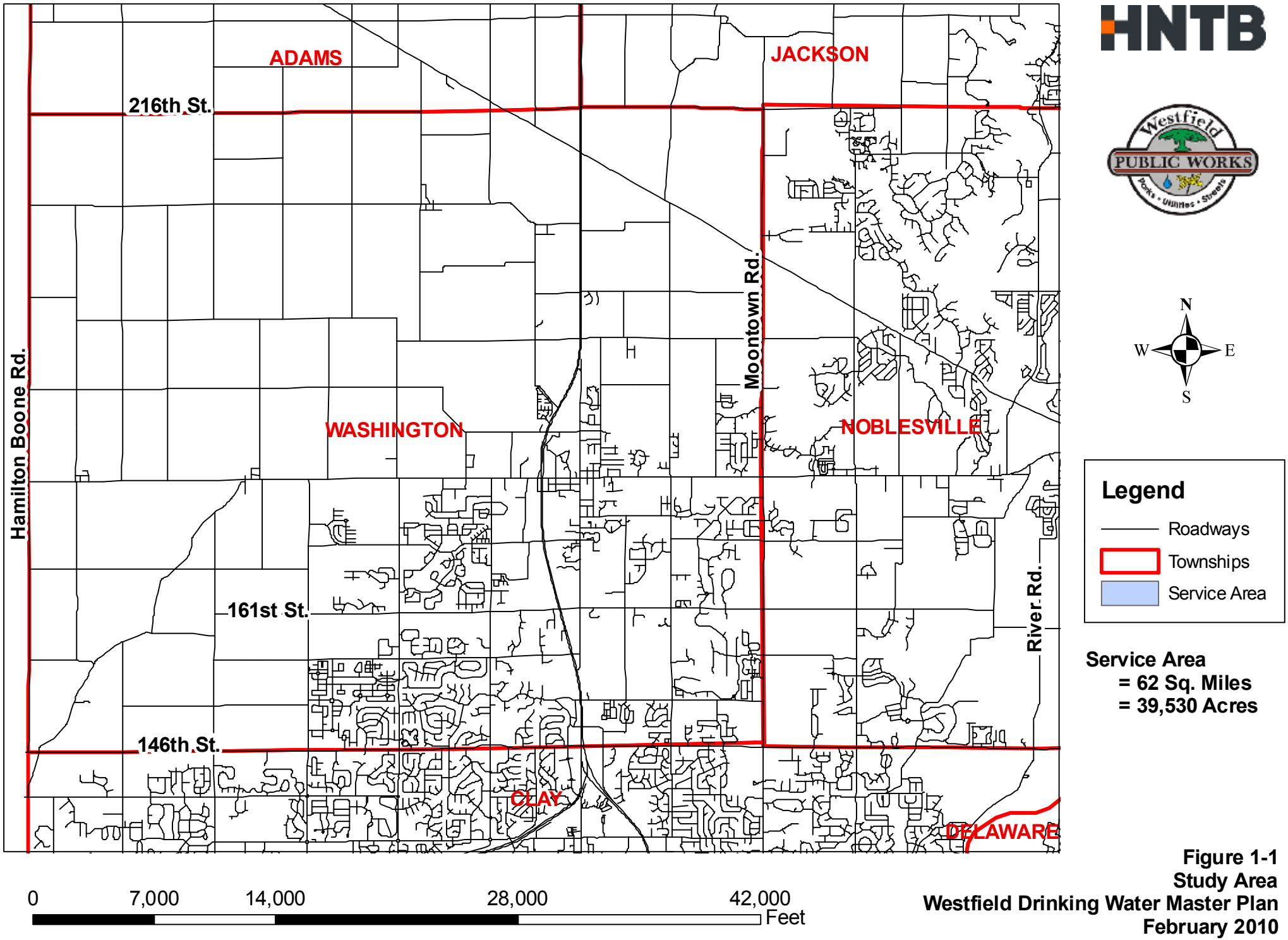


Figure 1-1
Study Area
Westfield Drinking Water Master Plan
February 2010

CHAPTER 2
METHODOLOGY

2.1 WATERCAD MODEL

The City of Westfield has an existing hydraulic model which is currently maintained by HNTB. A hydraulic model allows the user to simulate the functioning of the distribution network it models and to study the effect of changes on the network's operation through changes in user input. Modeling software such as WaterCAD utilizes the laws of conservation of mass and energy to determine pressure and flow distribution throughout the system. The City of Westfield's hydraulic model was created and has been maintained in the WaterCAD software platform.

The City's distribution network model is periodically updated and evaluated using data from AutoCAD and GIS files which are kept up to date by Westfield staff as water mains and other drinking water infrastructure are installed, replaced, or taken out of service. The model also includes the locations of proposed future mains and elevated tanks which have been planned by master-planning efforts. The locations and sizes of these proposed mains and tanks are evaluated and updated as necessary.

2.1.1 Calibration

In order to verify that the system model scenario analyses will provide results that will simulate actual conditions, the model must be calibrated using current field data. The City of Westfield's Supervisory Control and Data Acquisition (SCADA) system was used to collect field data for this purpose. Data including the system's tank levels, pump operational conditions, and pressure readings throughout the system were used to calibrate the hydraulic model by comparing measured data to simulated results and making any necessary corrections or modifications to the model input parameters until the model results matched observed field data. Typically, input parameters such as pipe roughness and specific node demand are adjusted in the calibration process.

As a first step, head-loss tests were performed on representative mains throughout the system to verify the accuracy of the C-factors used in the model for various pipe materials and ages. The two-hydrant method was used to measure C-factor. In the two-hydrant method, a pipe section is isolated by closing downstream valves and placing pressure gauges on an upstream and a downstream fire hydrant. The pressure at each of the two hydrants is first recorded in the static condition and then a measured flow is released from the downstream end and the change in pressure is recorded at both hydrants. The Hazen-Williams equation can then be used to calculate the C-factor for that specific pipe section and then the C-factors in the hydraulic model are corrected.

Once the pipe C-factors had been determined, the second step in the calibration process involved setting the pipe C-factors to those values calculated from the head-loss tests. These values are used not only for the exact mains upon which the tests were performed, but also other mains in the system of similar age and composition within the same pressure zone. As a part of this step,

the operational conditions of the model system were set to match the operational conditions of the calibration data set (i.e., tank levels and plant and booster station flows).

The third step in calibrating the hydraulic model involved comparing measured pressures in the distribution system to the pressures at the same locations in the simulated system (the hydraulic model). To make this comparison, a steady-state scenario was analyzed with all of the operational conditions in the simulated system set to match the operational conditions of the pumps, tanks, and valves in the actual system for the period during which the calibration data was taken. When the scenario analysis results are evaluated, adjustments are made to the C-factors and node demands in the areas in which the simulated pressures differ from the measured pressures. Usually, there are several iterations of changes until good agreement is reached between the measured and simulated results.

The calibration of the Westfield distribution system was performed using data from the SCADA system including tank levels, plant and booster station flow rates, and pressure measurements. The calibration data used was collected at one specific time during an average weekday to best exemplify the standard behavior of the system.

After the final scenario analysis, the simulated pressures are compared to the measured pressures from the calibration data set to determine the fitness of the simulated to measured results and hence the accuracy of the calibration. When the Westfield distribution system model was calibrated, there were nine locations at which the simulated and measured pressures were compared. Of these nine locations, all but one was found to be accurate to within six percent of the measured values. The one higher value resulted from a pressure taken at the 161st Booster Station where it pumps into the North Pressure Zone. This node's simulated pressure matched the measured pressure with 11.3-percent accuracy.

2.1.2 Scenarios

Several scenarios have been designed to assist in the analysis of existing and future system components. The scenarios described below will be used to evaluate the correct sizing and proper operation of existing system components and to evaluate the needed sizes for future system components. Each scenario that has been developed for Westfield's distribution system is designed for a specific purpose. A total of five scenarios have been developed.

The first scenario is based on the current condition of the system including the current average demand of the system, the physical condition of the system components, and the operations currently in effect. This scenario includes only the infrastructure currently installed. This scenario was developed as a base scenario from which the other scenarios are developed. The base scenario, called the 2008 Average Day Scenario, is used when calibrating the model.

Second, the 2008 Maximum Day Scenario was developed using a global multiplying factor to increase the demand across the system from the average day to the maximum day demand. This scenario is developed in order to evaluate the functioning of the distribution system in its current condition during a maximum-demand day and to determine if there are any signs of system stress which might denote undersized or deteriorated components within the distribution system.

System stress is demonstrated by the following: 1) pressure less than 35 psi; 2) pipe velocity greater than 10 ft/sec; and 3) head loss greater than 10 ft/1,000 ft.

A third scenario was developed in order to run a fire-flow analysis to determine the current availability of fire flow to each node in the system. This scenario is identical to the 2008 Maximum Day Scenario except that when the calculation is run, the model calculates the available fire flow at each node while maintaining residual pressure of 20 psi at the node being tested and in the rest of the system. The 20 psi lower limit on system pressure is a requirement of the *Ten States Recommended Standards for Water Works* suggested by IDEM as a basis for design of water supply systems in Indiana. The fire-flow analysis was performed using the 2008 Maximum Day Scenario because the occurrence of a fire during a high-demand day is considered to be the worst-case scenario for the system.

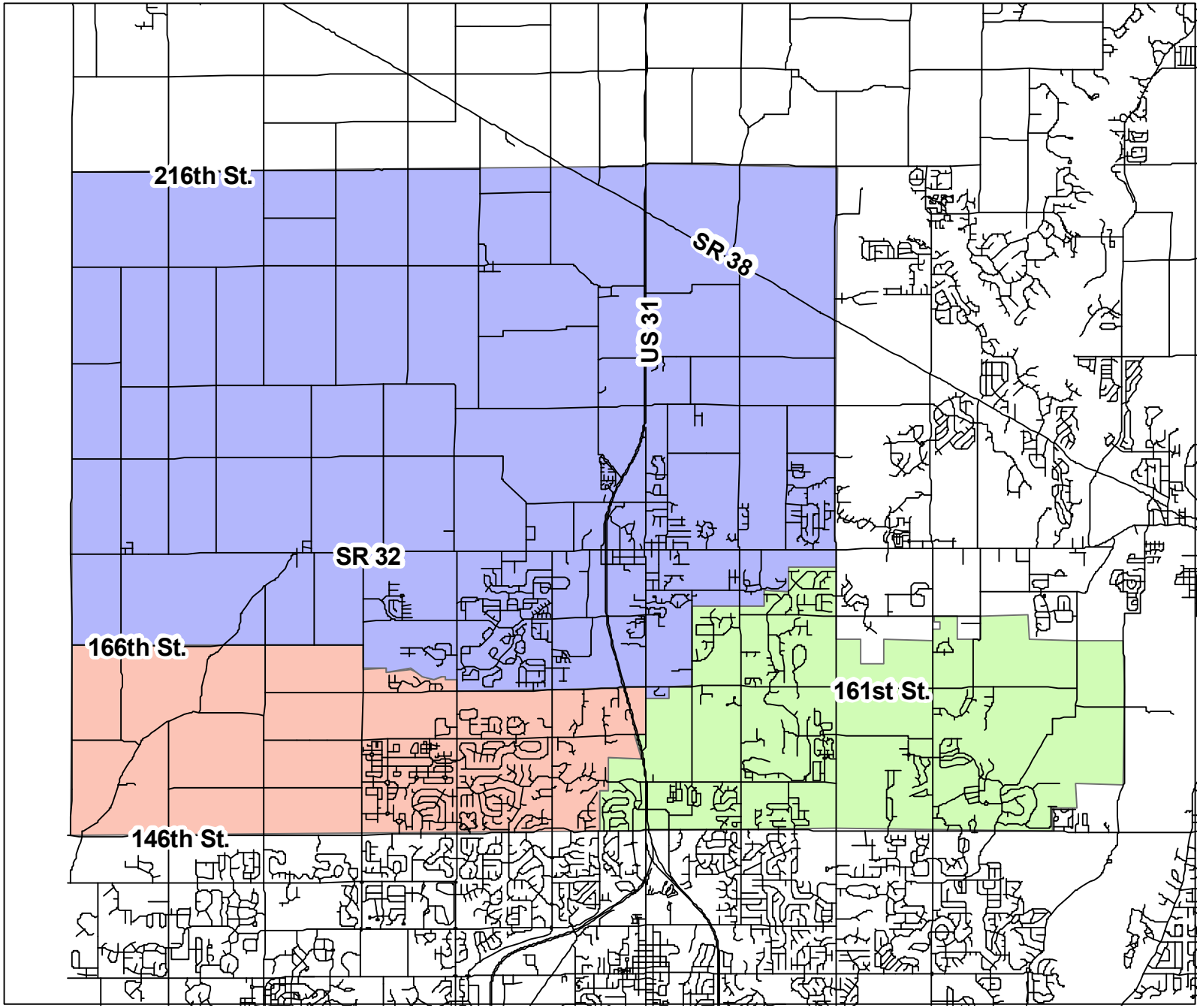
For the full buildout of the system two additional scenarios were developed. The physical conditions of these scenarios were developed using the future piping layouts from past master-planning efforts which followed major roadways throughout Washington Township. The system demand was then developed as described in **Chapter 3** for the fourth scenario, called the Ultimate Buildout Average Day Scenario, which has a projected demand of 22.14 million gallons per day (MGD). The fifth scenario, the Ultimate Buildout Maximum Day Scenario was developed by using a global multiplying factor (equivalent to the system-peaking factor discussed in **Chapter 3**) to increase the demand across the system to the maximum-day demand of 33.23 MGD for the full buildout condition. These two final scenarios were used to develop and examine the future proposed components of the system at the buildout condition.

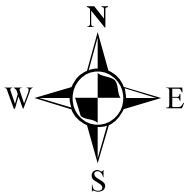
2.2 ASSUMPTIONS

This section summarizes the assumptions that were made for the master plan.

<u>Pressure Zones</u>

- Current pressure zone divisions are due mostly to the combination of the Hamilton Western Utilities distribution system and the Westfield distribution system. The Hamilton Western Utilities system, acquired in 2002, was bounded by 161st Street and 146th Street to the north and south, respectively, and consisted of two pressure zones, now the Southwest and Southeast pressure zones. The Westfield system was bounded to the south by 161st Street and consisted mostly of what is now considered the Northern pressure zone. Existing pressure-zone boundaries were extrapolated outward, toward the Washington Township boundaries, to create the ultimate buildout pressure-zone delineations shown in **Figure 2-1**. The Northern pressure zone is delineated by the extents of Washington Township to the north, west and east, and approximately by 161st Street to the south. The Southeast and Southwest pressure zones are delineated by approximately 161st Street and 166th Street to the north and by the Washington Township boundary to the south.





Legend

Roadways

Pressure Zones

- Northern
- Southeast
- Southwest

0 4,000 8,000 16,000 24,000 Feet

Figure 2-1
Pressure Zones
Westfield Drinking Water Master Plan
February 2010

Equivalent Dwelling Units (EDUs)

- The EDUs assigned to each area are net EDUs (meaning that roads and green space are not included). Gross EDUs (including roads and green space) were utilized for the main EDU assumption. For instance, if the undeveloped land was to be single-family homes, it was assigned a gross EDU of 3.0 EDUs per acre. However, the actual EDU assignment for that land was calculated based upon 3.0 gross EDUs per acre multiplied by 85 percent (assume that 15 percent will be used for roads), which computes to 2.6 net EDUs per acre.

- All land was considered to be developable. Therefore, the factor of 20-percent undevelopable that has been utilized in previous master plans was not utilized.

- A development class was determined for each area. The development classes included: D=developed; A=assumed development (future part of a subdivision, for instance); ES=existing served; EN=existing non-served; U=undeveloped.

- For development class D, the following assumptions were made:
  - The parcels were counted to determine the number of houses within the development. This parcel count is equal to the EDUs assigned to the development.

- For development class A, the following assumptions were made:
  - If a Planned Urban Development (PUD) exists for the planned development, then the PUD was utilized to determine the number of EDUs to assign to the development.
  - If the subdivision is partially developed, then the part that is parceled was assigned development class D, and the un-parceled portion was assigned development class A. The PUD was utilized to determine the number of EDUs remaining for the un-parceled portion.

- For development class ES or EN, the following assumptions were made:
  - If the ES/EN parcel was a residential parcel, then the EDUs were determined by the land use (one EDU if a single family home, 7.0 gross EDUs per acre if apartments).
  - If the ES/EN parcel was a commercial parcel, then the EDUs were assumed to be 3.0 gross EDUs per acre.
  - If the ES/EN parcel was a church or a school, then the EDUs were based upon the water usage, 10-month average. The water usage data were provided by the Westfield Public Works Department.
  - If the ES/EN parcel was thought to be currently served, then it was assigned development class ES. If it was thought to not be served currently, then it was assigned development class EN.

