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City of Turlock
Water Systems Model Update and
Distribution System Planning

WATER MASTER PLAN UPDATE

FINAL
May 2009

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City of Turlock

Water Systems Model Update and Distribution System Planning

WATER MASTER PLAN UPDATE

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INTRODUCTION

A previous Water System Master Plan was completed by Carollo Engineers P.C. (Carollo) in August 2003. This document is intended to update information provided in that master plan as follows:

- Surface Water Source. This update will examine the impacts of using the planned new surface water source to be provided by Turlock Irrigation District (TID). This report will examine the system hydraulics, emergency operation, and fire flow capacity using the new surface water source.
- Land Use Inventory and System Demand Update. This report will examine the City of Turlock's (City) current and planned land use, and provide estimates of existing and future water demands.
- Water Model Update. This report will also document the update of the City's water model performed on this project, detailing calibration efforts and model results.
- Planning Level Cost Estimate. A planning level cost estimate of the transmission main, terminal reservoir and pump station, pressure regulating valve (PRV) at connection points, and any additional wells needed will be provided in this report.

1.1 SYSTEM DESCRIPTION

The system currently relies on groundwater wells to supply water to its customers. The water is pumped directly into the distribution system and is not stored in any reservoirs. The City has elected to participate in the planning and design phases for a proposed new surface water supply to be developed by TID. The new TID project would supply wholesale water from a new surface water plant which would be built and operated by TID. This water treatment plant (WTP) will also serve the cities of Hughson, Modesto, and Ceres. The new surface water would supplement the City's current groundwater sources. The City also operates 2 booster pump stations (completion July 2009) with reservoirs for use during peak demand and fire flow protection.

1.1.1 Groundwater Wells

The City has 23 operational wells. Table 1.1 presents a summary of the City's wells and their capacity. Well Nos. 3, 4, 8, 10, 13, 14, 15, 19, 20, 21, 22, 24, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38 and 39 are currently in operation and connected to the distribution system. Well Nos. 1, 2, 5, 6, 7, 9, 11, 12, 16, 17, 18, 23, 25, and 26 have been closed due to casing or pump failures, high sand production, contamination, or lack of necessity. Contamination sources have included PCE, arsenic, nitrates, manganese, carbon tetrachloride, and hydrogen sulfide. A new well (Well No. 40) is currently under construction, and will be completed in 2010.

Table 1.1 Well Summary
Water Master Plan Update
City of Turlock

Well No.	Status	Location	Casing Depth (ft)	Average Yield (gpm)	Motor Size (hp)	Auxiliary Diesel Generator	Discharge Size (in)	Date Completed
1	Closed 1988 (Mechanical Failure)	Fire Station #1	199	N/A	30	---	6	Mar-1937
2	Closed 2002 (Sand)	Commerce Way	291	1,000	100	Yes	10	May-1971
3	Operational	Geer Rd & Regis St	365	1,200	100	Yes	10	Oct-1972
4	Operational	Tully Rd & Branding Iron Dr	340	900	100	---	10	Jul-1976
5	Closed 1992 (PCE & Nitrates)	D Street & Fifth St	244	N/A	50	---	8	Feb-1974
6	Closed 1990 (Mechanical Failure)	Canal Dr & Wolfe Ave	290	N/A	75	---	8	May-1959
7	Closed 1988 (Mechanical Failure)	Berkeley Ave & Canal Dr		N/A	50	---	8	1941
8	Operational	Palmer Dr & Greywolf Cir	420	1,100	100	---	10	Mar-1978
9	Closed 1999 (Carbon Tetrachloride)	First St & Orchard St	212	N/A	60	---	8	Jul-1950
10	Operational	Sycamore St & Colorado Ave	302	600	50	---	10	1947
11	Closed 1988 (Mechanical Failure)	Castor St & Spruce St	330	N/A	50	---	8	1947
12	Closed 1981 (Mechanical Failure)	West Ave South & South Ave	222	N/A	75	---	8	1948
13	Operational	Canal Dr & East Main St	432	1,000	100	---	10	Mar-1950
14	Operational	Johnson Rd & Canal Dr	285	650	50	Yes	8	Dec-1955
15	Operational	International Paper	424	1,400	100	Yes	10	Aug-1956
16	Closed 2000 (PCE & Nitrates)	Fifth St & F St	480	1,000	150	---	10	Nov-1959
17	Closed 1999 (Nitrates)	Marshall St & Corello St	406	1,100	75	---	8	Dec-1960
18	Closed 1995 (Mechanical Failure)	N Olive Ave & Minnesota Ave	407	N/A	60	---	8	Jan-1961
19	Operational	Donnelly Park	442	1,100	100	Yes	10	Sep-1965

Table 1.1 Well Summary
Water Master Plan Update
City of Turlock

Well No.	Status	Location	Casing Depth (ft)	Average Yield (gpm)	Motor Size (hp)	Auxiliary Diesel Generator	Discharge Size (in)	Date Completed
20	Operational	Monte Vista Ave & Crowell Rd	360	1,000	100	---	10	Mar-1978
21	Operational (irrigation only)	Pedretti Park		400		---	6	Jan-1978
21-P	Closed 2002	Pedretti Park	250	25	1.5	---	1.5	Feb-1994
22	Operational	Linwood Ave & Lander Ave	310	1,400	150	---	10	Dec-1980
23	Closed 1994 (Sand)	Craig Ct	355	N/A	125	---	8	Oct-1986
24	Operational	Quincy Rd & Sebastian Dr	400	1,950	150	Yes	12	Apr-1988
25	Closed 1990 (Nitrates)	West Ave South & Montana Ave	220	N/A	100	---	10	May-1988
26	Closed (Mn & H_2S)	Spengler Way	370	N/A	125	---	12	Jul-1992
27	Operational	Zeering Rd & Fosberg Rd	400	2,300	200	Yes(1)	12	Jul-1992
28	Operational	Tuolumne Rd & Tully Rd	405	1,400	175	---	12	Jul-1993
29	Operational	Hawkeye Ave & N Palm St	472	1,500	125	---	12	May-1996
30	Operational	Orange St & Bernell Ave	420	1,600	175	---	12	Aug-1994
31	Operational	University Jr. High	450	1,600	150	---	12	May-1995
32	Operational	Berkeley Ave & Alex Ct	525	2,000	200	---	12	Jul-1995
33	Operational	Sunnyside Park	450	2,100	200	Yes(1)	12	Oct-95
34	Operational	Dianne Pond	430	1,300	125	---	N/A	Jan-2000
35	Operational	Monte Vista West of Hwy 99	495	2,800	250	Yes	N/A	Mar-2003
36	Operational	Canal Dr & Soderquist Rd	580	2,000	200	---	N/A	Mar-2003
37	Operational	Crowell Rd & Taylor Rd	N/A	3,000	350	---	12	Jan-2003
38	Operational	Zeering Rd & Mountain View Rd	615	3,000	350	---	12	Jan-2003
39	Operational	Christoffersen Pkwy and Wellington Lane	375	800	N/A		N/A	Jul-2007