- For development class U, the following assumptions were made.
  - If the parcel was in a projected residential area, then the following residential land uses were utilized. The boundaries listed here are general guidelines.
    - East of Towne Road/Centennial Road – 3.0 gross EDUs per acre

    - West of Towne Road/Centennial Road and south of 186th Street – 2.5 gross EDUs per acre
    - West of Towne Road/Centennial Road and north of 186th Street – 1.5 gross EDUs per acre. This lower land use was utilized to reflect the fact that this section of the study area is remote from the City center and will likely keep its rural nature.
- If the parcel was in the commercial area, then the land use was assumed to be 3.0 gross EDUs per acre.
- If the parcel was in the employment area, then the land use was assumed to be 1.5 gross EDUs per acre.

CHAPTER 3
DEMAND ANALYSIS

3.1 POPULATION

One of the ways to evaluate whether or not the expectation of future increase in finished water demand is practical is by looking at population trends in the area. The community of Westfield has experienced steady population growth since approximately the 1940s as illustrated in the Census data shown in **Figure 3-1**. This growth is comparable to both Washington Township and Hamilton County in which it resides (see **Figures 3-2 and 3-3**). The most marked growth has occurred most recently as shown by the 230-percent increase in population between the 2000 Census and the most recent population estimate in 2008, which estimated 21,171 residents in Westfield, Indiana. More recent growth has been due in part to the annexation of southern portions of Washington Township as well as rapid population growth in Westfield's established neighborhoods. On January 1, 2008 the Town of Westfield was incorporated and is now the City of Westfield, Indiana. All decennial Census data reference in this report has been procured from the U.S. Census bureau web site: http://www.census.gov.

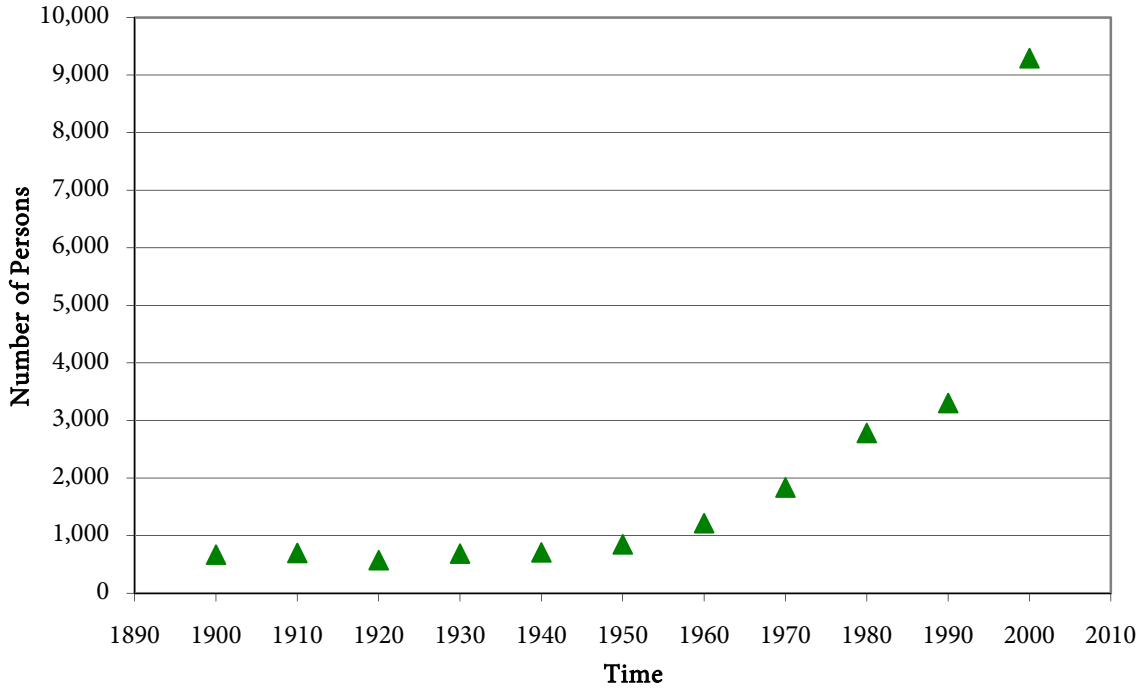


Figure 3-1
HISTORICAL POPULATION OF WESTFIELD IN HAMILTON COUNTY, INDIANA

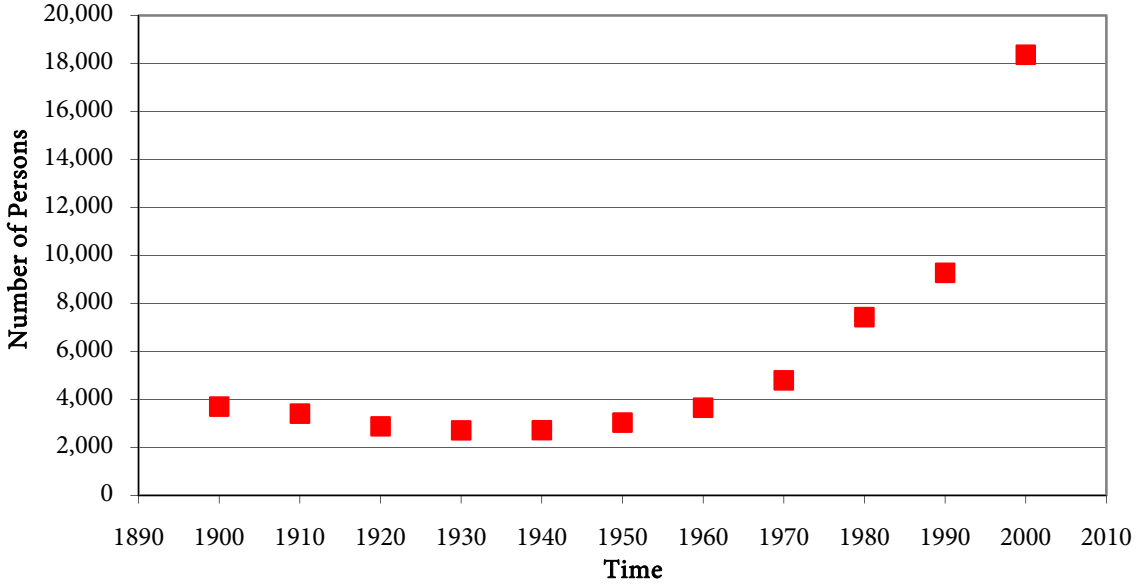


Figure 3-2
HISTORICAL POPULATION OF WASHINGTON TOWNSHIP IN HAMILTON COUNTY, INDIANA

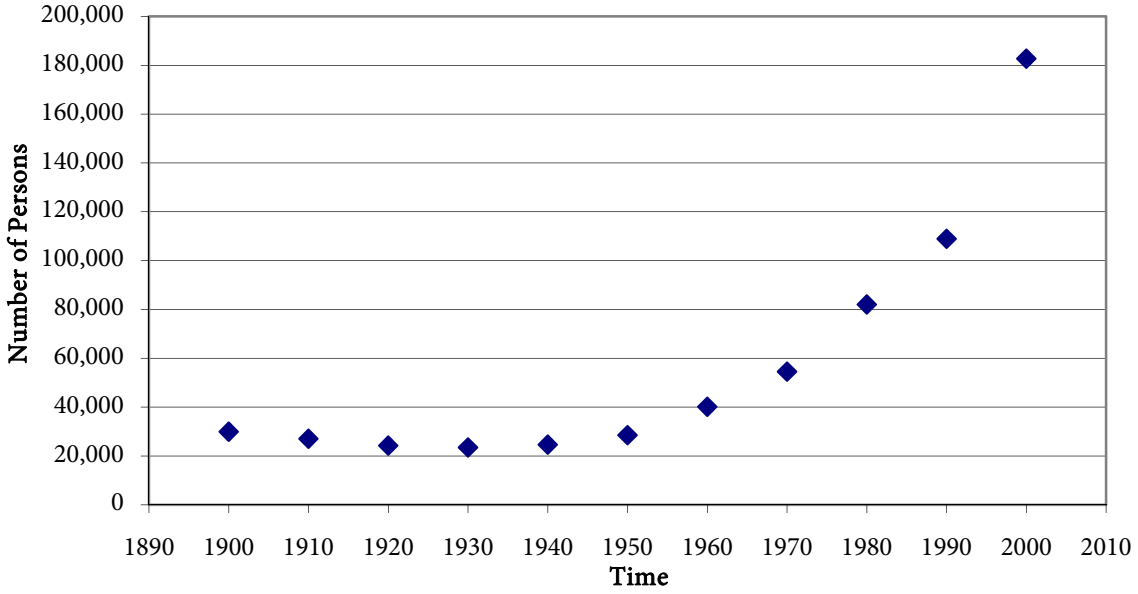


Figure 3-3
HISTORICAL POPULATION OF HAMILTON COUNTY, INDIANA

Decennial Census data, as well as more recent yearly estimates of the population of Westfield, Washington Township, and Hamilton County, Indiana have been included in **Table 3.1** to illustrate the continued growth occurring in the City of Westfield and the surrounding area since the last Census in 2000. As shown in this table, the percentage of Washington Township residents living in Westfield has increased steadily, from approximately 26 percent in 1940, to 51 percent in 2000, and an estimated 64 percent according to the most recent population estimate in 2008.

Table 3.1
SUMMARY OF HISTORICAL POPULATION DATA

Year	*Hamilton County*	*Washington Township*	*Westfield*
1900	29,914	3,696	670
1910	27,026	3,402	700
1920	24,222	2,874	574
1930	23,444	2,706	688
1940	24,614	2,714	709
1950	28,491	3,032	849
1960	40,132	3,651	1,217
1970	54,532	4,789	1,837
1980	82,027	7,425	2,783
1990	108,936	9,272	3,304
2000	182,740	18,358	9,293
2001*	196,259	20,217	15,793
2002*	207,733	22,237	16,477
2003*	219,631	24,206	17,107
2004*	231,568	26,396	17,886
2005*	241,796	28,330	18,564
2006*	251,611	29,805	19,134
2007*	260,804	31,079	19,980
2008*	269,785	32,948	21,171

**Yearly estimates have been obtained from STATS Indiana, the information utility for the State of Indiana which is maintained by the Indiana Business Research Center at Indiana University's Kelley School of Business.*

3.2 DEMAND

Data from the City of Westfield's SCADA system was used to determine the current average and maximum day demands. Daily flow data from January 2008 through December 2008 was analyzed to determine that the average daily demand was approximately 3.1 million gallons per day (MGD) and the maximum daily demand reached up to 5.2 MGD. Based on these numbers, the current Maximum Day to Average Day ratio is approximately 1.7:1, which falls well within the range of 1.5:1 to 2.8:1 common for most distribution systems according to the *Water Distribution Systems Handbook* (McGraw-Hill, New York 2000). The ratio used for planning in the previous master-planning efforts was 1.5:1, which will continue to be used for planning purposes. As the number of customers increase, the ratio of Maximum Day to Average Day demand typically decreases.

The goal of planning for the City of Westfield's finished water distribution infrastructure at full buildout required the hydraulic model to be loaded with the total demand projected for full development of the study area. Demand for buildout was assigned throughout the system model based on existing land use and, in areas that are currently undeveloped or zoned agricultural, projected land use. In areas that have already been developed, a number of equivalent dwelling units (EDUs) was assigned based on the current land use. In residential areas, the houses were counted and each was assigned one EDU. For a commercial development or employment area the area was assumed to develop at rates that were determined in the previous master-planning efforts: 3.0 EDUs per acre for commercial and 1.5 EDUs per acre for employment area. The development rates were determined based on the fact that a commercial development would likely be a large water user, while the employment area would be comprised of light industrial development, which would not be a large water user.

For undeveloped and farmland areas, the potential land use is considered. Since most of the study area is rural in nature, the existing land use is less than 1.0 EDU per acre. However, development trends in the area have shown that when farmland is developed into residential housing developments, buildout is approximately 3.0 gross EDUs per acre. A gross EDU is defined as the total number of houses divided by the total land area for the entire development. It is normally assumed that 85 percent of the land is actually developed, with the other 15 percent being roads, green space, and other areas that do not contain houses. Therefore, 3.0 gross EDUs per acre equates to 2.6 net EDUs per acre ($3.0 \times 0.85 = 2.6$). Since some of the township is fairly remote from Westfield's historical downtown area (the area where population is most dense within the City), varying residential land uses were utilized for different portions of the study area.

To further assist in the master-planning process, development classes were established and assigned to each area. The assumptions made for each development class are outlined in **Chapter 2**. Any development that has been proposed is considered as already developed. The development classes are as follows:

- D = developed, as in an existing subdivision
- A = assumed development, as in a proposed subdivision

- ES = existing served, as in an existing house/business that is connected
- EN = existing non-served, as in an existing house/business that is not connected
- U = undeveloped

To determine the size of future water mains, the fully developed condition for each area was evaluated. The fully developed condition is the condition in which all of the currently undeveloped (or agricultural) area is developed at the land uses shown in **Figure 3-4**. For each EDU, it is assumed that 310 gallons per day (gpd) is the average demand based on an average of 3.1 persons per household and 100 gpd per capita water usage.

Full development of the study area shown in **Figure 3-4** would increase the total number of EDUs served to approximately 71,425. This equates to an increased average day demand of approximately 22.2 MGD for the design condition based on 310 gpd per EDU. Given a Maximum Day to Average Day ratio of 1.5:1, the projected maximum daily demand for full buildout of the study area is 33.3 MGD. **Table 3.2** shows a breakdown of system demand into each pressure zone.

Table 3.2
BREAKDOWN OF PROJECTED SYSTEM DEMAND

Zone	*EDUs*	*Average Day (MGD)*	*Maximum Day (MGD)*
Southeast	4,603	1.43	2.15
Southwest	12,578	3.90	5.85
North	54,246	16.81	25.23
TOTAL	71,425	22.14	33.23

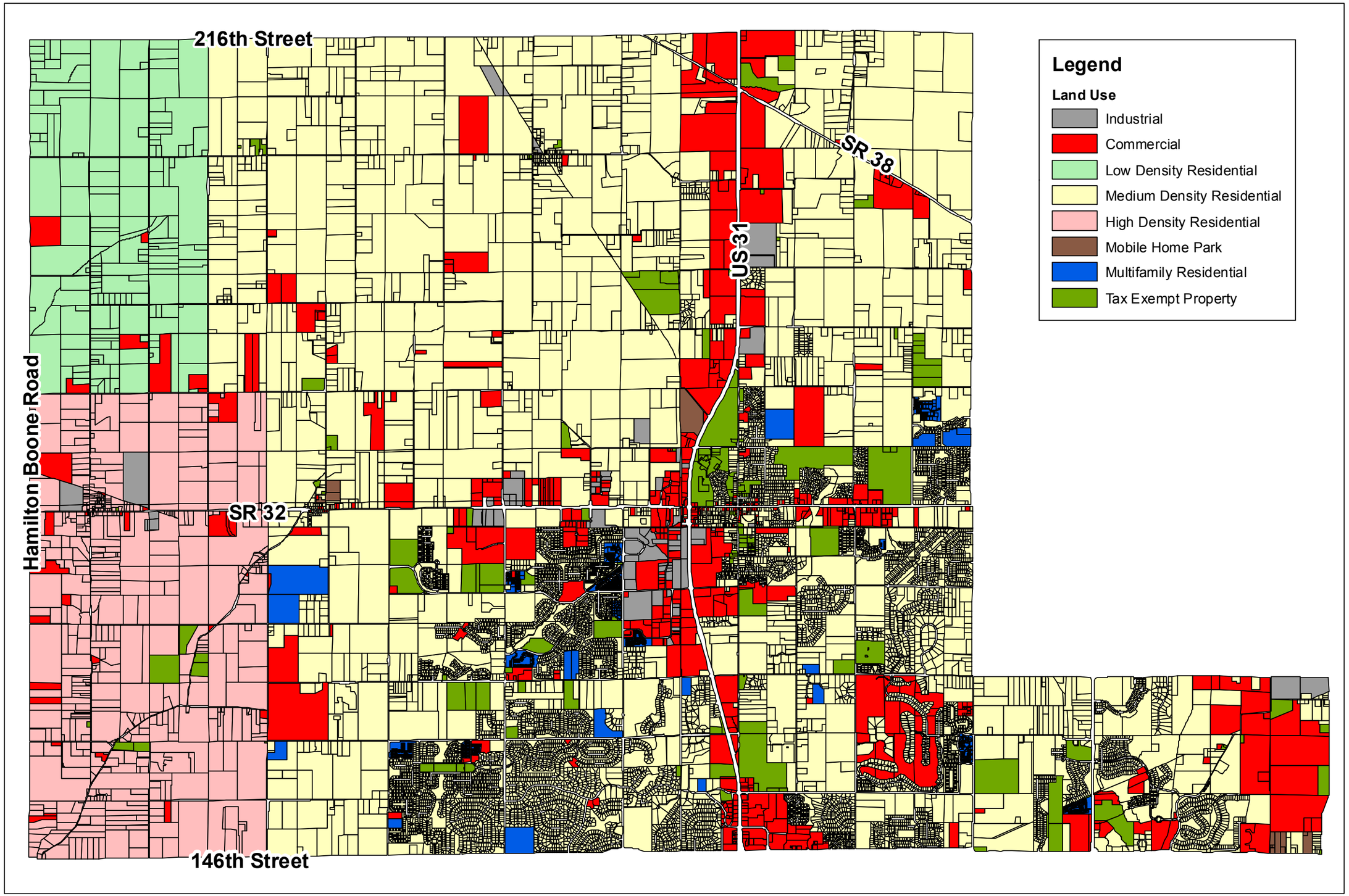


Figure 3-4
Projected Land Use Map
Westfield Drinking Water Master Plan
February 2010