Note:
(1) Generators are currently not permitted by the San Joaquin Valley Air District
gpm = gallons per minute.

A brief summary of pumping levels and well capacity of operational wells is presented in Table 1.2.

Table 1.2 Operational Well Information
Water Master Plan Update
City of Turlock

Well #	Well Capacity[1] (gpm)	Max Pumping Level[2] (ft)	Depth to Starting Perforation (ft)	Depth to Top of Pump Bowl (ft)	Casing Depth (ft)
3	900	121	156	180	365
4	900	N/A	158	210	340
8	1,000	150	350	210	420
10	500	133	N/A	180	302
13	1,000	N/A	132	180	432
14	500	105	140	160	285
15	1,300	N/A	180	195	424
19	800	107	172	160	442
20	800	168	160	210	360
22	1,100	N/A	150	200	310
24	1,700	128	140	216	400
27	1,700	N/A	130	165	400
28	1,400	153	205	200	405
29	1,300	169	204	180	472
30	1,400	176	215	195	420
31	1,700	186	200	185	450
32	1,600	195	195	190	525
33	2,100	160	150	180	450
34	1,100	195	305	230	430
35	3,000	146	205	200	495
36	2,100	139	290	250	580
37	2,900	200	285	270	590
38	2,900	170	285	270	615
39	800	187	250	270	375
Total Capacity	34,500				

Notes:
(1) Well capacity observed during operation on July 10, 2008.
(2) Water surface elevation measured during well operation. Data from 1998 - 2002.
N/A - Data Not Available.

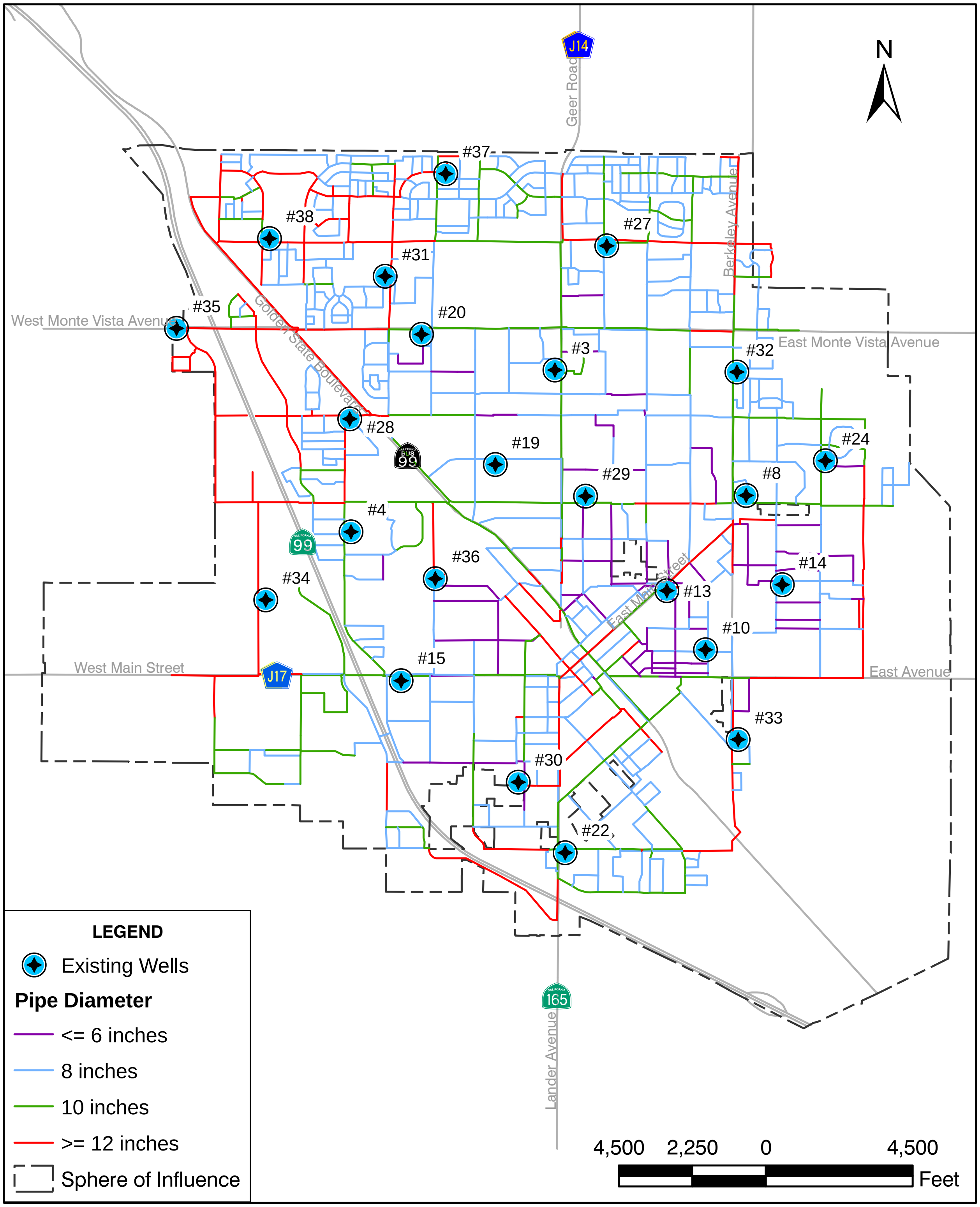
Figure 1.1 shows the location of each of the operational wells, and shows the location and size of existing pipes. The water system is primarily composed of 8-inch pipelines with the majority less than 12 inches in diameter. This is typical of a system supplied by groundwater sources distributed throughout the system. As identified in the 2003 Master Plan Update, the existing water system is deficient to meet future demands. The City plans to supplement the current groundwater supply system with surface water from TID and additional wells.

1.1.2 Reservoirs and Pump Stations

Two reservoirs and pump stations are currently under construction. Each reservoir has a storage capacity of 1 million gallons (MG). The pump stations at each reservoir site have a firm capacity of 5,400 gallons per minute (gpm). Each station is equipped with a standby pump and engine driven generator. The reservoirs and pump stations are intended to supplement peak hour and/or fire flow supply capacity. The reservoirs are located at Third and D Street and on Kilroy Road.

1.1.3 Planned Surface Water System

TID is currently designing a new WTP that will utilize water from the Tuolumne River. The City has partnered with TID and other utilities to design this new surface WTP. A decision on the City's participation in construction of the plant is expected in early 2009. Water from the new plant, currently up to 15 million gallons per day (mgd), will be delivered by a transmission main to a City-owned and operated terminal reservoir and pump station on property in northeastern Turlock. Based on conversations with TID, finished water will be delivered to the terminal reservoir at a pressure of approximately 20 pounds per square inch (psi). From there the surface water will be boosted into the City’s water system to supplement the groundwater supply.



Turlock CITY OF INC. 1908

carollo Engineers...Working Wonders With Water

Figure 1.1
EXISTING WATER SYSTEM
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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Nov 20, 2007

PLANNING AND ENGINEERING CRITERIA

Planning and engineering criteria from the 2003 Water Master Plan Update were used in this study. Criteria defined for the project include those for system design, storage and supply, and cost estimating. These criteria are presented in the following sections.

2.1 SYSTEM DESIGN CRITERIA

System design criteria were established to evaluate the capacity and hydraulic condition of the City of Turlock's (City) distribution system. The design criteria, as shown in Table 2.1, were established using information from the 2003 Water System Master Plan Update.

Table 2.1 System Design Criteria
Water Master Plan Update
City of Turlock

Condition	Criteria
Supply Capacity	
Reliable	Total Installed Capacity minus 6,000 gpm(1)
Standby	6,000 gpm
Minimum Pressure	
Maximum Day	35 psi
Peak Hour	30 psi
Maximum Day Plus Fire Flow	20 psi
Emergency Scenario	20 psi
Maximum Pressure at Service Connection	70 psi
Distribution Pipeline Velocity	
Peak Hour	8 fps(2)
Maximum Day Plus Fire Flow	12 fps(2)
Distribution Pipeline Headloss	
Pipeline Diameter < 16 inches	10 feet per 1,000 feet
Pipeline Diameter > 16 inches	3 feet per 1,000 feet

Notes:
(1) Reliable capacity is sufficient to meet all demands with 6,000 gpm well capacity off-line.
(2) fps - feet per second.

System pressure criteria were compared with hydraulically modeled pressures under various scenarios to determine if all portions of the system are within the acceptable range. Lower pressures adversely affect customers and irrigation systems, while higher pressures require pressure-reducing valves at service connections to avoid damage to customer piping and appliances.

Velocity and headloss criteria were also used in the hydraulic modeling process. High velocity or headloss in the distribution system can serve as a warning that a pipe is nearing the limit of its carrying capacity. However, upgrades due solely to headloss are not recommended unless maximum velocity and pressure conditions are simultaneously outside of the acceptable ranges.

2.2 SYSTEM SUPPLY/STORAGE CRITERIA

A distribution system must be able to provide for three types of uses: operational, fire and emergency. In the case of a groundwater system, the source of supply for these uses is a groundwater well drawing from a potable water aquifer. Alternatively, a combination of wells and reservoir storage could be used to provide water for these system uses.

Operational use is the amount of storage/supply used to meet demands in excess of the maximum day demand rate. Fire storage/supply is the volume of storage or well supply capacity held in reserve for a set number of fire events. In this study, 18 different fire flow locations were individually evaluated. Operational and fire needs above maximum day demand would typically be met in this type of system either with additional wells or with storage reservoirs such as the two under construction.

Emergency storage/supply is the supply capacity held in reserve at all times to meet water demands in the event of a supply failure. Because the system comprises many point sources of supply (i.e., wells), aboveground storage (tanks) was not considered to be the sole source for emergency storage. Rather, the reliable capacity of both the system's wells and the new reservoirs under construction comprise the basis for evaluating emergency water supply.

Water system supply criteria, defined in Table 2.2, were used for evaluating the system capacity.

Table 2.2 Water Supply Criteria
Water Master Plan Update
City of Turlock

Condition	Recommended[1]
Operational Supply	Peak Hour + Fire Flow Demand
Fire Supply[2]	4,000 gpm - Industrial 3,000 gpm - Commercial 1,500 gpm - Residential
Emergency Supply	50 Percent of Maximum Day Demand

Notes:
(1) Supply met with reliable well capacity.
(2) Based on 2003 Master Plan Update fire flow requirements.

LAND USE INVENTORY AND WATER USE PROJECTIONS

Land use inventory and projections are used to distribute existing water demands and to project future demands. The methodology and assumptions made in their development for each planning period will be described below. Land use data were provided by the City of Turlock (City) Planning Department for existing (2006) and build-out conditions (2020) at the parcel level.