CHAPTER 4
SUPPLY ANALYSIS

4.1 EVALUATION OF SUPPLY

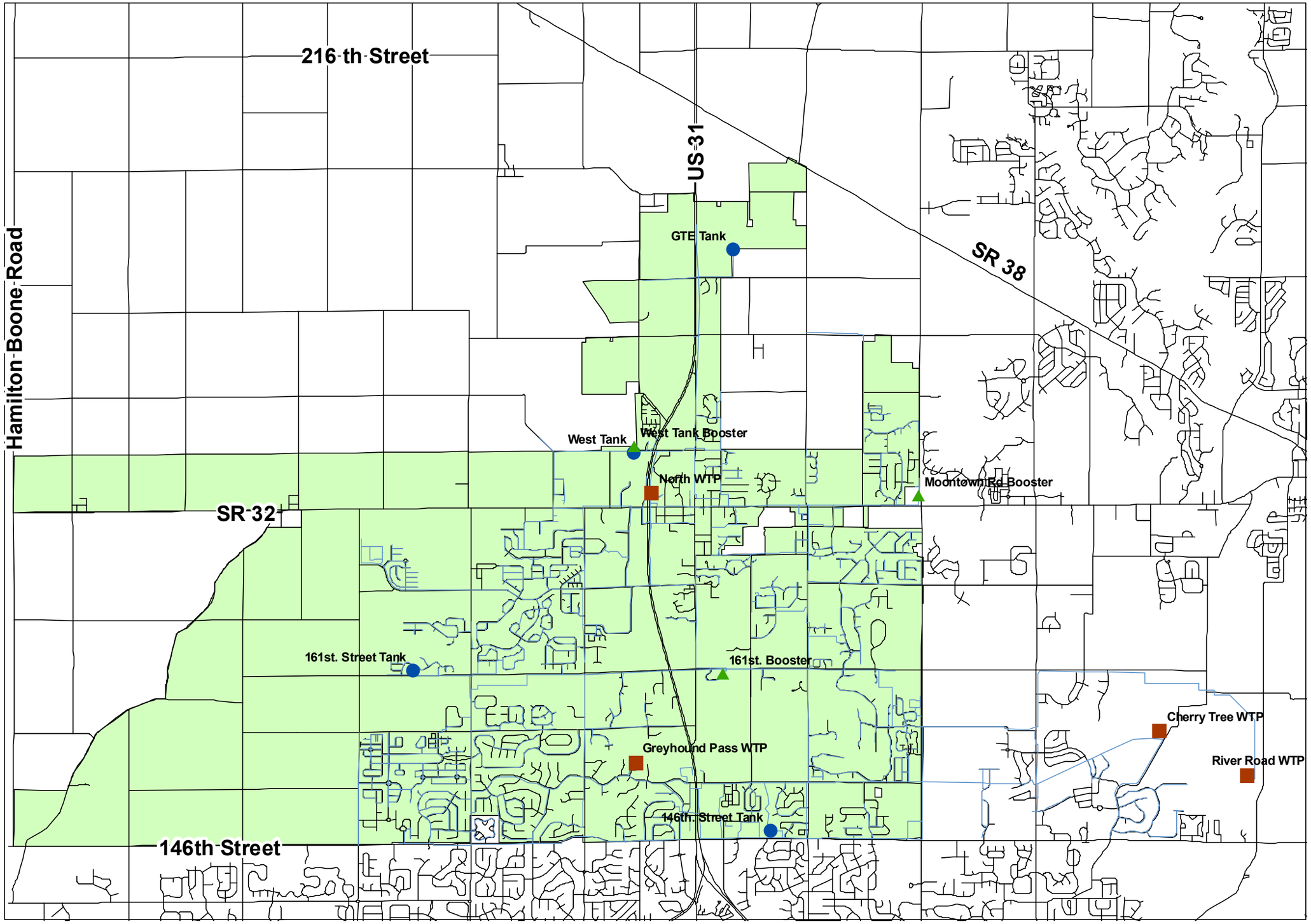
Raw water supply to the City of Westfield is currently solely supported by groundwater. It is beneficial to the City to continue to take advantage of the groundwater in the area for raw water supply due to the additional facilities, and associated costs, which would be required to treat surface water for potable water use. In order to evaluate the available groundwater in the area to meet the future demands of the Westfield System, Eagon & Associates was contracted to prepare a well field capacity analysis for the River Road area well fields. This analysis has been included as **Appendix A** of this report.

4.1.1 Sources of Supply

Currently, the City of Westfield draws water from two unconsolidated aquifers: the Tipton Till Plain Aquifer System and the White River and Tributaries Outwash Aquifer System (Eagon & Associates, 2009). Within these two aquifers there are 11 wells from which the City currently pumps raw water for drinking water supply and wholesale. One well is located at the North Treatment Plant site, one is located at the Greyhound Pass Treatment Plant, three are located at the Cherry Tree Well Field at Cherry Tree Avenue and East 160th Street, and six are located along River Road, north of 146th Street, at the River Road and Welcome Well Fields. The raw water mains serving the wells at the Cherry Tree, River Road and Welcome locations are interconnected to provide these facilities with the ability to delegate additional raw water flow to either the Cherry Tree or River Road treatment facilities, or to the Indianapolis Water connection at River Road.

Table 4.1 provides a listing of the existing and planned wells within Westfield's system, and their respective capacities. The North and Greyhound Pass well fields are located on the same site as their respective treatment facilities as shown in **Figure 4-1**. The Cherry Tree, River Road, Welcome, and Horseshoe (DeWitt) well fields are located in the vicinity of the River Road Treatment Plant as shown on **Figure 1** in **Appendix A**. The combined production capacity of Westfield's wells is a maximum of approximately 18 million gallons per day (MGD). Five MGD of that raw water capacity is contracted to Indianapolis Water to supplement their raw water supply until 2017. Indianapolis has a 24-inch main running along River Road to which the contracted water is pumped via a combination of vertical turbine pumps at Wells 5, 7, 8, 11, 12 and 13.

Plans are currently underway to add another 5 to 6 MGD of raw water production to the existing Westfield system. A new well field is planned for the Horseshoe Property along 160th Street near the west bank of the White River. This site contains several observation wells and one test / production well (Well No. 14). Both a stepped-rate test and a 48-hour pumping test were performed at Well No. 14 in 2009. The Well Field Capacity Estimate prepared by Eagon & Associates is included in this report in **Appendix B**.



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Legend

- Westfield Corporate Limits
- Booster Stations
- Treatment Facilities
- Elevated Tanks
- Water Mains 8 in. & Larger
- Roadways

Figure 4-1
Existing Drinking Water Infrastructure
Westfield Drinking Water Master Plan
February 2010

Table 4.1
SUMMARY OF RAW WATER PRODUCTION

Well Field	*Well ID*	*Sustainable Flow (gpm)*	*Max Flow (gpm)*
North	North Well	100 gpm	100 gpm
Greyhound Pass	Well No. 4	300 gpm	300 gpm
Cherry Tree	Well No. 5	1,600 gpm	1,800 gpm
	Well No. 6	1,320 gpm	1,475 gpm
	Well No. 7	850 gpm	970 gpm
River Road	Well No. 8	2,000 gpm	2,055 gpm
	Well No. 9	2,100 gpm	2,150 gpm
	Well No. 10	2,000 gpm	2,000 gpm
Welcome	Well No. 11	1,400 gpm	1,400 gpm
	Well No. 12	700 gpm	700 gpm
	Well No. 13	700 gpm	700 gpm
Horseshoe	Well No. 14*	TBD	1,400 gpm
	Well No. 15*	TBD	1,400 gpm
	Well No. 16*	TBD	1,400 gpm

**Wells at the Horseshoe well field (also referred to as the DeWitt Property) have not yet been installed as of the date of this report.*

4.1.2 Source Planning for Full Development of the Study Area

The addition of the Horseshoe property wells will add another 5 to 6 MGD to the City's raw water supply, increasing the total raw water supply to approximately 24 MGD. This will boost raw water supply above the 22.14 MGD projected for the full buildout average day demand of the study area shown in **Chapter 1**. However, an additional 9 MGD of raw water would be needed to supply the 33.23 MGD projected for the maximum day demand.

The aquifer which supplies the wells at the Cherry Tree, River Road, Welcome, and Horseshoe well fields is the White River and Tributaries Outwash Aquifer according to the capacity analysis performed by Eagon & Associates. The analysis indicates that this aquifer is capable of providing the projected maximum day demand under high recharge conditions. The installation of additional wells within this aquifer or the acquisition of existing nearby wells will be required in order to access the available capacity. In addition, one new well on the River Road site, east of the existing Well No. 9, has been proposed and a test boring has been completed to verify that the correction formation exists at this location.

The Tipton Till Plain Aquifer System, which supplies the North and Greyhound Pass well fields, has a recharge rate that will most likely not exceed two inches per year and is not likely to provide a reliable source of additional raw water.

4.2 EVALUATION OF TREATMENT CAPACITY

Prior to the acquisition of Hamilton Western Utilities (HWU) and its inclusion into the Westfield distribution system, the Westfield system had two water treatment facilities each with its own well or wells: the North Plant and the South Plant. The North Plant is located directly east of US 31 and North of State Road 32, on the site of the Westfield-Washington Schools (see **Figure 4-1**). The South Plant is no longer in use, but was formerly located on the site of the Westfield Utilities Office on East 171st Street.

As a part of the HWU acquisition, three existing treatment plants were added to Westfield's system: Greyhound Pass, Cherry Tree, and River Road. These plants' locations are shown on **Figure 4-1**. All three of these plants are still in operation and have received some upgrades since being brought into the Westfield system. Further discussion of the treatment plants currently in operation has been provided in subsequent sections. A summary of the currently available treatment capacity in Westfield's system is given in **Table 4.2**.

Table 4.2
SUMMARY OF EXISTING TREATMENT

Facility ID	*Treatment Capacity*
River Road WTP	7.5 MGD
Cherry Tree WTP	3.5 MGD
Greyhound Pass WTP	0.43 MGD
North Plant	0.14 MGD
TOTAL	**11.57 MGD**

4.2.1 River Road Water Treatment Plant

The River Road Water Treatment Plant (WTP) is located in the Southeast Pressure Zone and is currently the largest finished water producer in Westfield's distribution system. The plant is located outside of Washington Township, in neighboring Noblesville Township. This plant is primarily fed by the River Road Well Field which has a maximum capacity of approximately 8.2 MGD, but can draw additional water from the Cherry Tree and Welcome well fields via mains that interconnect them.

Treatment at this facility consists of aeration, detention, filtration, chlorination, and fluoride addition. A 500,000-gallon clearwell is also housed at this facility. Recent upgrades to this facility include the addition of a new chlorine building, two 34,500-gallon detention tanks, a new aeration unit, two low-pressure filters, various electrical and instrumentation and controls upgrades, a new security system, and improvements to increase the chlorine and fluoride feed systems capacities to treat as much as 10 MGD at the maximum daily feed rate.

The rated capacity of this facility remains at 7.5 MGD due to the limitation of available detention time. Currently, there is a total of 138,000 gallons of tank volume which allows for 26.5 minutes of detention time for 7.5 MGD. For a full 10 MGD, the detention time in these tanks drops to approximately 20 minutes. It can be shown that iron and manganese oxidation reactions are

complete in the 20 minutes of detention time currently available at this facility, but a demonstration of this to the state permitting agency must be completed before the facility can increase its capacity rating to the full 10 MGD.

4.2.2 Cherry Tree Water Treatment Plant

The Cherry Tree Treatment Plant provides up to 3.5 MGD of finished water to Westfield's Southeast Pressure Zone. Treatment at this facility consists of aeration, detention, filtration, and chlorination. Approximately 30,000 gallons of finished water storage is housed at this facility in the underground clearwell which feeds the high-service pumps.

A future expansion of this facility could increase treatment capacity to 5 MGD to more closely match the output of the Cherry Tree Well Field. Expansion of the distributive pumping capabilities at this site would not be necessary since capacity already exceeds 5 MGD. Facilities requiring expansion are the aeration, detention, filtration, and chemical feed. In addition, a 300,000-gallon clearwell is being considered at the site of the treatment plant.

4.2.3 Greyhound Pass Water Treatment Plant

The Greyhound Pass Treatment Plant provides finished water to the Southwest Zone and has a treatment capacity of approximately 330 gallons per minute (gpm). Treatment at this facility consists of chlorination as well as aeration, detention, and filtration which is implemented via a packaged treatment plant called a Unilator. Typical production from this plant is usually around 300 gpm due to the limited production of the onsite wells.

Finished water storage is also housed at this facility in the form of two ground-level reservoirs, one 150,000-gallon reservoir and one more recently constructed 300,000-gallon reservoir which was installed in 2007. A unique feature of this treatment facility is that the finished water reservoirs are fed both from the treatment plant and from the adjacent Southeast Pressure Zone. Finished water from the Southeast Pressure Zone is fed to the reservoirs via a 4-inch main located directly to the east of the plant. This main was installed in 2006 to provide additional finished water capacity to the Southwest Pressure Zone. There are no plans for expansion of this facility.

4.2.4 North Water Treatment Plant

The North Water Treatment Plant is the only treatment facility currently providing finished water directly to Westfield's North Pressure Zone. This plant originally used pressure filter treatment technology and has the capacity to treat and pump more water than the onsite well can supply. The maximum amount of finished water that can be provided by this facility is therefore approximately 330 gpm; however, this facility typically produces around 140,000 gpd (0.14 MGD). The last major rehabilitation of this facility was completed in 2005 when a Unilator was added to the treatment train to replace the original pressure filter. There are no plans for expansion of this facility.

4.2.5 Future Treatment Facilities

Ultimately, the projected maximum day demand for full buildout of the study area would require a maximum of 33.23 MGD of treatment capacity within the Westfield System. There is currently 11.57 MGD of treatment capacity in the system from the four treatment plants described above. Since the current maximum day demand reaches approximately 5.1 MGD, there is still room for the distribution system to grow without the construction of additional treatment facilities. One example of the amount of growth that would be possible with the existing capacity of the system would be the following scenario: 100-percent buildout of the Southeast Pressure Zone, 100-percent buildout of the Southwest Pressure Zone, and 15-percent buildout of the North Pressure Zone. These zones are currently built out to 47 percent, 17 percent and 4 percent, respectively, with the majority of development occurring in the Southwest and Southeast Pressure Zones.

In order to reach the capacity projected for full buildout of the study area, additional treatment facilities are required. To add capacity, the River Road and Cherry Tree treatment plants can be expanded to reach their full buildout capacities of 10 MGD and 5 MGD, respectively. The total available treatment capacity within the distribution system would increase to 15.57 MGD (or 47 percent of full buildout) after the expansion of these facilities.

The proposed future treatment facilities for expanding system treatment capacity to 33.23 MGD are both located in the Southeast Pressure Zone due to the availability of raw water in the area. One new facility would consist of a second 10.0 MGD treatment plant at the River Road site. This new facility would be nearly identical to the existing River Road WTP with aeration, detention, filtration and chemical addition including chlorine and fluoride. The other proposed facility would be located at the Horseshoe property. This property is the site of the new wells proposed for installation in 2010; Well No. 14, Well No. 15, and Well No. 16. Projected capacity for this site is currently around 6 MGD, but an additional amount of raw water could be diverted from nearby well fields by connecting the raw water mains between existing wells in the area. **Table 4.3** gives a summary of the treatment facilities that would be in operation at full buildout.

Table 4.3
SUMMARY OF FUTURE TREATMENT

Facility ID	*Maximum Treatment Capacity*
River Road WTP No. 1	10.0 MGD
*River Road WTP No. 2**	10.0 MGD
Cherry Tree WTP	5.0 MGD
Greyhound Pass WTP	0.43 MGD
North Plant	0.14 MGD
*Horseshoe WTP**	8.0 MGD
TOTAL	**33.57 MGD**

**Future treatment facilities*

4.3 EVALUATION OF PUMPING CAPACITY

According to Ten States Standards, pumping facilities should be designed to provide a sufficient amount of finished water to the distribution system during the system's maximum day demand

with the largest pump out of service. This assessment of the pumping capacity with the largest pump taken out is also called firm capacity. In the City of Westfield, finished water is pumped into the distribution system by way of the four existing treatment plants discussed above and supplemented through two booster stations: the 161st Street Booster Station and the West Tank Booster Station. Firm capacity for this system has been evaluated for each pressure zone in the subsequent sections.

One unique feature in this system that should be mentioned is the operational flexibility of the 161st Street Booster Station. This booster station has three pumps, one of which can be used to pump water to either the North Pressure Zone or the Southwest Pressure Zone, depending on demand. The pump can be switched between pressure zones by changing the configuration of opened and closed valves at the station. These valves are manually operated and therefore require the presence of Westfield staff to make the switch. Each of the other two pumps at the station are dedicated, one to the North Pressure Zone and one to the Southwest Pressure Zone. The Westfield distribution system's pumping facilities are summarized in **Tables 4.4, 4.5 and 4.6** for each pressure zone.