In general, the land use designations are similar to those reported in the 2003 Master Plan. Land use was divided into 6 major categories: agriculture, commercial, industrial, low and medium density residential, high-density residential, and open space. Table 3.1 presents the current and build-out land use for the City.

Table 3.1 Land Use Projections
Water Master Plan Update
City of Turlock

Land Use Category(1,2)	From 2003 Master Plan Update	Updated in This Study	
	2017 Area (Acres)	2006 Area (Acres)	*2020 Area (Acres)*
Commercial	2,137	1,713	2,807
Industrial	1,646	505	1,402
LDR and MDR	5,344	3,219	5,174
HDR	246	342	258
Open Space	375	265	332
Total	**9,748**	**6,044**	**9,973**

Notes:
(1) Vacant and unknown land use categories were not counted in present and future land use.
(2) Agriculture areas are not shown and are assumed to have a zero water demand.
LDR - Low Density Residential.
MDR - Medium Density Residential.
HDR - High Density Residential.

3.1 EXISTING LAND USE (2006)

Figure 3.1 illustrates the existing land use distribution for 2006. The City’s industrial areas are concentrated in the southwest and southeast quadrants with commercial areas scattered throughout the City along main roads. Agricultural areas are located along the City’s boundary. Residential areas comprise most of the remaining space with few housing developments west of State Route 99.

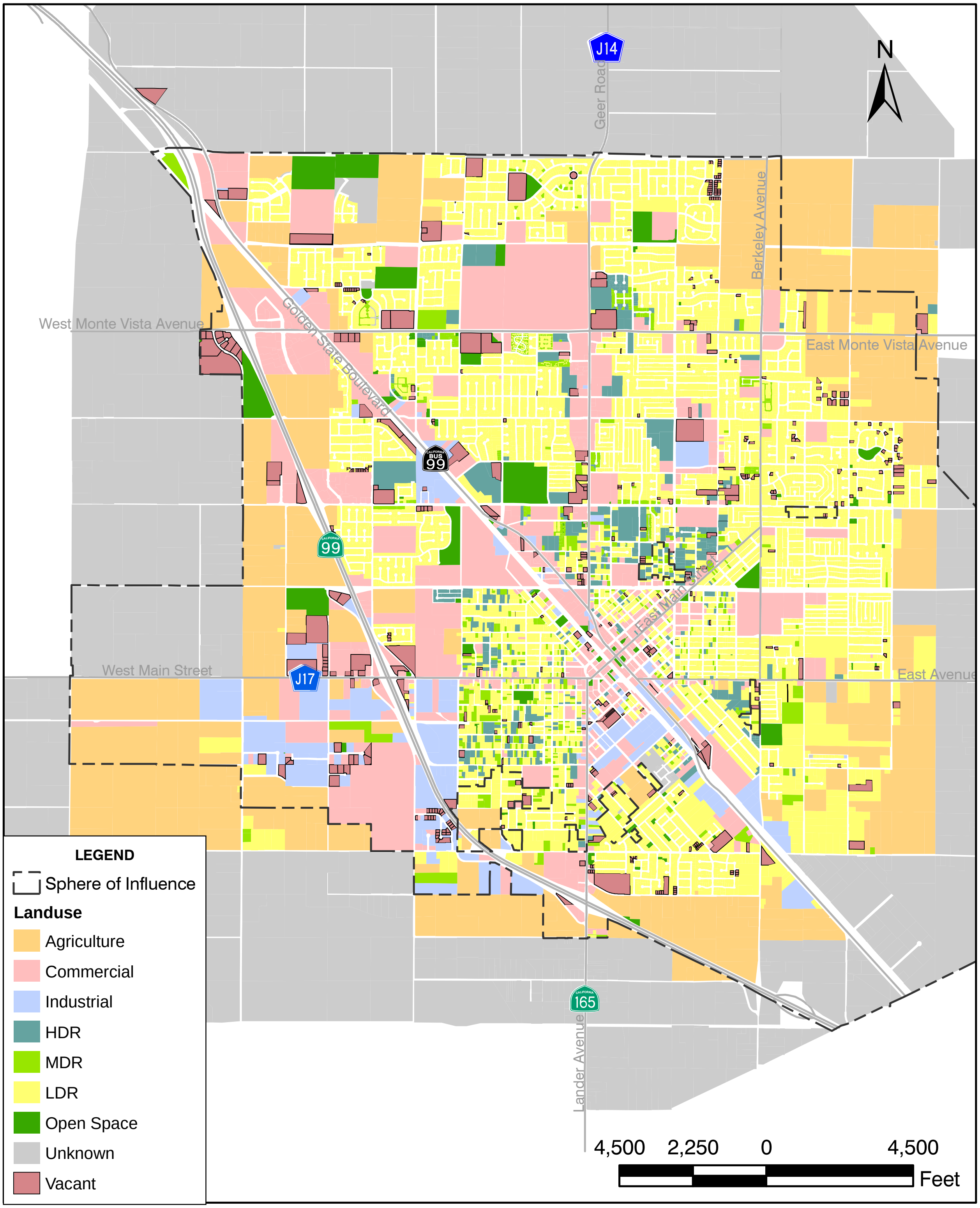

Figure 3.1
EXISTING LANDUSE (2006)
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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Oct 4, 2007

3.2 LAND USE

The City's Planning Department and General Plan anticipate build-out to occur in 2020. At build-out, the City's boundary is expected to be fully expanded to the secondary sphere of influence, which results in a 50 percent increase in total area. Residential development is expected to occur in the north, northeast, and southeast quadrants of the City. Industrial development is expected in the southwest quadrant. Figure 3.2 illustrates the land use distribution at build-out. Both industrial and residential land use categories experience significant increases in area. Commercial area increases slightly and agricultural area decreases slightly. All other categories see minimal change. It can also be seen that there was an increase in total land use from the previous master plan, particularly in commercial and high-density residential land use. There was a slight decrease in industrial land use from what was predicted in the 2003 Master Plan.

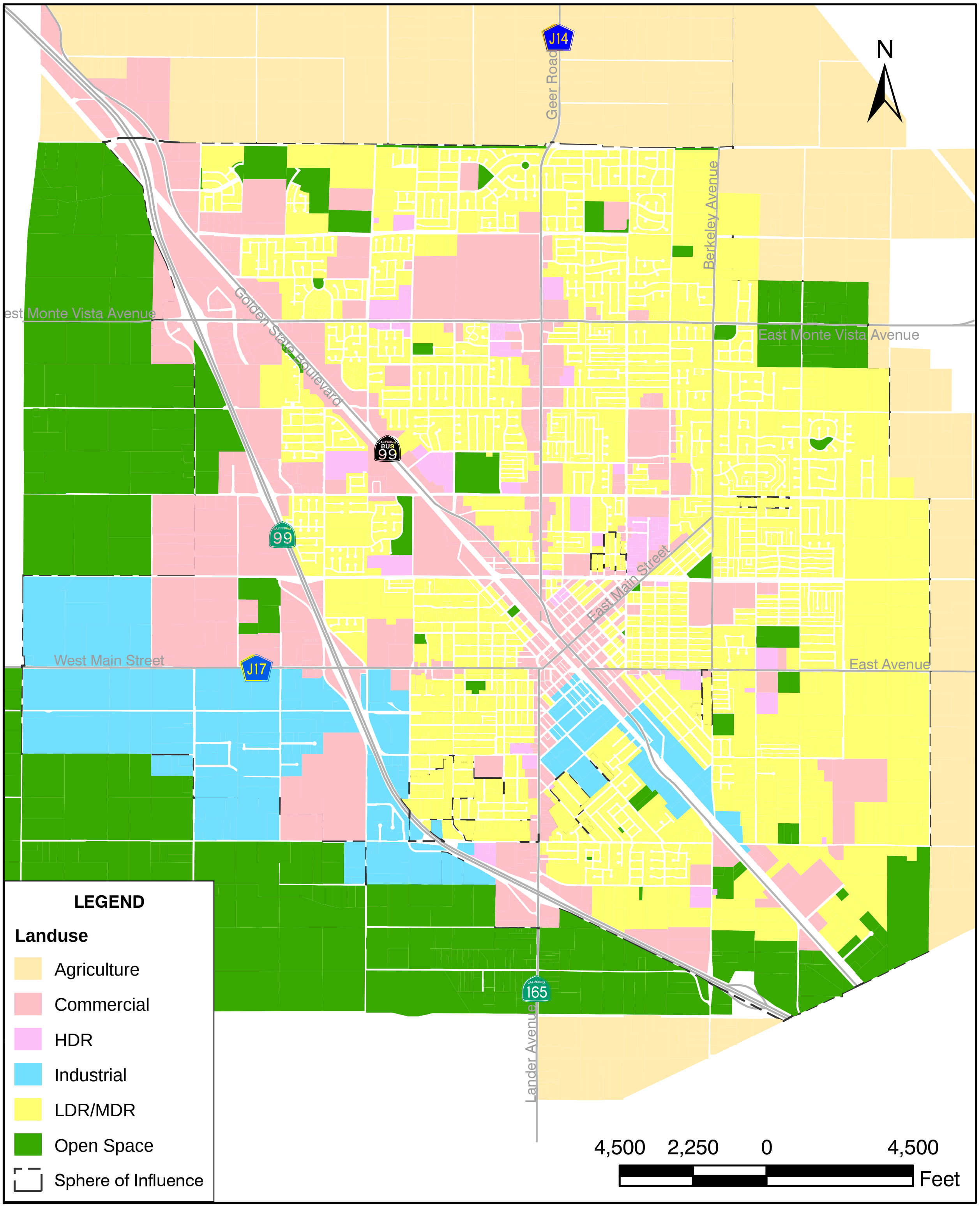

Figure 3.2
BUILD-OUT LANDUSE (2020)
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

WATER MODEL UPDATE

The role of a hydraulic model is to identify hydraulic deficiencies in the City of Turlock's (City) distribution system and simulate the effect of future growth and the new surface water source. This section will discuss the development and calibration of the hydraulic model and will identify system deficiencies under existing and future demand and supply conditions.

4.1 WATER DEMAND UPDATE

Using the existing land use, the water demand was updated for the model. The 2006 water demand coefficients were calculated from water demands using the City's 2005 Urban Water Management Plan (UWMP). For commercial and residential, the build-out land use water demands were calculated based on an expected 15 percent reduction in the currently un-metered account demand due to water metering. This decrease in water demand is based on similar decreases seen by other California water utilities that have implemented water metering. It was assumed that open space and the currently metered commercial water demand would remain the same because the entire system is already metered.

Industrial sources are already entirely metered, but the City is expecting that most new industries will have a significantly lower water demand. Water demand coefficients were calculated assuming that existing land use water demand will remain the same (5.1 gallons per minute per acre [gpm/ac]), and that new industry would have a lower water demand coefficient of 1.9 gpm/ac. The water demand use coefficient for industrial would be 3.1 gpm/acre for this composite land use of both "wet" and "dry" industrial uses.

The estimated reduction in the 2020 demand described above was less than the 30 percent per capita decrease in the UWMP. For capital improvement planning a more conservative decrease is recommended. It is recommended that the City monitor water demands as water metering is further implemented to verify that the water demands are reduced as predicted.

Agricultural land is irrigated with a separate water source and therefore has a demand factor of 0 gpm/ac. Table 4.1 shows the recommended water demand used for the different land use types.