4.3.1 Southeast Pressure Zone

Any finished water that is required in the Southwest and North Pressures Zones but not produced in them is currently produced in the Southeast Pressure Zone and then pumped into the others. Currently as much as 18.8 MGD (15.8 MGD firm capacity) can be pumped into the Southeast Zone directly from the Cherry Tree and River Road water treatment plants if that amount of supply and treatment were available.

Table 4.4
SUMMARY OF SOUTHEAST ZONE PUMPING

Facility ID	*Pump ID*	*Design Flow*	*Design TDH*
River Road WTP	HSP No. 4	2,100 gpm (3.0 MGD)	366 feet
	HSP No. 5	2,100 gpm (3.0 MGD)	366 feet
	HSP No. 6	2,100 gpm (3.0 MGD)	347 feet
	HSP No. 7	2,100 gpm (3.0 MGD)	366 feet
Cherry Tree WTP	HSP No. 1	1,700 gpm (2.4 MGD)	242 feet
	HSP No. 2	1,700 gpm (2.4 MGD)	242 feet
	HSP No. 3	1,400 gpm (2.0 MGD)	242 feet
Total Capacity		**18.8 MGD**	
Firm Capacity		**15.8 MGD**	

The future average day demand of the Southeast Zone is 1.43 MGD, and the future maximum day demand is 2.15 MGD. Therefore, this zone does not currently have need for additional pumping facilities to meets its own customer demands for the projected future demand condition. However, increasing the pumping and treatment capacity of the River Road WTP, the Cherry Tree WTP, or both, will be a part of the solution to increasing the capacity available to the Southwest and North pressure zones provided that additional booster facilities are constructed to feed those zones.

4.3.2 Southwest Pressure Zone

The Southwest Zone is currently served by the Greyhound Pass WTP and the 161st Street Booster Station. **Table 4.5** summarizes the pumps currently in use at these two facilities. The Greyhound Pass WTP has one well that produces approximately 300 gpm; however, this facility also serves as a booster station. In 2006, a 4-inch line was installed to connect the Southeast Zone to the Greyhound Pass finished water reservoir. This new connection allows finished water from the Southeast Zone to fill Reservoir No. 1 which can then be pumped into the Southwest Zone via the WTP's high-service pumps. A second reservoir (Reservoir No. 2) was added in 2007 to increase the storage capacity of this facility from 150,000 gallons to 450,000 gallons.

Table 4.5
SUMMARY OF SOUTHWEST ZONE PUMPING

Facility ID	*Pump ID*	*Design Flow*	*Design TDH*
Greyhound Pass WTP	HSP No. 1	160 gpm (0.23 MGD)	150 feet
	HSP No. 2	200 gpm (0.29 MGD)	150 feet
	HSP No. 3	300 gpm (0.43 MGD)	150 feet
161st Street Booster	Pump No. 1	2,000 gpm (2.9 MGD)	120 feet
	Pump No. 2*	1,000 gpm (1.4 MGD)	120 feet
	Total Capacity	**5.25 MGD**	
	Firm Capacity	**2.35 MGD**	

**Can be used for North or Southwest Pressure Zone.*

The combination of flow from the 161st Street Booster Station and the Greyhound Pass WTP is capable of providing the 3.9 MGD average day demand projected for full buildout of this pressure zone. However, to meet Ten States Standards requirements, these two facilities must be able to supply the maximum day demand with the largest pump out of service. The firm pumping capacity into this pressure zone is 2.35 MGD.

The projected maximum day demand for full buildout in this pressure zone is approximately 5.85 MGD. This future demand will be met via booster facilities designed to supply water to the Southwest Pressure Zone from the Southeast Pressure Zone. The Greyhound Pass facility in conjunction with the 161st Street Booster Station is capable of meeting the projected maximum day demand provided that all flow from the booster station is directed to the Southwest Pressure Zone. This can be achieved by closing the gate valve on the 12-inch line to the North Pressure Zone and opening both of the butterfly valves on the 12-inch discharge header at the station. The 20-inch main, which connects the 161st Street Booster Station to the Southwest Pressure Zone, is capable of conveying the entire projected demand of that zone while staying below a velocity of 6 feet per second (fps). If all of the flow from this existing booster station is redirected to the Southwest Pressure Zone, additional facilities will need to be constructed to convey finished water to the North Pressure Zone.

4.3.3 North Pressure Zone

Existing demand in the North Pressure Zone is met via the combination of flows from the North Water Treatment Plant and the 161st Street Booster Station. The North WTP is currently capable of supplying 0.14 MGD and the 161st Street Booster Station has the capability of supplying as much as 2.9 MGD if Pump No. 2 and No. 3 at that station are both set to pump into the North Zone (1.4 MGD if only Pump No. 3 is directed into the North Zone). This pressure zone, because of its size compared to the other two zones, has the most growth potential in terms of both land development and demand. Currently, the North Zone average day demand is less than 2 MGD.

Table 4.6
SUMMARY OF NORTH ZONE PUMPING

Facility ID	*Pump ID*	*Design Flow*	*Design TDH*
North WTP	HSP No. 1	300 gpm (0.43 MGD)	168 feet
	HSP No. 2	300 gpm (0.43 MGD)	160 feet
161st Street Booster	Pump No. 2*	1,000 gpm (1.4 MGD)	120 feet
	Pump No. 3	1,000 gpm (1.4 MGD)	120 feet
	Total Capacity	**3.66 MGD**	
	Firm Capacity	**2.26 MGD**	

**Can be used for North or Southwest Pressure Zone.*

The full buildout condition for the North Pressure Zone is calculated to have an average day demand of as much as 16.81 MGD with a maximum day demand of 25.23 MGD. Providing for this amount of projected demand will require additional infrastructure to either produce more finished water within the pressure zone, or to increase the amount of finished water being pumped into the zone. Since the Wabash Tributaries Aquifer along the White River in the Southeast Pressure Zone will continue to be the most abundant source of raw water within the service area, according to the Capacity Analysis completed by Eagon & Associates, the construction of additional booster pumping facilities to convey finished water from the Southeast Pressure Zone to the North Pressure Zone is the best option.

CHAPTER 5
DISTRIBUTION SYSTEM ANALYSIS

5.1 BACKGROUND

The Westfield Distribution System contains several types of pipe materials including cast iron, cement-asbestos, ductile iron and PVC. Some of the first water mains installed in the system were installed as early as the 1930s. These older mains are generally the smaller mains in this system and cause the most frequent maintenance issues, such as main breaks and pressure problems. - Newer mains, those installed after the 1970s, are in generally good condition and typically cement lined ductile iron or PVC pipe materials.

5.2 FIRE-FLOW ANALYSIS

Analysis of the current condition of the City of Westfield's distribution system revealed favorable results regarding the capability of existing infrastructure to provide needed fire flows. As discussed in **Section 2.1.2** a fire-flow analysis was calculated using the 2008 Maximum Day Scenario to determine available fire flow. The results of the analysis indicate that 45 percent of the system nodes have more than 3,000 gallons per minute (gpm) of available fire flow, approximately 72 percent had more than 2,000 gpm available and 99.5 percent had 1,000 gpm or more available fire flow. Only 30 out of a total of approximately 1,500 system nodes had less than 1,000 gpm of available fire flow. Of those 30, all but two small groupings of nodes were limited by the fact that they were fed by 4-inch or smaller mains.

The first grouping is the Maple Knoll subdivision off of Spring Mill Road just south of 169th Street in the Southwest Pressure Zone, which is served by 8-inch mains throughout. The other grouping is the Ashfield subdivision off of Moontown Road, just north of SR 32, which is fed by 8-inch mains with courts that are served by 6-inch mains. Both areas have more than 800 gpm of available fire flow at each node, according to the model, and both will have access to a greatly increased amount of fire flow once planned infrastructure is put in place that provides looping back to larger mains in the system.

5.3 EVALUATION OF STORAGE CAPACITY

The required storage volume for Westfield's distribution system was assessed based on the amount necessary for fire storage plus the amount needed to equalize the usage peaks during the maximum day demand. According to the Insurance Services Office (ISO) which rates communities' ability to suppress fires, the maximum basic fire flow is 3,500 gpm. The American Waterworks Association's Manual titled 'Distribution Requirements for Fire Protection' states that for a modest-sized fire requiring up to 3,500 gpm, the duration is three hours. Thus, the fire storage requirement is calculated to be 3,500 gpm over 3 hours or 0.63 million gallons (MG).

The amount of storage needed to provide equalization for diurnal fluctuations during a maximum demand day is called equalization storage. The equalization storage in a system allows the system's pumps to continue running at constant speed while demands increase above or

decrease below their design capacities (i.e., without pumps turning on and shutting off as demand fluctuates). This allows tank levels to rise and fall with varying demand instead of pumps turning off when demand drops and turning back on when demand rises. Thus, the peak-hour demand is met via the storage in the system.

For this system, equalization storage requirements are evaluated based on the percentage of the total day's demand that would have been needed to serve the peak-hour demands for several maximum demand days in 2008. The maximum day demand in 2008 was determined to be 5.2 MGD from data collected through the City of Westfield's SCADA system. Three days in 2008 reached this maximum demand: Thursday, July 17th; Friday, August 22nd; and Sunday, September 28th. **Figure 5-1** illustrates the variations in demand experienced by the distribution system during these three days. In this figure the demand for each hour is normalized based on the total demand for that day, so that 1.00 is the average demand flow for that day. In this way we can compare each of the days based on their variation from the average.

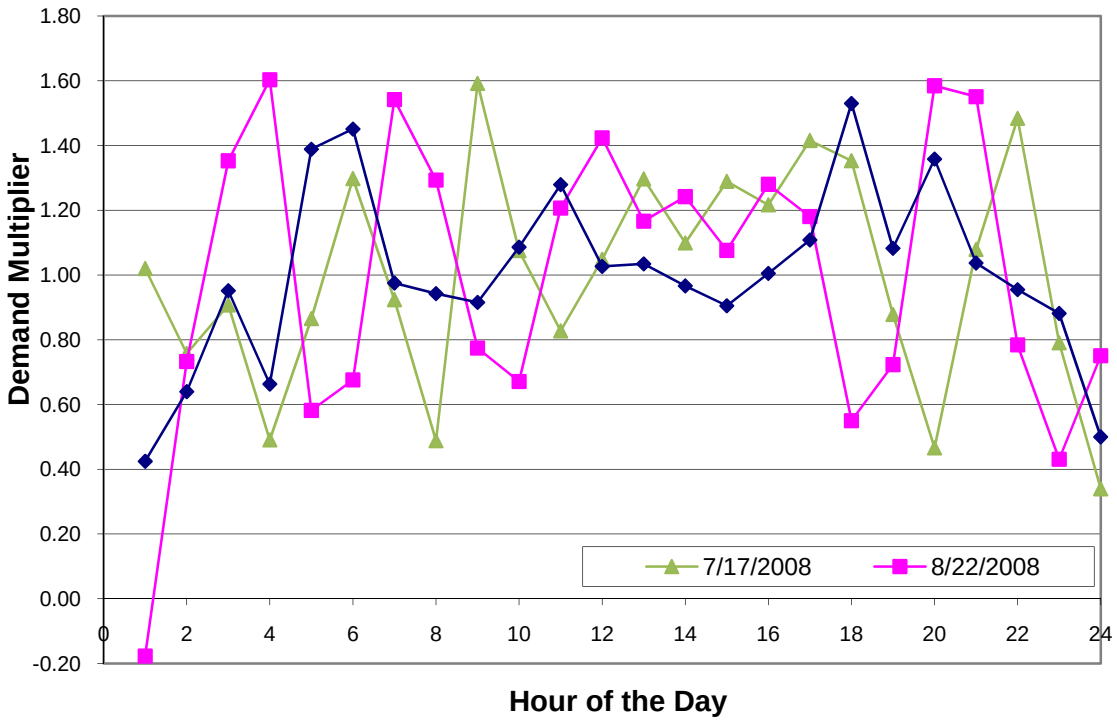

Figure 5-1
MAXIMUM DAY DIURNAL DEMAND

Table 5.1
EQUALIZATION STORAGE ANALYSIS

Day	*Storage Needed*	*% Equalization*
Thurs. 07/17/2008	0.42 MG	13.6 %
Fri. 08/22/2008	0.56 MG	18.8 %
Sun. 09/28/2008	0.50 MG	9.7 %

Table 5.1 lists the equalization storage that was needed for each of the three days reviewed and the percentage of the total day's demand. The equalization percentages average out to approximately 14 percent, which fits well within the range of 5 to 15 percent typical for a system using constant speed pumps set to follow demand throughout the day. A somewhat more conservative value of 15 percent has been chosen to evaluate the needed storage capacity. Based on this, the Westfield distribution system's current condition requires approximately 15 percent of 5.2 MGD, or 0.78 MG of equalization storage. This amount plus the amount of fire storage needed (0.63 MG) gives the City of Westfield's current storage need: 1.41 MG of finished water. **Table 5.2** contains a summary of the system's existing finished water storage which currently totals approximately 2.6 MG.

Table 5.2
SUMMARY OF SYSTEM STORAGE

Tank ID	*Pressure Zone*	*Volume*	*Type*
146th Street	Southeast	500,000 gallons	Elevated
161st Street	Southwest	500,000 gallons	Elevated
181st Street (West)	North	300,000 gallons	Elevated and Boosted
GTE	North	300,000 gallons	Elevated
River Road Clearwell	Southeast	500,000 gallons	Ground
Cherry Tree Clearwell	Southeast	30,000 gallons	Ground
Greyhound Pass Reservoir No. 1	Southwest	150,000 gallons	Ground
Greyhound Pass Reservoir No. 2	Southwest	300,000 gallons	Ground
TOTAL		**2.58 MG**	

For the full buildout condition, which we have determined has a projected maximum day demand of 33.23 MGD, the City of Westfield would need to increase its storage capacity to 5.6 MG (0.63 MG for fire, plus 15 percent of 33.23 MGD) to meet equalization and fire storage needs. In order to obtain this amount of storage, additional tanks have been placed in the system hydraulic model; the planned and previously designed 1.0 MG tank at Maple Knoll Subdivision, as well as three 0.75 MG elevated tanks in the locations shown on **Figure 5-2**: Towne Road, 186th Street, and 206th Street.

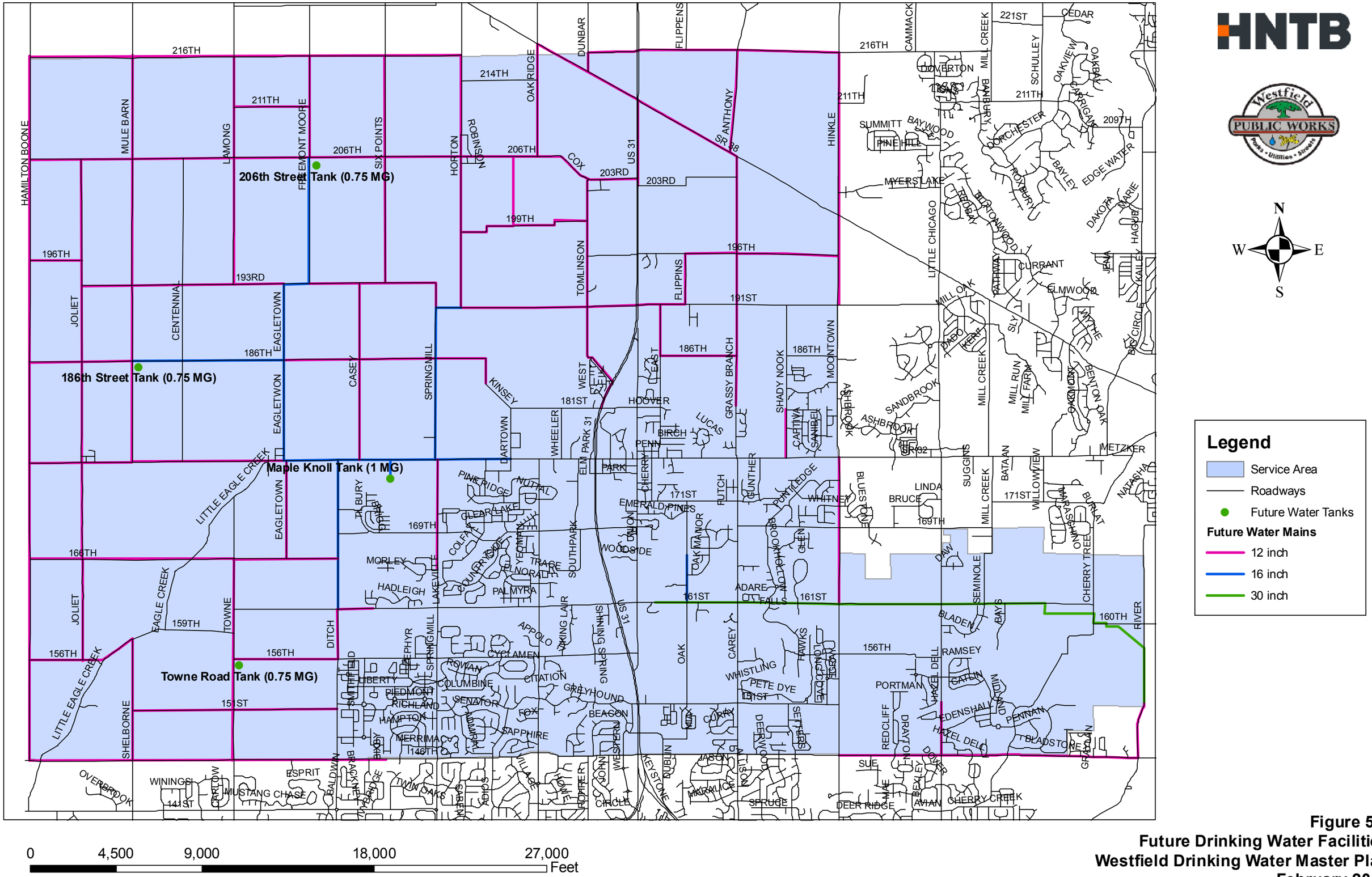

Figure 5-2
Future Drinking Water Facilities
Westfield Drinking Water Master Plan
February 2010

The current storage capacity of the distribution system is sufficient to serve Westfield until the system's maximum day demand begins to approach 13 MGD according to the design criteria described above. **Table 5.3** illustrates what additional storage infrastructure should be installed at what times in the growth of the system's maximum day demand. The order in which the three 0.75 MG elevated storage tanks is built will depend on the area of the distribution system that develops first. For instance, as demand nears the 19.7 MGD milestone, if development has continued to spread mostly to the west, the Towne Road Tank would be the next to be installed.