Using these water demand coefficients and a population of 67,876 in 2006 (provided by the City), it is estimated that the total average annual production (per capita) in 2006 was approximately 338 gallons per day capita (gpdc). This is a decrease in the average day annual production that was measured from the years 1996 through 2006. Table 4.2 lists total average daily production over the last 13 years. The population is expected to be 108,616 by 2020 (build-out). The total per capita water demand at build-out is estimated to be 287 gpdc based on the reductions in consumption due to metering as discussed previously.

Table 4.1 Water Demands for Land Use Type
Water Master Plan Update
City of Turlock

	Existing (2006)			Build-Out (2020)		
Land Use Type (units)	**Updated Existing Land Use (acres)(1)**	**Water Demand Coeff. (gpm/ac)**	**Average Day Demand (2006) (gpm)**	**Build-Out Land Use (acres)(1)**	**Water Demand Coeff.(2) (gpm/ac)**	**Average Day Demand (2020) (gpm)**
Commercial	1,713	1.1	1,884	2,807	1.1	3,087
Industrial(6)	505	5.1	2,576	1,402	3.1	4,346
LDR/MDR(3)(4)	3,219	2.6	8,369	5,174	2.3	11,900
HDR(5)	342	7.6	2,599	258	6.8	1,754
Open Space(7)	265	1.9	504	332	1.9	631
Total	**6,044**		**15,932**	**9,973**		**21,719**

Notes:
(1) Gross area.
(2) Expected demand reductions due to new residential metering and less water-intensive industrial development.
(3) LDR - Low Density Residential.
(4) MDR - Medium Density Residential.
(5) HDR - High Density Residential.
(6) Industrial demand coefficient is a composite of “wet” industry at 5.1 gpm/acre for 505 acres (existing), and “dry” industry at 1.9 gpm/acre for 897 acres.
(7) Open space is currently metered, no reduction in unit demand is expected.

Table 4.2 Historical Water System Per Capita Daily Demands
Water Master Plan Update
City of Turlock

Year	Total Average Day Annual Production (gpdc)
1996	350
1997	378
1998	335
1999	365
2000	377
2001	367
2002	365
2003	365
2004	347
2005	347
2006	338
2007	330
2008	320
13-Year Average	**353**

Note:
(1) Based on population and flow data provided by the City.

4.1.1 Peaking Factors

Peaking factors represent the seasonal and daily demand water use variations, above or below the average annual water demand. Peaking factors developed in the 2003 Water Master Plan were used in this hydraulic analysis. The maximum day and average day peaking factors and total system projected water demands are summarized in Table 4.3. An analysis of recent demand data from the City verified that these peaking factors are still valid.

Table 4.3 Updated Demands and Peaking Factors
Water Master Plan Update
City of Turlock

	Average Day Demand	Maximum Day Demand	Peak Hour Demand
Existing (2006)	15,932 gpm	26,287 gpm	34,253 gpm
Build-Out (2020)	21,719 gpm	35,836 gpm	46,695 gpm
Peaking Factor	-	1.65	2.15

Notes:
(1) Peak hour peaking factor = peak hour demand/average day demand.
(2) Maximum day peaking factor = max day demand/average day demand.

4.1.1.1 Maximum Day Demand

The maximum day demand (MDD) is the highest water demand during a 24-hour period of the year. The MDD peaking factor is expressed as a multiplier applied to the average day demand (ADD). Water system supplies are typically sized to meet the anticipated MMDs of a water system.

Maximum Day Demand = 1.65 x Average Day Demand

4.1.1.2 Peak Hour Demand

The peak hour demand (PHD) is the highest water demand during any one-hour period of the year. The PHD is expressed as a multiplier applied to the average annual demand. Peak hour demands simulate high water use throughout the system during peak demands and identifies areas of the distribution system that may experience low pressures.

Peak Hour Demand = 2.15 x Average Day Demand

4.2 HYDRAULIC MODEL UPDATE

The distribution model was developed using WaterCAD by Heastad Methods (Version 7). City staff constructed the original hydraulic model. Carollo subsequently updated and calibrated the model, initially for the 2003 Water Master Plan Update. The following information was added to the model in this update (since the 2003 Master Plan Update):

- New pipes over 8 inches.
- Well Nos. 37, 38 and 39 pump curves, and pump control data.
- Current land use data and expected demands.
- Updated MDDs based on Supervisory Control and Automated Data Acquisition (SCADA) data.

4.2.1 Model Calibration

The model was calibrated using well production data from the City's SCADA system and residual and static pressure from hydrant tests performed by the City's fire department. static pressure calibration was performed using the SCADA data while residual pressure calibration used the hydrant test data. The use of hydrant test data results in a better calibrated model since the system can be calibrated under "stressed" conditions. The hydrant testing procedure involves measuring the pressure drop and flow at a hydrant when it is opened. Table 4.4 summarizes the static and residual calibrations.

The calibration process resulted in a hydraulic model that should be considered representative of existing conditions. Nine out of the eleven hydrant locations modeled have a static pressure differential (difference between modeled and actual test pressure) of less than 5 percent. In addition, eight of the hydrant locations have a residual differential of less than 10 percent; this is within accepted industry standards for water modeling and demonstrates that the model is an accurate representation of the water system.

4.3 RESULTS OF HYDRAULIC ANALYSIS

The hydraulic network model was used to evaluate if the existing distribution system was adequate to meet the pressure, headloss, and velocity criteria presented in Section 3. Components that did not meet the criteria were noted as deficiencies. Deficiencies include:

- Extremely low or high pressures at nodes.
- High velocity or high headloss in pipelines.

The hydraulic network model was run under the 2006 conditions to evaluate the distribution system performance under the following scenarios:

- Maximum day demand (MDD).
- Peak hour demand (PHD).
- MDD plus fire flow.