Table 5.3
FUTURE SYSTEM STORAGE

Maximum Day Demand Reached	*Infrastructure to Be Added*	*New Storage Volume Total*
13.0 MGD	1 MG (Maple Knoll)	3.58 MG
19.7 MGD	0.75 MG Tank	4.33 MG
24.7 MGD	0.75 MG Tank	5.08 MG
29.7 MGD	0.75 MG Tank	5.83 MG

5.4 DISTRIBUTION SYSTEM PLANNING

Figure 5-2 shows the Westfield distribution system as it is projected for full buildout of the study area. The distribution mains shown can be used as a pattern for the Utilities staff as development occurs in areas not yet served. However, development will most likely occur somewhat randomly and may therefore not initially have the benefits of the looping that is built in to the current full buildout scenario. It is for this reason that each new development should be individually considered, and planned mains that are not yet constructed should be given special consideration.

The total amount of new piping needed for the proposed layout of the full buildout distribution system is approximately 590,000 feet. This planned piping network consists of two sizes of distribution piping in the amounts listed in **Table 5.4**.

Table 5.4
FUTURE DISTRIBUTION PIPING

Pipe Size	*Length (feet)*
16-inch	60,000
12-inch	530,000

5.5 SPECIAL PROJECTS

Several large water main projects have been discussed as projects that would provide improvement to the functioning of the distribution system. Two specific projects will be discussed here: the Proposed River Road Water Treatment Plant (WTP)/Cherry Tree WTP Interconnection; and the Proposed 30-Inch Water Main from River Road.

5.5.1 Proposed River Road WTP/Cherry Tree WTP Interconnection

The River Road WTP/Cherry Tree WTP Interconnection project seeks to connect the existing 12-inch main running northeast along Cherry Tree Road to the 24-inch main running west along 160th Street as shown in **Figure 5-3**. The project involves the installation of approximately 2,600 feet of 12-inch main. The existing 16-inch main connecting the Cherry Tree WTP to the distribution system travels approximately 6,500 feet before connecting into a looped portion of the system. The connection of this 12-inch main would provide a secondary supply / connection point for both the Cherry Tree WTP and the River Road WTP.

Currently, the Cherry Tree WTP discharges into a 16-inch main that extends approximately 3,000 feet before connecting to a point in the distribution system that is looped within the system. The 24-inch main connecting the River Road WTP to the distribution system travels for approximately 17,000 feet before connecting into a looped portion of the system. Making the proposed connection would eliminate the reliance on a large portion of the 16- and 24-inch mains as the single point of connection from each plant, allowing water to be provided from both plants during the event of a main break or similar instance at any point west of the interconnect point. In addition, analysis using the WaterCAD model of the distribution system reveals that the high-service pumps at both the Cherry Tree WTP and the River Road WTP will run at more efficient points on their curves as a result of the proposed main.

5.5.2 Proposed 30-Inch Water Main from River Road WTP

This project is based on the need for additional finished water in the North and Southwest Pressure Zones which are both fed by the 161st Street Booster Station. The new 30-inch main will travel north along River Road from the River Road WTP and then travel west along 161st Street to the 161st Street Booster Station. The necessity for this project will not become a high priority until the overall demand of the system has reached approximately 15 MGD. As flow from the River Road WTP increases, the head that is required to overcome friction losses in the 24-inch main will increase causing the high-service pumps to operate further back on their curves, which will decrease the available output. The addition of this proposed main would reduce the overall friction losses that the River Road high-service pumps must overcome, allowing the pumps to operate farther out on their curves and increasing available output.

In addition, the proposed main would provide a secondary connection from the River Road WTP to the distribution system providing redundancy in the case of a main break. This redundancy is especially critical for this facility due to the fact that the River Road WTP provides the majority of the finished water to the Westfield distribution system. This new main will also be essential to the utility's ability to transport additional flow to the distribution system's three pressure zones as new treatment facilities are constructed in the area to meet increasing demand.

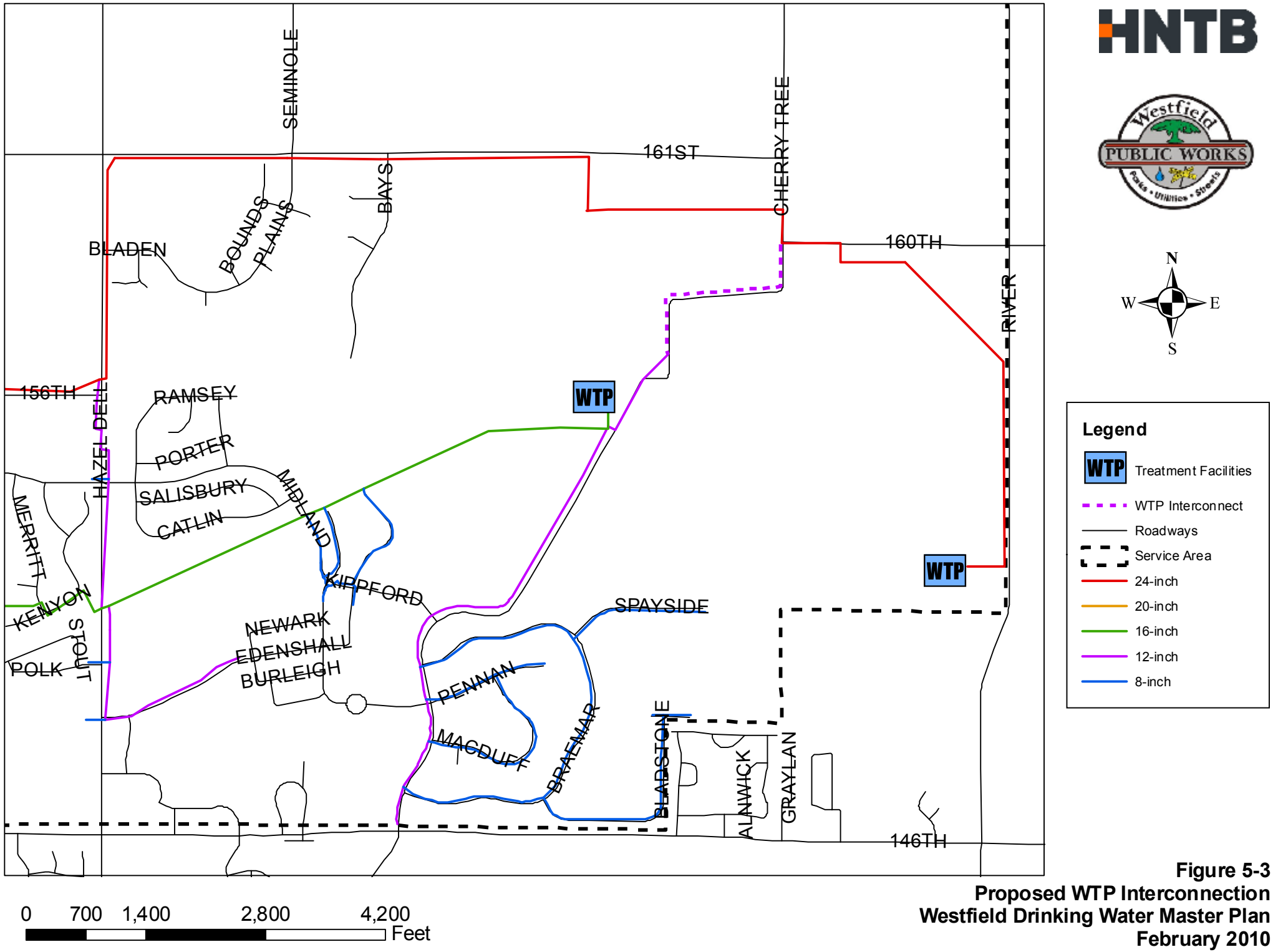

Figure 5-3
Proposed WTP Interconnection
Westfield Drinking Water Master Plan
February 2010

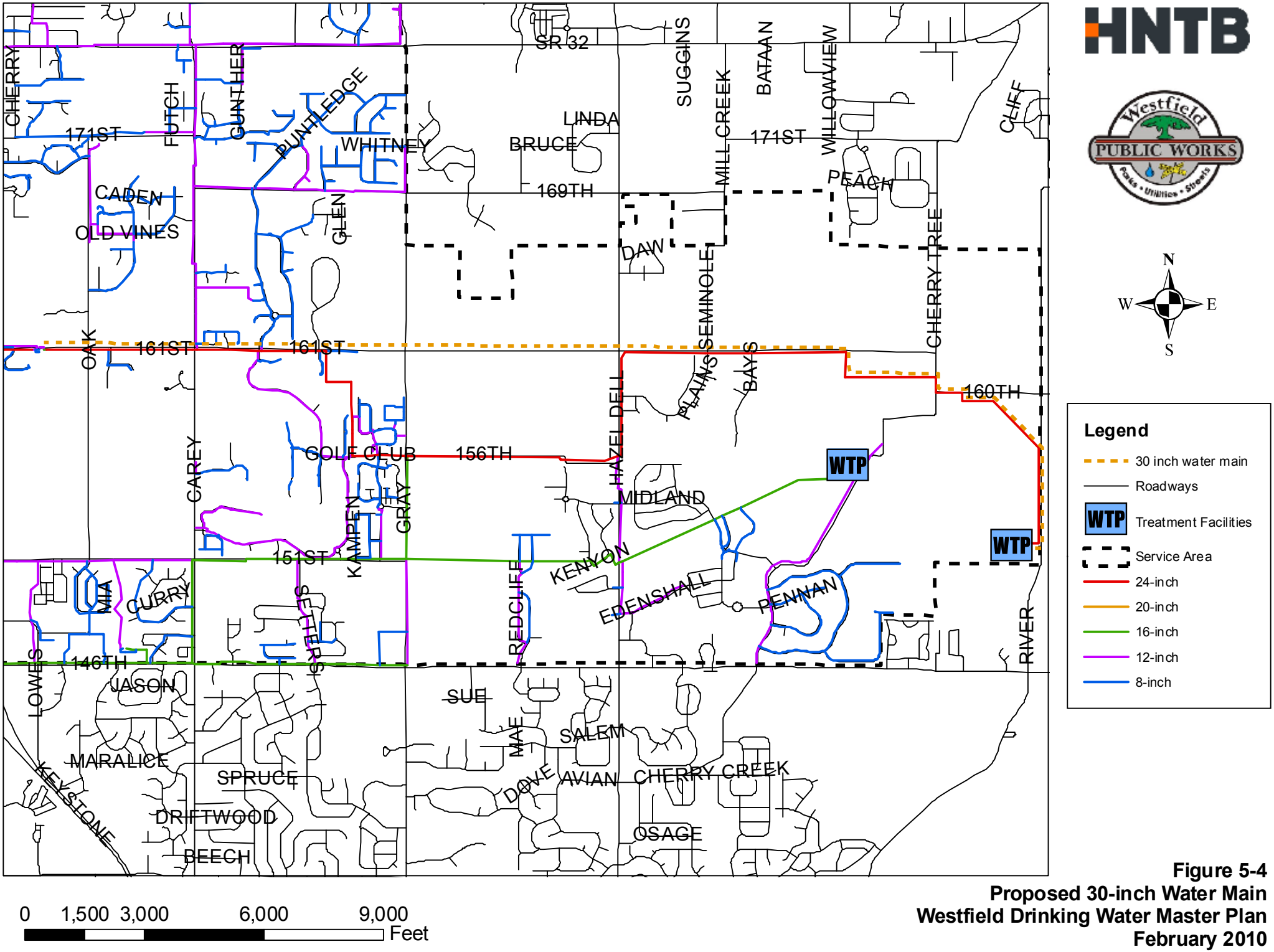

Figure 5-4
Proposed 30-inch Water Main
Westfield Drinking Water Master Plan
February 2010

CHAPTER 6
SUMMARY OF RECOMMENDATIONS

6.1 SUPPLY AND TREATMENT

Analysis of the system's existing components shows that in terms of raw water availability, treatment capacity, and pumping capacity the City of Westfield's system either meets or exceeds requirements for current demand conditions. The existing raw water supply will be able to keep up with drinking water demand of up to 18 MGD without the Horseshoe wells, or up to approximately 24 MGD after the Horseshoe wells have been constructed. As maximum day demands of the system increase above the amount of available treatment (11.57 MGD or 14.07 MGD with River Road at 10 MGD), either existing facilities will need to be expanded or new facilities will need to be constructed.

The existing pumping capacities of the system could last for some time into the future depending upon where future development occurs since each of the zones is not equally equipped to handle future increases in demand. As maximum day demand in the Southwest Pressure Zone reaches approximately 4 MGD, new or expanded pumping facilities will be needed to transport additional finished water from the Southeast Zone unless the full flow capacity of the 161st Street Booster Station is directed to the Southwest. A new booster pumping facility designed to convey finished water to the North Pressure Zone would have to be completed before the 161st Street Booster Station could be repurposed in this fashion. As the system is currently arranged, the North Pressure Zone pumps can handle up to nearly 3 MGD before additional pumping facilities will be required. In contrast, the Southeast Pressure Zone already has more pumping capacity than required for the full buildout scenario. Any new pumping facilities incorporated into this pressure zone would be installed with the intent of conveying an increased amount of finished water to the other two pressure zones.

6.2 DISTRIBUTION SYSTEM

Like many communities, the City of Westfield's oldest distribution infrastructure lies in the heart of the City in the downtown area just east of US 31 at State Road 32. The City's older mains do not currently cause any serious maintenance issues for the City and do not have imminent need of replacement based on the distribution system's capability of meeting demand in that area. However, as future unrelated infrastructure projects (roadway reconstruction, sidewalk replacement, etc.) are proposed in the areas of these older cast-iron and cement pipes, it would be considered prudent to replace some of these pipes with newer, and in some instances, larger pipes.

As development occurs in the remote parts of the Westfield Public Works Department (WPWD) Service Area, the map of future drinking-water facilities provided in **Chapter 5** can be used to determine the appropriate size of water distribution mains that should be installed in the area. It is advantageous for the WPWD staff to continue their practice of crafting agreements with developers to pay the cost of upsizing some water mains as this approach provides cost savings and future distribution infrastructure for the utility. Future interconnect and large water main

projects as described in **Chapter 5**, will require appropriate planning and funding to be put in place so that the projects can be completed in the necessary timeframe. Existing system storage will continue to provide adequate capacity to the WPWD distribution system until the maximum demand of the system approaches 13 MGD.

6.3 PROPOSED TIMELINE FOR IMPROVEMENTS

In order to outline future drinking water infrastructure projects in a way that prescribes a timeline which can be used for planning purposes, it is first necessary to make some assumptions about the geographical way in which development will grow. **Figure 6-1** shows five areas, numbered in the sequence in which they are expected to develop. Each area also corresponds to a milestone in the percentage of full buildout reached. For example, once Area 1 has been developed, we expect the system to be 20 percent built out; once Area 2 has been developed, the system should be approximately 40 percent built out, and so on. By designating which portions of the currently undeveloped service area will develop first, we have the ability to plan on a more meaningful timeframe for infrastructure projects. **Table 6.1** shows the improvements that would be required prior to reaching each of the demand milestones.

Table 6.1
PROPOSED SYSTEM IMPROVEMENTS TIMELINE

% Buildout	*Maximum Demand*	*Projects*
20	6.65 MGD	Obtain IDEM Approval for 20-minute detention time at River Road WTP
40	13.29 MGD	Towne Road Tank (0.75 MG)
		New Booster Station from Southeast to North Pressure Zone (8 to 9 MGD)
60	19.94 MGD	30-inch Water Main
		River Road WTP No. 2 (10 MGD)
		Maple Knoll Tank (1 MG)
		Horseshoe Wells (5 to 6 MGD)
80	26.58 MGD	186th Street Tank (0.75 MG)
		New 5 to 6 MGD well field
		Direct all flow from 161st Booster Station to Southwest Zone
100	33.23 MGD	206th Street Tank (0.75 MG)
		New 5 to 6 MGD well field
		New Booster Station from Southeast to North Pressure Zone (8 to 9 MGD)
		Horseshoe WTP (8 MGD)

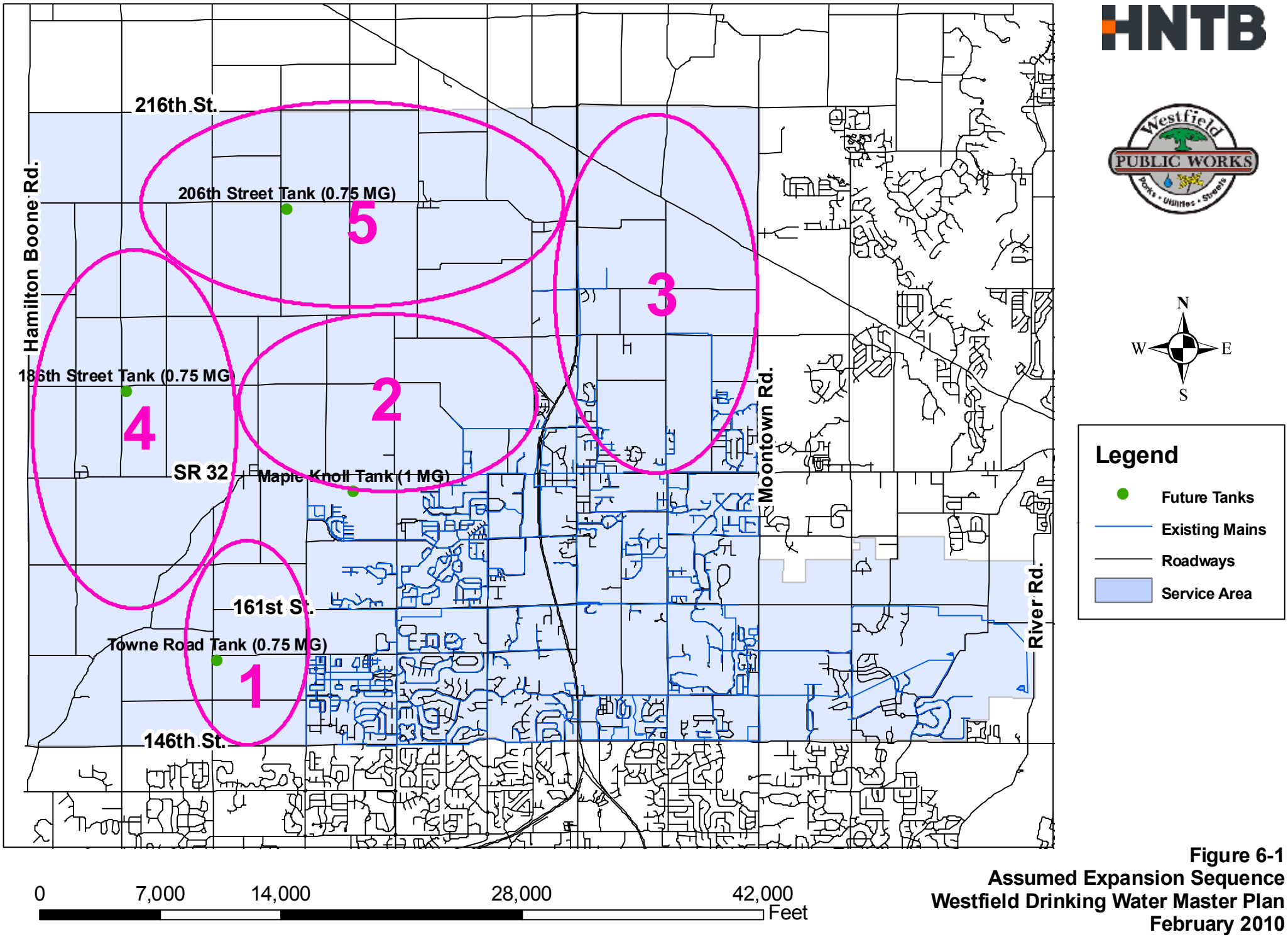

Figure 6-1
Assumed Expansion Sequence
Westfield Drinking Water Master Plan
February 2010

APPENDIX A

HYDROLOGIC CAPACITY ANALYSIS BY EAGON & ASSOCIATES

WELL-FIELD CAPACITYANALYSIS FOR THE RIVER ROAD AREA WELL-FIELDS WESTFIELD, INDIANA

Prepared for:

CITY OF WESTFIELD DEPARTMENT OF PUBLIC WORKS
2706 E. 171st Street
Westfield, IN 46074

Prepared by:

EAGON & ASSOCIATES, INC.
Worthington, Ohio 43085

November 3, 2009

LICENSED
STEPHEN J. CHAMPA
No. 2247
STATE OF INDIANA
PROFESSIONAL GEOLOGIST

Submitted by:

Stephen J. Champa, LPG
Hydrogeologist
Indiana License No. 2247

EAGON & ASSOCIATES, INC.
100 Old Wilson Bridge Road, Suite 115
Worthington, Ohio 43085
(614) 888-5760

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INTRODUCTION

The purpose of this report is to present the results of a well-field capacity evaluation for the City of Westfield Well Fields in the River Road area. Existing facilities are the River Road Well Field, the Cherry Tree Well Field, and the Welcome Well Field. The locations of wells and well fields are shown on Figure 1. Included in this report are an evaluation of current well-field capacity under various hydrologic conditions and an analysis of the additional capacity that might be developed in the River Road area to achieve the projected ground-water demand described in Section 2.0 of the Drinking Water Master Plan. Based on growth projections, the future demand for potable water will be approximately 22 million gallons per day (MGD) on average with a peak day demand of as much as 33 MGD. Additional model simulations also were performed to assess the impact on well-field production from filling of the Kinsey pond and the Heritage gravel pit.

BACKGROUND

As shown on Figure 1, there are currently nine production wells (Wells 5 through 13) at the three Westfield well fields in the River Road area. Well 14 has been installed and tested on the DeWitt property east of River Road between the gravel pit and the White River, but has not yet been put into production. Well construction details and current production rates are summarized on Table 1.

Wells at the Cherry Tree Well Field were installed between 1986 and 1988 and production rates vary from about 700 gpm for Well 7 to 1400 gpm for Well 5. Wells 8 and 9 at the River Road Well Field were installed in 1997 and 1998, respectively. Well 10 was installed in 2003. Pumping rates for each of the River Road Wells are around 2100 gpm. The Welcome Well Field was developed in 2007 and 2008. Well 11 has a production capacity of 1400 gpm and Wells 12 and 13 can produce 700 gpm each. The total capacity of all of the existing wells is approximately 18 MGD assuming good well performance and recharge.

Well 14 at the DeWitt Well Field was installed and tested in 2009. Analysis of the pumping-test data from the test of Well 14 indicates that between 5 and 6 MGD production can be achieved at the DeWitt Well Field with the installation of additional wells.

HYDROGEOLOGIC SETTING

The Westfield Well Fields are located within the Tipton Till Plain physiographic region (Fenelon & Bobay et. al., 1994). This area is characterized by relatively low surface relief and thick glacial deposits above bedrock. The thickness of the glacial drift in the area is generally between 100 to 250 feet. However, the depth to bedrock can also be quite shallow as is the case along the White River near Noblesville and at the Martin Marietta limestone mine west of Cherry Tree Avenue, just north of the Cherry Tree water treatment facility.

Unconsolidated aquifers within the study area can be separated into two aquifer systems as described in "Ground-Water Resources in the White and West Fork White River Basin, Indiana" (IDNR, 2002): the Tipton Till Plain Aquifer System and the White River and Tributaries Outwash Aquifer System.

The Tipton Till Plain Aquifer System consists of intratill sand and gravel deposits. These deposits are highly variable in depth and lateral extent and generally exist under confined aquifer conditions. Well logs indicate that these deposits can reach a thickness of up to 60 feet, but are typically 10 to 15 feet thick. The Westfield North Water Treatment Plant wells and Greyhound Pass wells are completed in this type of aquifer. The North Water Treatment Plant wells are near the northeast corner of the intersection of State Route 32 and U.S. 31 approximately five miles northwest of the Cherry Tree Well Field. Greyhound Pass Well 4 (GP-4) is located west of U.S. 31 just north of the intersection with 156th Street about five miles west of the Cherry Tree Well Field. Well GP-4 can produce about 300 gpm and this is probably as much as can be expected from these intratill deposits. Recharge to these deposits will vary depending on the depth to the deposit, but will most likely not exceed two inches per year.

The second unconsolidated aquifer system in the study area is the White River and Tributaries Outwash Aquifer System. This aquifer system generally parallels the White River and contains large volumes of sand and gravel that were deposited by meltwater from the retreating glaciers that filled the stream valleys. The aquifer in the White River Valley has a saturated thickness of up to 150 feet and is generally unconfined. This aquifer system is capable of providing large quantities of ground water to properly constructed wells. Well 8 at the River Road Well Field was tested at over 1800 gpm. The unconfined conditions allow for a relatively high recharge rate including recharge that is available under pumping conditions from induced infiltration of surface water from the White River.

Numerous gravel pits, both active and inactive, are present in the River Road area. The gravel pits represent a large volume of water in storage and offer the opportunity for rapid recharge from precipitation and run off. Recharge through the gravel pits also will occur if the river floods high enough to enter the gravel pits. According to Martin Marietta, most of the water pumped from the limestone mine is pumped into the gravel pits in the vicinity of the River Road Well Field, although some is discharged to Vestal Ditch and flows to the west, away from the well fields. Evaporation from the gravel pits depletes the volume of water in the pits. The evaporation rate from surface water in central Indiana is approximately 33 inches per year, according to "Ground-Water Resources of the United States" (Todd, 1983).

In places there is no unconsolidated aquifer material and the underlying carbonate bedrock is used as a ground-water supply for residential wells. The bedrock is generally not considered to be a good source for relatively higher yield wells. However, Indiana American Water (INAW) operates a bedrock well in Noblesville that produces about 700 gpm. Martin Marietta also pumps ground water from the bedrock at the underground limestone mine west of the River Road Well Field, but the amount of water coming from the bedrock is difficult to determine since discharge from the mine includes surface-water runoff and ground-water discharge is not metered.

WELL-FIELD CAPACITY

This well-field capacity analysis was performed using pumping-test data from the wells combined with the results of ground-water flow model simulations to determine steady-state drawdowns under conditions of high recharge, reduced recharge, and drought. Available drawdowns were determined for each well based on well construction data and static water–level elevations. The static-water levels, screen elevations, and available drawdown for each well are shown on Table 2. These values of available drawdown were used for the analysis of well-field capacity under high recharge conditions and were reduced based on changes in model non-pumping water-level elevations for the analyses of well-field capacity under reduced recharge and drought conditions.

Pumping-test data from the production wells were analyzed to determine long-term drawdowns and well loss at the pumping rates shown on Table 1. The time-drawdown trend of the data from the constant-rate tests was extrapolated to 180 days. The assumption of this analysis is that after 180 days the cone-of-influence will have spread far enough to capture sufficient recharge to balance pumping withdrawals and the system will be at steady-state. Well loss determined from the stepped-rate test data from each well was then subtracted from the 180 day drawdown. Well loss is the portion of drawdown in a pumped well that results from the turbulent flow of water into the well. The remaining drawdown is the formation drawdown. Formation drawdown was adjusted proportionally to determine drawdown at the current production rate of each well. Well loss was then recalculated for the production rate and added to the formation drawdown to determine the total steady-state drawdown for each well.

A ground-water flow model was used to determine interference drawdown between wells. The ground-water flow model was originally developed for the Wellhead Protection Area delineation performed in 2001. The model has been updated based on more recent test drilling and aquifer testing and has been calibrated to the pumping tests of the Westfield wells. To determine interference drawdowns, each well was pumped at the production rate shown on Table 1 and the drawdown at the other well locations was recorded for each pumping simulation. The total

interference drawdown at each well is the sum of interference drawdowns from each individual well pumping simulation.

High Recharge Conditions

The results of the analysis for 18 MGD production and conditions of high recharge are shown on Table 3. High recharge conditions simulate observed water-levels typical of spring and early summer. High recharge conditions as simulated in the model are:

- Recharge from precipitation of 10 inches per year (in/yr) over the outwash aquifer.
- Recharge from precipitation of 4 in/yr over areas outside of the outwash aquifer.
- River depth of three feet.
- Gravel pit depth of 20 feet.

As shown on Table 3, there is available remaining drawdown at all wells, however only Well 9 has more than 30 percent of the available drawdown remaining. Our standard practice is to leave 30 percent of the available drawdown remaining for loss of well efficiency over time and seasonal low water levels. For most of the wells where the remaining available drawdown is less than 30 percent it will be very important to maintain the condition of the wells at a high level of efficiency in order to achieve 18 MGD from these wells. A relatively small increase in well loss can be significant for wells with only a few feet of remaining available drawdown.

Simulation of 6 MGD production from three wells at the DeWitt Well Field also was performed under high recharge conditions. Interference drawdown at the other production wells ranged from 0.10 to 0.37 feet. So under high recharge conditions, production from the DeWitt Well Field will have a minimal affect on production from the other Westfield wells.

Model simulations were performed to assess drawdown from operation of Veolia Water Wells WRN-1, WRN-2, and WRN-3A. These wells are located south of the River Road Well Field, as shown on Figure 1. They were installed in 2007, but have not been used for production and we

have no information regarding the performance characteristics of these wells. According to the *Phase 2- Yield and Demand Study* for Veolia Water (Black & Veatch, October 2008), individual well capacities are between 1100 to 1500 gpm. Simulations of 1400 gpm production from each of the Veolia wells (6 MGD total) resulted in additional interference of 3.35 feet at Well 9, 2.81 feet at Well 10, 1.84 feet at Well 8 and between 0.10 and 0.49 feet at the other Westfield production well locations.

If Westfield were to acquire the Veolia Wells and the production rates used here are realistic, an additional 12 MGD production can be achieved from the DeWitt and Veolia Wells bringing the total production to 30 MGD under high recharge conditions. Combined with the approximately 1 MGD production from Greyhound Pass Well GP-4 and North Water Treatment Wells 2 and 4, that would leave a 2 MGD deficit compared to the maximum day demand of 33 MGD.

It is likely that this additional 2 MGD capacity can be developed in the River Road area to meet peak demand, but suitable well sites are becoming scarce due to existing land use and the limited extent of the best part of the aquifer. A test well was drilled for Indianapolis Water about 2100 feet south of the DeWitt test well (Well 14) on the east side of the White River. The depth to bedrock at that location is 66 feet and sand and gravel was logged from 32 to 66 feet. This is similar to the conditions that were encountered at the DeWitt property. Test borings completed west of the White River and North of 146th Street show that bedrock is very shallow (less than 30 feet)in that area.

In the past, the potential for a well west of Well No. 9 at the River Road Well Field has been evaluated. A well at this location would affect pumping levels at Wells 8, 9, and 10 and Veolia Wells WRN-1 and WRN-3A. It is likely that a well at this location would supply some additional capacity during high recharge conditions. Under conditions of reduced recharge or drought, a well west of Well No. 9 may only provide redundancy and the ability to distribute production between more wells.

Model simulations also were performed using the high recharge conditions to assess the effect of filling of the Kinsey pond and the Heritage gravel pit. For each simulation, recharge was reduced to two inches per year where the ponds were to represent reduced recharge that would occur due to the placement of low permeable compacted fill material in the ponds. The recharge rate through these materials could be even less. Model hydraulic conductivities also were reduced proportional to the reduction in aquifer saturated thickness. For the simulation of filling of the Kinsey pond, drawdown at Well 5 would be about 2.4 feet greater and drawdown at Well 13 would be about 3.2 feet greater. Drawdowns at Wells 6, 7, and 12 would increase by one to two feet and Wells 8, 9, 10, and 11 would be relatively unaffected. Filling of the Heritage gravel pit had no effect on pumping levels or interference drawdowns at the DeWitt property or any of the other Westfield Wells.

Reduced Recharge Conditions

The results of the analysis for 18 MGD production and conditions of reduced recharge are shown on Table 4. Reduced recharge conditions simulate observed water-levels typical of late summer and early autumn. Reduced recharge conditions as simulated in the model are:

- Recharge from precipitation of 7.4 inches per year (in/yr) over the outwash aquifer.
- Recharge from precipitation of 3 in/yr over areas outside of the outwash aquifer.
- River depth of 1.5 feet.
- Gravel pit depth of 15 feet (heads reduced by five feet).