Table 4.4 Hydraulic Model Calibration
Water Master Plan Update
City of Turlock

Hydrant No.	Cross Street	Date of Test	Junction Modeled	Static Pressure: Measured Pressure (psi)	Static Pressure: Modeled Pressure (psi)	Static Pressure: Pressure Difference (psi)	Static Pressure: Percent Error	Residual Pressure: Measured Discharge (gpm)	Residual Pressure: Measured Pressure (psi)	Residual Pressure: Modeled Pressure (psi)	Residual Pressure: Pressure Difference (psi)	Residual Pressure: Percent Error
259	Trade Way at Commerce Way	1/27/2006	J252	64	65.2	1.2	1.9%	1,087	54	52.1	-1.9	3.5%
258	Geer at Christofferson	1/27/2006	J11	54	54.9	0.9	1.6%	1,007	48	46.5	-1.5	3.2%
257	Billmore and Vasoncellos	1/20/2006	J924	58	58.4	0.4	0.6%	1,210	52	47.5	-4.6	8.7%
261	Crowell and Monte Vista	2/21/2006	J292	58	61.4	3.4	5.8%	1,163	52	51.0	-1.0	1.8%
260	Main and Broadway	3/15/2006	J1307	54	53.5	-0.5	0.9%	1,186	50	52.0	2.0	4.0%
266	3701 Mountain View	3/15/2007	J253	58	60.0	2.0	3.4%	1,021	50	49.5	-0.5	1.1%
292	Delbon and Olive	9/14/2006	J1373	58	61.5	3.5	6.0%	1,163	54	56.9	2.9	5.4%
271	Dauberberger at Zephyer Ct.	4/27/2006	J89	44	45.8	1.8	4.1%	978	38	36.5	-1.5	4.1%
290	Tully and Homer	8/30/2006	J1153	61	60.8	-0.2	0.4%	1,198	55	52.9	-2.1	3.9%
273	Marshall and Hamilton	4/28/2006	J1301	50	50.8	0.8	1.6%	1,061	46	41.7	-4.3	9.3%
262	500 South Center	3/6/2006	J543	56	56.9	0.9	1.6%	1,087	48	47.7	-0.3	0.6%

4.3.1 MDD Analysis

Results of the MDD are shown in Figure 4.1; two areas of the system were deficient. The northwest area of the City and the area near Well No. 30 had pressures that exceeded 70 pounds per square inch (psi). However, these pressures were within the margin of error of the hydraulic model. It is not recommended that any corrective actions be taken, because modeled pressures are within the margin of error (1 to 2 psi). It is recommended, however, that the City monitor pressures in that area to verify excessive pressures are not occurring

4.3.2 Peak Hour Demand

Results of the PHD are shown in Figure 4.2. These results do not include the operation of either new reservoir and pump station under construction (refer to Section 4.4.1). The criteria shown in Table 2.1 were exceeded for headloss criteria for 2 pipes. However, these pipes did not exceed the velocity criteria so no corrective action is recommended.

4.3.3 MDD Plus Fire Flow

In the event of a fire, the distribution system must be able to provide adequate fire flow wherever it is needed. To evaluate the systems response to a fire, 18 nodes were selected and individually tested using existing MDD plus fire flow. Fire nodes were selected based on surrounding land use and separated into three categories: residential, commercial, and industrial. A fire flow was then applied according to the land use nearby. The following fire flows were used:

- Residential Nodes: 1,500-gpm fire flow.
- Commercial Nodes: 3,000-gpm fire flow.
- Industrial Nodes: 4,000-gpm fire flow.

Figure 4.3 illustrates the location of the fire nodes. The following locations were identified in the modeling as deficient with respect to fire flow under existing MDD plus fire flow conditions:

- Node 252: Commerce and Trade Way - Did not meet 4,000 gpm while maintaining 20 psi.
- Node 277: S. Walnut at Venture Lane - Did not meet 3,000 gpm while maintaining 20 psi.

It was recommended in the 2003 Master Plan that additional capacity be installed in the southwestern area of the City to supply water to these areas during a fire flow event.

4.4 SYSTEM ANALYSIS SUMMARY

The results of the original hydraulic analysis can be seen in the 2003 Master Plan Update. Some of the deficiencies that were identified in the Master Plan Update were no longer