The reduction of recharge used for this scenario and the drought scenario are based on evaluation of drought conditions using the Palmer Drought Index data for the Central Indiana Region. The data covers the time period from 1895 to the present. The most extensive drought in the period of record occurred between July 1933 and August 1936. Mild to extreme drought conditions prevailed throughout the 37 month period. The precipitation departure from normal throughout the drought was -10.56 inches per year, which is about a 26 percent reduction in precipitation compared to current normal precipitation amounts.

Table 4 shows that under reduced recharge conditions, pumping levels exceed available drawdown at Well 6 and are very close to exceeding available drawdowns at most of the other wells. Only Well 9 has a significant amount of remaining available drawdown. Since these projections are based on pumping well drawdowns when the wells were new, a relatively small increase in well loss from like-new conditions will result in pumping levels that exceed available drawdowns at many of the other wells.

This is consistent with what has been observed at the Welcome and Cherry Tree Well Fields over the last two years. A model scenario based on recent production from the Cherry Tree and Welcome Well Fields was performed to check the accuracy of the model. The simulation included total production of 6 MGD; 4 MGD from the Welcome Wells and 2 MGD from Well 5. Observation Well 07-2 at the Welcome Well Field is equipped with a transducer and data logger for continuous water-level monitoring. The results from this model scenario were consistent with the observed water level at Observation Well 07-2 in October 2009.

Drought Conditions

To simulate drought conditions, water levels in the gravel pits were reduced by an additional three feet. River depth and recharge from precipitation used for this scenario are the same as were used for the reduced recharge scenario. The precipitation amounts used for the reduced recharge scenario were based on drought conditions and flow on the White River is supported by discharge from Morse Reservoir and so the depth is unlikely to drop significantly lower than 1.5 feet.

The results of the analysis for 18 MGD production during drought conditions are shown on Table 5. Under the simulated drought conditions, available drawdown is exceeded at all of the wells except Wells 9 and 13. This indicates that production of 18 MGD from the existing production wells is not possible on a consistent basis during drought conditions. Depending on actual conditions and well performance at the time, it may not even be possible to meet a peak demand of 18 MGD.

The simulated drought conditions seem to be fairly extreme, but conditions could be even worse than simulated. To make a reality check of the model results, an empirical analysis of recharge available under drought conditions was performed. Sources of recharge to support well-field production are estimated as follows:

- Recharge from precipitation (2.35 MGD),
- Runoff to the gravel pits (0.65 MGD),
- Riverbed infiltration (1.9 MGD),
- Water pumped into the gravel pits from the limestone mine (3.5 MGD),
- Water in storage in the gravel pits (2.7 MGD), and
- Water in storage in the aquifer (6.7 MGD).

The ground-water flow model was used to determine the amount of water available from recharge and riverbed infiltration. Recharge was determined for the approximate area encompassed by the one foot drawdown contour for an 18 MGD pumping scenario. Total recharge was 2.35 MGD.

Runoff is a very complex parameter to estimate because it depends on the amount of rainfall in a particular precipitation event, the ground conditions, land use, and topography. As a rough approximation, runoff is about one third of total precipitation. Normal precipitation in the central Indiana area is about 40 inches per year, but that amount is reduced by about 26 percent during drought conditions. If precipitation during drought conditions is 30 inches per year, runoff would be about 10 inches per year. For the purpose of this analysis, we have assumed that 50 percent of runoff makes it to the gravel pits over an area of approximately 1750 acres west of the White River generally centered on the Westfield Well Fields, which results in a total of 0.65 MGD of recharge from runoff.

The amount of water available from induced riverbed infiltration was estimated based on the difference in the amount of water entering the model from the White River and Cicero Creek

between a simulation without the Westfield Wells pumping and a simulation of 18 MGD production from the Westfield wells. The difference in riverbed infiltration was 1.9 MGD.

According to Martin-Marietta most of the water pumped from the limestone mine is pumped to the gravel pits and the reported pumpage from the gravel pits in 2007 was 4.3 MGD. We used 3.5 MGD for this analysis, because Martin-Marietta claims that most of the water pumped from the mine is pumped to the gravel pits around the River Road Well Field.

The area of the gravel pits was calculated from aerial photos and the depth of the gravel pits was assumed to be 20 feet. The Heritage gravel pit and gravel pits north of the Heritage gravel pit were not used in this calculation. We assumed that 50 percent of water in the gravel pits could be captured resulting in 2.7 MGD of water available for recharge to pumping wells.

For the estimation of water available from aquifer storage, the amount of water per foot of aquifer over an area of 164 million square feet (3765 acres) was calculated using a storage coefficient of 0.2. This amount is 0.67 MGD per foot of aquifer so if 10 feet of aquifer is dewatered the rate of dewatering would be 6.7 MGD. The total amount of water available based on this analysis is 17.1 MGD.

CONCLUSIONS AND RECOMMENDATIONS

Based on the analyses presented in this report, the projected average and peak day demands of 22MGD and 33 MGD, respectively, can be achieved under high recharge conditions. Additional capacity will be required to maximize production potential. Acquisition of Veolia Wells WRN-1, 2, and 3A should be pursued and the DeWitt Well Field should be developed to help meet the projected future demand. Depending on the actual capacity that is available from the Veolia wells, it will most likely be necessary to develop some additional capacity. The best location would be south of the DeWitt wells between the gravel pit and the White River. Wells there would need to be sufficiently far from the DeWitt wells to avoid interference drawdown between the Well Fields (1000 to 2000 feet). An additional well at River Road Well Field could be installed. A test boring has

already been completed at this location so we know the formation is there. There will be interference between that well and the existing River Road wells and Veolia Wells WRN-2 and WRN-3A. The new well will probably supply some additional capacity during high recharge conditions, but may only provide redundancy under low recharge or drought conditions. At peak demand it may not be possible to continue selling water to Indianapolis, even under high recharge conditions.

Under reduced recharge conditions, it will most likely be possible to meet average daily demands with expansion of the system as suggested. It may be possible to meet peak demands, but probably not for extended periods of time. In either case, flexibility within the system will be very important. Future planning should include modifications to the distribution system that will allow water from any well to be pumped to any zone so that pumping can be distributed to maximize production based on water levels and well performance.

Wells also should be equipped with variable frequency drives (VFDs) so that pumping rates can be optimized to maximize production. Well maintenance also will be very important. A routine maintenance schedule should be developed so that the wells can be maintained at a high level of performance.

For new or replacement wells, well design using shorter, larger diameter well screens should be considered. Gravel pack designs, such as has been used at Well No. 9, can also be used. In theory a larger diameter well screen and gravel pack should increase the effective well radius and decrease well screen entrance velocities that should result in a decrease in drawdown due to well loss. Shorter well screens will allow for more available drawdown, but could result in lower specific capacities, so potentially there are some theoretical trade-offs that are difficult to quantify in advance of their implementation.

Under drought conditions, the recharge analysis shows that approximately 17.1 MGD is available. Adding in the approximately one MGD from GP-4 and the Water Treatment Plant wells, this leaves a deficit of about 4 MGD. Meeting even the projected average day demand will require

either development of additional sources of supply, possibly the use of surface water from the White River, or innovative methods to enhance recharge to the wells.

One way to enhance recharge to meet demands during times of low recharge or drought would be to use the gravel pits as artificial recharge basins. Water from the White River could be pumped into the gravel pits to keep heads in the ponds higher and supply recharge to the aquifer when it is most needed. Stream withdrawals during dry seasons might not be possible, but recharging during periods of high stream flow to increase aquifer storage to support pumpage during dry seasons may be beneficial. Based on recent observations, pumping into the DeWitt gravel pit pond and the Kinsey pond would have the most direct influence on pumping levels at the Westfield wells. If discharge from the limestone mine increases as the mine expands, this could also increase recharge to the aquifer as long as that discharge is directed into the gravel pits in the River Road area. Artificial recharge may also avoid the need for treatment as surface water as opposed to a direct surface-water supply. It might also be possible to construct an up-ground storage reservoir that could be used to pump water to the ponds during low recharge or drought conditions.

As new wells are added and more data is collected this analysis should be updated to include production and performance characteristics of the new wells. Continuing analysis of water-level data from the production and observation wells combined with production data, surface-water, and climate data will help to further refine our understanding of the ground-water and surface-water hydrology that affect well-field capacity in this area. It is clear that prudent planning and management will be required in order to meet demands for the water resources from this area.

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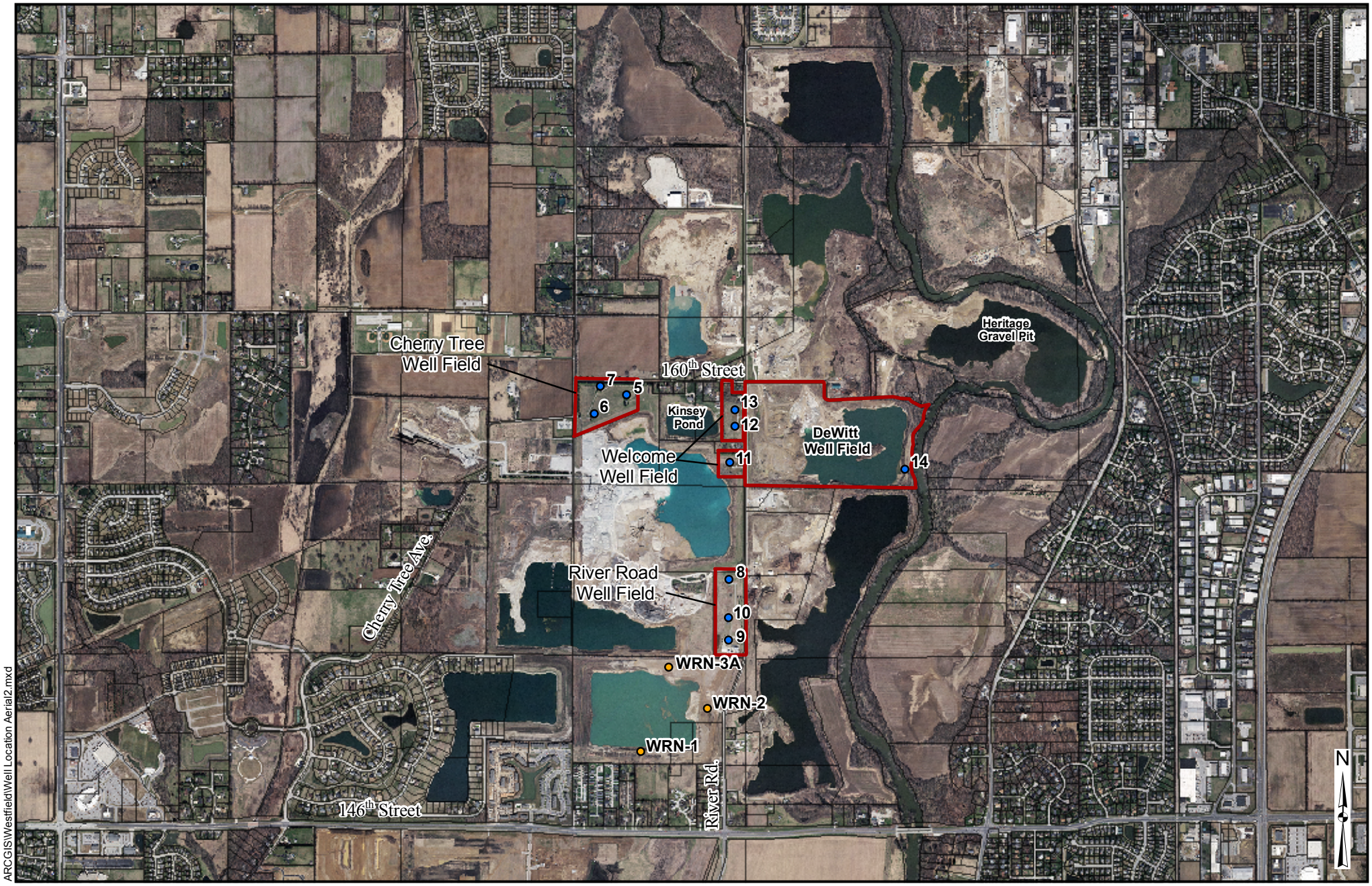
FIGURES



Legend

- Westfield Production Wells
- Veolia Wells
- Well Field Property Boundaries

0 2,000 4,000

Scale in Feet

Figure 1. Well Field Location Map

TABLES

TABLE 1.
WELL CONSTRUCTION SUMMARY
CITY OF WESTFIELD

Well Number	Well Field	Ground Elevation (feet, MSL)	Well Depth[1] (feet, bgl)	Depth to Top of Screen (feet, bgl)	Screen Length (feet)	Screen Slot-Size (inches)	Current Pumping Rate (gpm)
WELL 5	CHERRY TREE	764	78.0	55.0	23.0	0.100	1400
WELL 6	CHERRY TREE	766	75.5	52.0	23.5	0.100	1320
WELL 7	CHERRY TREE	764	72.0	50.0	22.0	0.100	700
WELL 8	RIVER ROAD	754	98.0	48.0	50.0	0.130	2100
WELL 9	RIVER ROAD	754	92.0	62.0	30.0	0.200	2100
WELL 10	RIVER ROAD	754	97.0	55.0	47.0	0.080 - 55-70 ft 0.040 - 70 -90 ft 0.100 - 90-97 ft	2050
WELL 11	WELCOME	767	84.0	62.0	20.0	0.100 - 62 - 76 ft 0.140 - 76 - 84 ft	1400
WELL 12	WELCOME	764	78.0	56.0	22.0	0.110 - 56 - 71 ft 0.090 - 71 - 78 ft	700
WELL 13	WELCOME	759	85.0	60.5	24.5	0.050 - 60.5 - 68 ft 0.120 - 68 - 85 ft	700
WELL 14	DeWITT	752	76.0	56.0	20.0	0.140	--

TABLE 2.
ELEVATIONS AND AVAILABLE DRAWDOWN
CITY OF WESTFIELD

Well Number	Well Field	Ground Elevation (feet, MSL)	Static Depth to Water (feet, bgl)	Water-Level Elevation (feet, MSL)	Depth to Top of Screen (feet, bgl)	Top of Screen Elevation (feet, MSL)	Available Drawdown (feet)
WELL 5	CHERRY TREE	764	16.5	747.5	55.0	709.0	38.5
WELL 6	CHERRY TREE	766	17.5	748.5	52.0	714.0	34.5
WELL 7	CHERRY TREE	764	17.5	746.5	50.0	714.0	32.5
WELL 8	RIVER ROAD	754	8.9	745.1	48.0	706.0	39.1
WELL 9	RIVER ROAD	754	11.8	742.2	62.0	692.0	50.2
WELL 10	RIVER ROAD	754	9.0	745	55.0	699.0	46.0
WELL 11	WELCOME	767	25.3	741.7	62.0	705.0	36.7
WELL 12	WELCOME	764	23.3	740.7	56.0	708.0	32.7
WELL 13	WELCOME	759	14.7	744.3	60.5	698.5	45.8
WELL 14	DeWITT	752	12.1	739.9	56.0	696.0	--

TABLE 3.
18 MGD DRAWDOWN ESTIMATES - HIGH RECHARGE CONDITIONS
CITY OF WESTFIELD

Well Number	Well Field	Pumping Rate (gpm)	Available Drawdown (feet)	Pumping Well Drawdown (feet)	Interference Drawdown (feet)	Total Drawdown (feet)	Remaining Available Drawdown (feet)	Remaining Available Drawdown (percent)
WELL 5	CHERRY TREE	1400	38.5	21.99	9.95	31.94	6.56	17.0
WELL 6	CHERRY TREE	1320	34.5	21.00	11.26	32.26	2.24	6.5
WELL 7	CHERRY TREE	700	32.5	12.59	15.88	28.47	4.03	12.4
WELL 8	RIVER ROAD	2100	39.1	23.24	9.66	32.90	6.20	15.9
WELL 9	RIVER ROAD	2100	50.2	17.94	11.56	29.50	20.70	41.2
WELL 10	RIVER ROAD	2050	46.0	26.96	13.33	40.29	5.71	12.4
WELL 11	WELCOME	1400	36.7	23.68	7.84	31.52	5.18	14.1
WELL 12	WELCOME	700	32.7	17.22	12.77	29.99	2.71	8.3
WELL 13	WELCOME	700	45.8	25.85	10.81	36.66	9.14	20.0

TABLE 4.
18 MGD DRAWDOWN ESTIMATES - REDUCED RECHARGE CONDITIONS
CITY OF WESTFIELD

Well Number	Well Field	Pumping Rate (gpm)	Available Drawdown (feet)	Pumping Well Drawdown (feet)	Interference Drawdown (feet)	Total Drawdown (feet)	Remaining Available Drawdown (feet)	Remaining Available Drawdown (percent)
WELL 5	CHERRY TREE	1400	33.74	21.99	9.88	31.87	1.87	5.5
WELL 6	CHERRY TREE	1320	29.71	21.00	11.20	32.20	-2.49	-8.4
WELL 7	CHERRY TREE	700	28.75	12.59	15.85	28.44	0.31	1.1
WELL 8	RIVER ROAD	2100	34.34	23.24	9.68	32.92	1.42	4.1
WELL 9	RIVER ROAD	2100	45.52	17.94	11.49	29.43	16.09	35.3
WELL 10	RIVER ROAD	2050	41.30	26.96	13.29	40.25	1.05	2.5
WELL 11	WELCOME	1400	31.96	23.68	7.30	30.98	0.98	3.1
WELL 12	WELCOME	700	28.01	17.22	11.79	29.01	-1.00	-3.6
WELL 13	WELCOME	700	41.15	25.85	10.57	36.42	4.73	11.5

TABLE 5.
18 MGD DRAWDOWN ESTIMATES - DROUGHT CONDITIONS
CITY OF WESTFIELD

Well Number	Well Field	Pumping Rate (gpm)	Available Drawdown (feet)	Pumping Well Drawdown (feet)	Interference Drawdown (feet)	Total Drawdown (feet)	Remaining Available Drawdown (feet)	Remaining Available Drawdown (percent)
WELL 5	CHERRY TREE	1400	31.24	21.99	9.97	31.96	-0.72	0.0
WELL 6	CHERRY TREE	1320	27.28	21.00	11.17	32.17	-4.89	0.0
WELL 7	CHERRY TREE	700	26.40	12.59	15.83	28.42	-2.02	0.0
WELL 8	RIVER ROAD	2100	31.64	23.24	9.69	32.93	-1.29	0.0
WELL 9	RIVER ROAD	2100	42.90	17.94	11.44	29.38	13.52	31.5
WELL 10	RIVER ROAD	2050	38.66	26.96	13.45	40.41	-1.75	0.0
WELL 11	WELCOME	1400	29.28	23.68	7.14	30.82	-1.54	0.0
WELL 12	WELCOME	700	25.36	17.22	11.87	29.09	-3.73	0.0
WELL 13	WELCOME	700	38.53	25.85	10.59	36.44	2.09	5.4

APPENDIX B

DEWITT PROPERTY WELL FIELD CAPACITY ESTIMATE

EAGON & ASSOCIATES, INC.
Consulting Geologists

100 Old Wilson Bridge Road, Suite 115 / Worthington, Ohio 43085 / (614) 888-5760 / FAX (614) 888-5763

May 27, 2009

Mr. Kurt Wanninger
Director of Public Works
City of Westfield
2706 E. 171st Street
Westfield, Indiana 46074

RE: Well Field Capacity Estimate
DeWitt Property

Dear Kurt:

The purpose of this letter is to present you with a well-field capacity evaluation for the DeWitt Property. The attached Figure 1 shows the locations of test borings 08-1 and 08-2, observation wells 09-1 and 09-2, and the test/production well (Well No. 14) at the DeWitt property.

On May 19, 2009 a stepped-rate test was performed. Well No. 14 was pumped at rates of 1070 gallons per minute (gpm), 1397 gpm, 1705 gpm, and 1999 gpm. The short-term specific capacity ranged from 87.3 gpm/ft at 1070 gpm to 77.5 gpm/ft at 1999 gpm. Well loss analysis of the stepped-rate test data indicates that well loss is approximately 24 percent at 1906 gpm.

On May 20-22, 2009, Well No. 14 was pumped at a rate of 1906 gpm for 48 hours. Pressure transducers were used to collect data from observation wells 09-1 and 09-2, the White River, and the gravel pit. Data from Well No. 14 were collected manually throughout the 48 hour test. The White River was on a declining trend throughout the test. Water-level data from the observation wells and the gravel pit show that water levels in the aquifer and the gravel pit were on a similar trend indicating a good hydraulic interconnection between the river, the aquifer, and the gravel pit. Drawdown data from the observation wells were corrected to account for the downward trend in water levels. Time-drawdown graphs of the pumping-test data indicate that recharge is encountered very rapidly under pumping conditions.

The data collected during the test were used to determine pumping well and interference drawdowns. Time-drawdown trends for the data from each well were extended to 180 days. It is assumed that after 180 days of continuous production the cone of influence will have reached far enough to intercept sufficient recharge to balance the pumping withdrawals. The extrapolated drawdowns after 180 days were used to construct a distance-drawdown graph. The distance drawdown graph was used to determine long-term interference drawdowns between wells. The pumping well drawdown is based on the extrapolated drawdown from Well No. 14. For the purposes of this analysis it is assumed that the specific capacity and well loss of any new well will be similar to the specific capacity and well loss of Well No. 14. This is a reasonable assumption based on the test-boring logs and grain-size data from location 09-2.

The attached Table 1 shows well and well screen depths and pumping well and interference drawdowns for Well No. 14 and hypothetical Well No. 15 near the location of observation well 09-2. Observation well 09-2 is located approximately 300 feet north of Well No. 14. This analysis is based on each well pumping at 1700 gpm or approximately 2.45 MGD for a total of 4.9 MGD well-field production. As can be seen from Table 1, the remaining available drawdown exceeds 30 percent at each well location. Our standard practice is to allow about 30 percent of the total available drawdown for loss of well efficiency over

time and for periods of low precipitation when surface water and aquifer water levels may be unusually low. River stage and flow data from the U.S. Geological Survey for the gage on the White River at Noblesville indicates that river stage could be as much as approximately five feet lower than it was when the pumping test of Well No. 14 was performed.

Table 2 shows the same analysis with both wells pumping 1800 gpm for a total withdrawal of 5.2 MGD. In this scenario, the remaining available drawdown at Well No. 15 is a little less than 30 percent. Clearly this analysis indicates that a capacity of 5 MGD can safely be developed with the installation of a second well near the location of observation well 09-2. If the well performance at this location is better than the well performance of Well No. 14, the well-field capacity could be slightly higher.

Figure 2 shows well performance curves for Well No. 14. At a pumping rate of 1700 gpm, Curve E shows that, allowing for interference from a second well and seasonal low water levels, there would be about 9.5 feet of remaining available drawdown at steady state conditions.

This analysis indicates that production of 5 MGD from two wells at the locations shown on Figure 1 is feasible and we believe that recharge in the area should be sufficient to support withdrawals of this magnitude. The distance drawdown data indicates that if aquifer conditions are suitable 100 feet north and 100 feet south of observation well 09-2, it might be possible to operate three wells spaced 200 feet apart at 1400 gpm each for a total withdrawal of 6.0 MGD. Additional test drilling would be necessary to confirm the nature of the aquifer materials at potential production well locations.

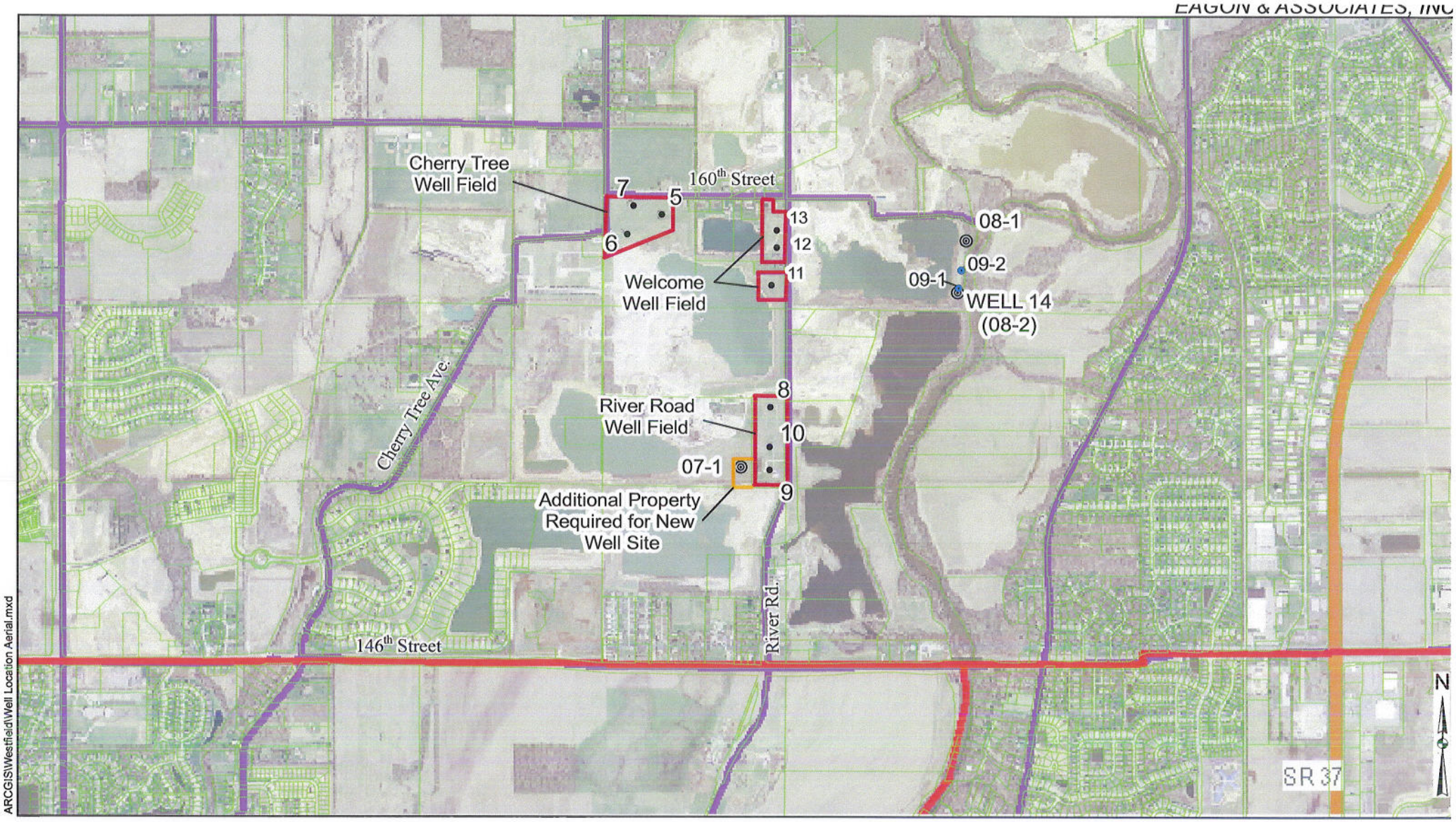
Data from the transducers at the River Road, Welcome, and Cherry Tree Well Fields did not show any response to pumping at the DeWitt property. It is important that some significant portion of the gravel pit be maintained on the DeWitt property. Water in the gravel pit acts as a source of recharge to support pumping withdrawals, and buffers interference drawdown between well fields. If the gravel pit is filled in, the water in storage is lost and recharge is decreased due to the placement of lower permeable materials as fill in the gravel pit.

We will provide you with a complete report documenting the aquifer test results, analyses, and conclusions. Please call if you want to discuss this information.

Sincerely,

Stephen J. Champa, PG
Hydrogeologist
Indiana License No. 2247

SJC/kj
encl.
cc: Jim Logan, w/encl.



Legend

- TestWells
- Existing Well

0 2,000 4,000
Scale in Feet

Figure 1. Well Location Map

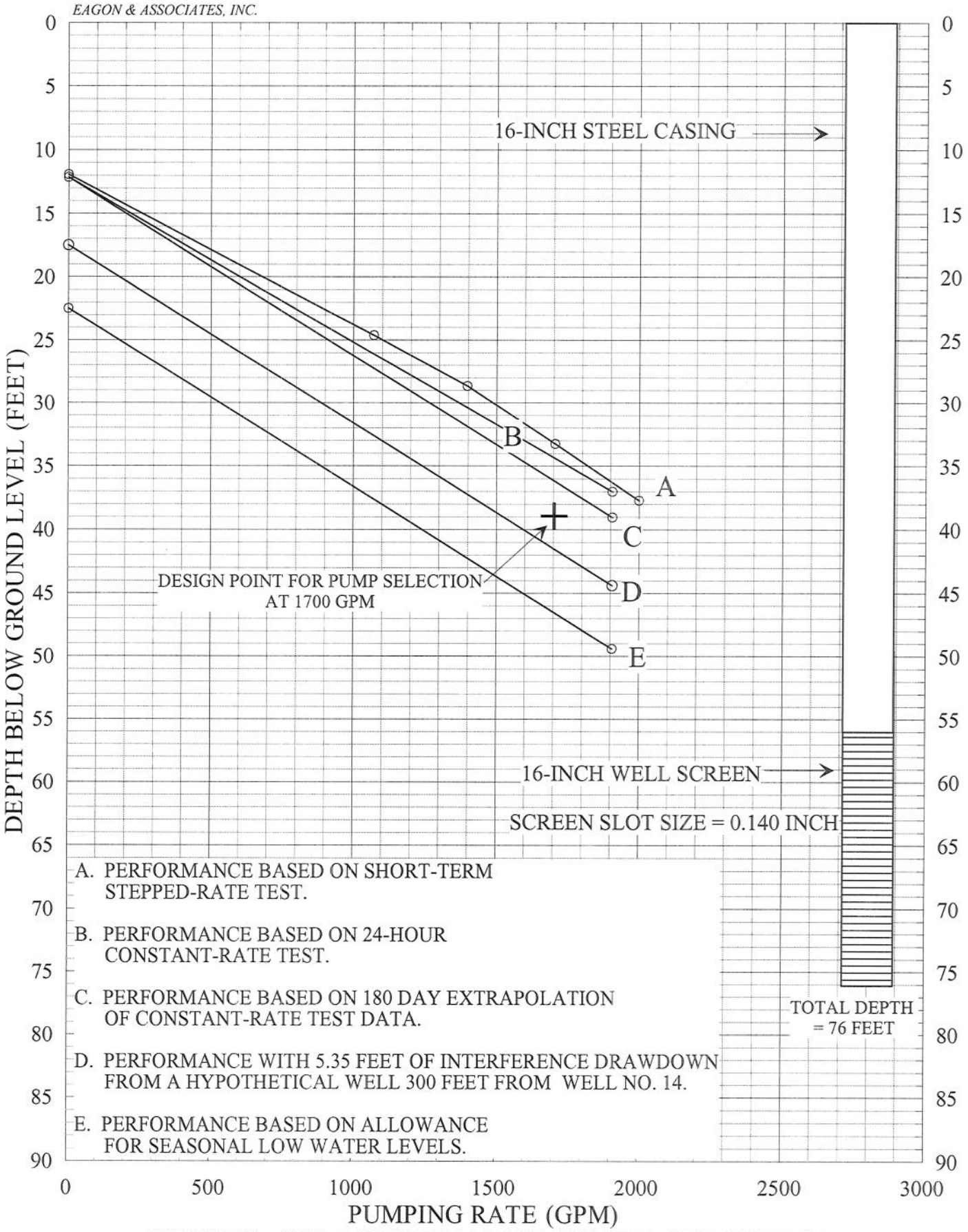


FIGURE 2. WELL PERFORMANCE CURVES FOR WELL 14
WESTFIELD, INDIANA

WESTFIELD/WELL14/14PERF.GRF
5/26/09

TABLE 1
WELL-FIELD CAPACITY EVALUATION
WESTFIELD PUBLIC WORKS - DEWITT PROPERTY

	WELL NO. 14	WELL NO. 15
Well Depth (feet, bgl)	76	78
Static Water Level (feet, bgl)	12.13	16.88
Depth to Top of Well Screen (feet, bgl)	56	58
Available Drawdown (feet)	43.87	41.12
Pumping Well Drawdown at 1700 GPM (feet)	22.12	22.12
Interference Drawdown (feet)	4.77	4.77
Pumping Level with Interference (feet, bgl)	39.02	43.77
Total Drawdown (feet)	26.89	26.89
Remaining Available Drawdown (feet)	16.98	14.23
Percentage of Available Drawdown Remaining	38.7%	34.6%

Notes:

Data for weLL No. 14 are based on construction details and data collected during the May 2009 pumping tests.

Data regarding well depth and screen length for Well No. 15 are based on the boring log and grain-size data from Test Boring 09-2.

Interference drawdown and pumping well drawdown are based on 180 day extrapolations of the data from the 48-hour aquifer test of Well No. 14.

Assumptions:

Specific capacity of Well No. 15 will be similar to Well No. 14.

Well loss is the same at Well No. 15 as it is at Well No. 14.

TABLE 2
WELL-FIELD CAPACITY EVALUATION
WESTFIELD PUBLIC WORKS - DEWITT PROPERTY

	WELL NO. 14	WELL NO. 15
Well Depth (feet, bgl)	76	78
Static Water Level (feet, bgl)	12.13	16.88
Depth to Top of Well Screen (feet, bgl)	56	58
Available Drawdown (feet)	43.87	41.12
Pumping Well Drawdown at 1800 GPM (feet)	24.42	24.42
Interference Drawdown (feet)	5.05	5.05
Pumping Level with Interference (feet, bgl)	41.60	46.35
Total Drawdown (feet)	29.47	29.47
Remaining Available Drawdown (feet)	14.40	11.65
Percentage of Available Drawdown Remaining	32.8%	28.3%

Notes:

Data for weLL No. 14 are based on construction details and data collected during the May 2009 pumping tests.

Data regarding well depth and screen length for Well No. 15 are based on the boring log and grain-size data from Test Boring 09-2.

Interference drawdown and pumping well drawdown are based on 180 day extrapolations of the data from the 48-hour aquifer test of Well No. 14.

Assumptions:

Specific capacity of Well No. 15 will be similar to Well No. 14.

Well loss is the same at Well No. 15 as it is at Well No. 14.