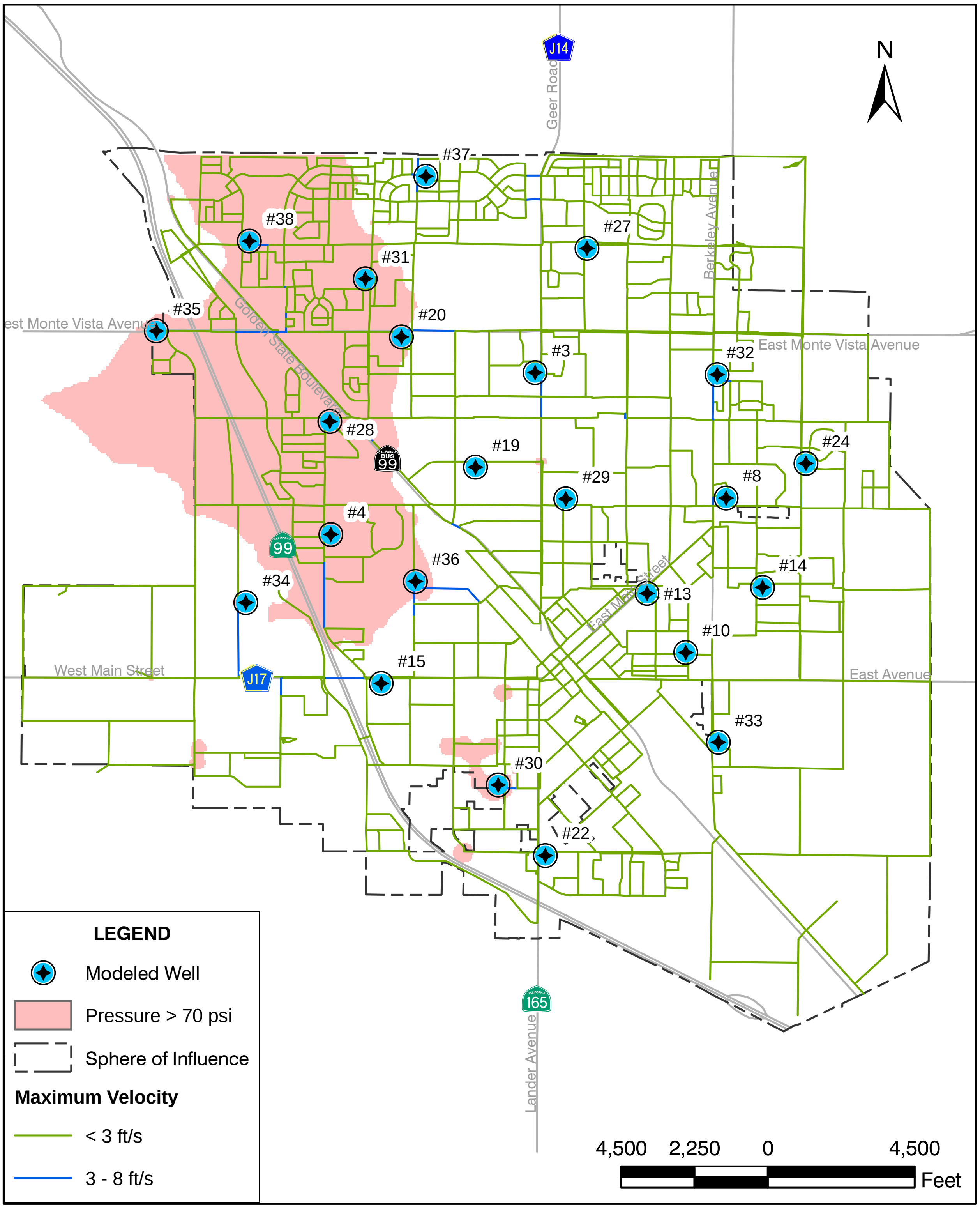

Figure 4.1
EXISTING MAXIMUM DAY DEMAND MODEL RESULTS
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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September 4, 2008

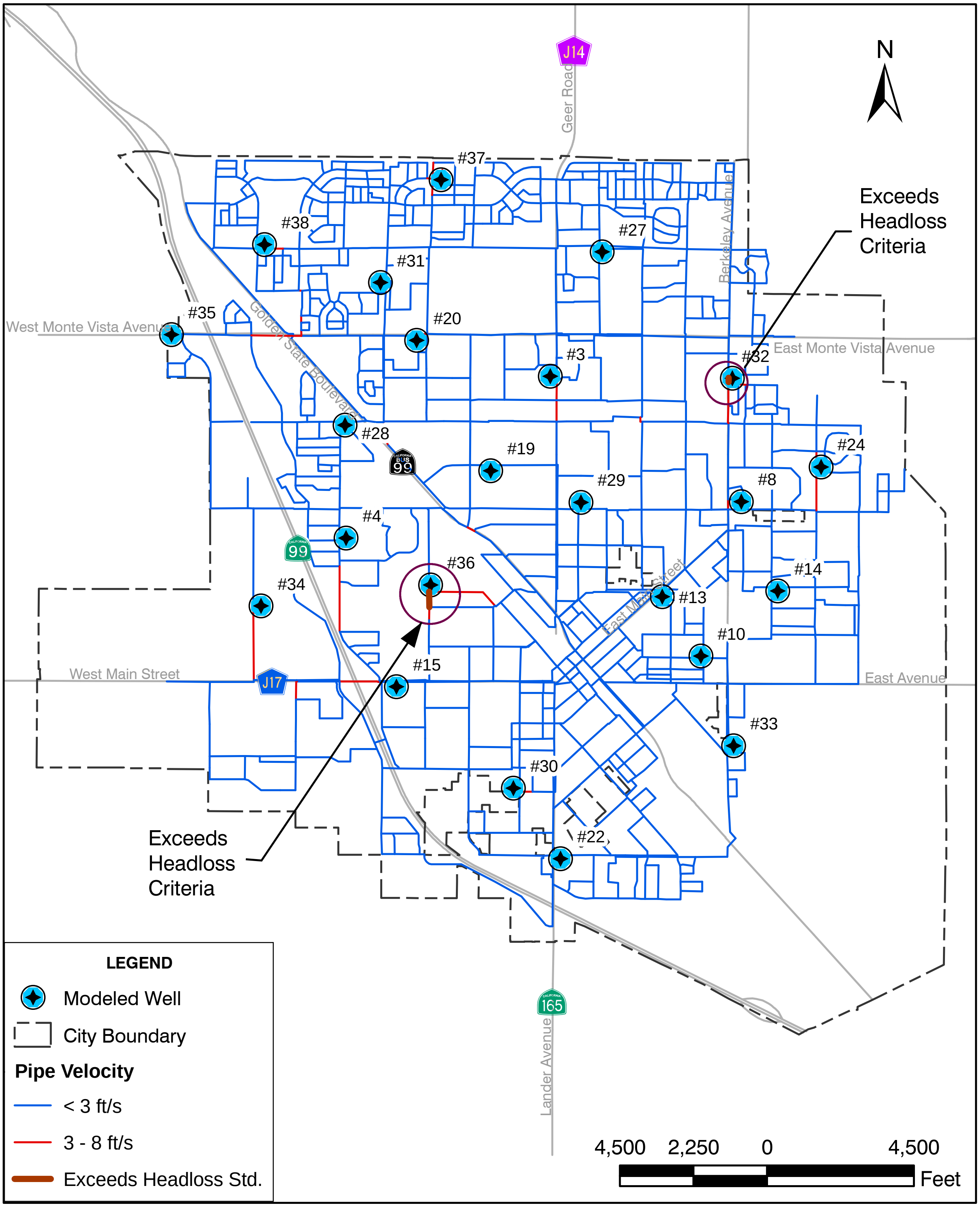

Figure 4.2
EXISTING PEAK HOUR MODEL RESULTS
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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September 4, 2008

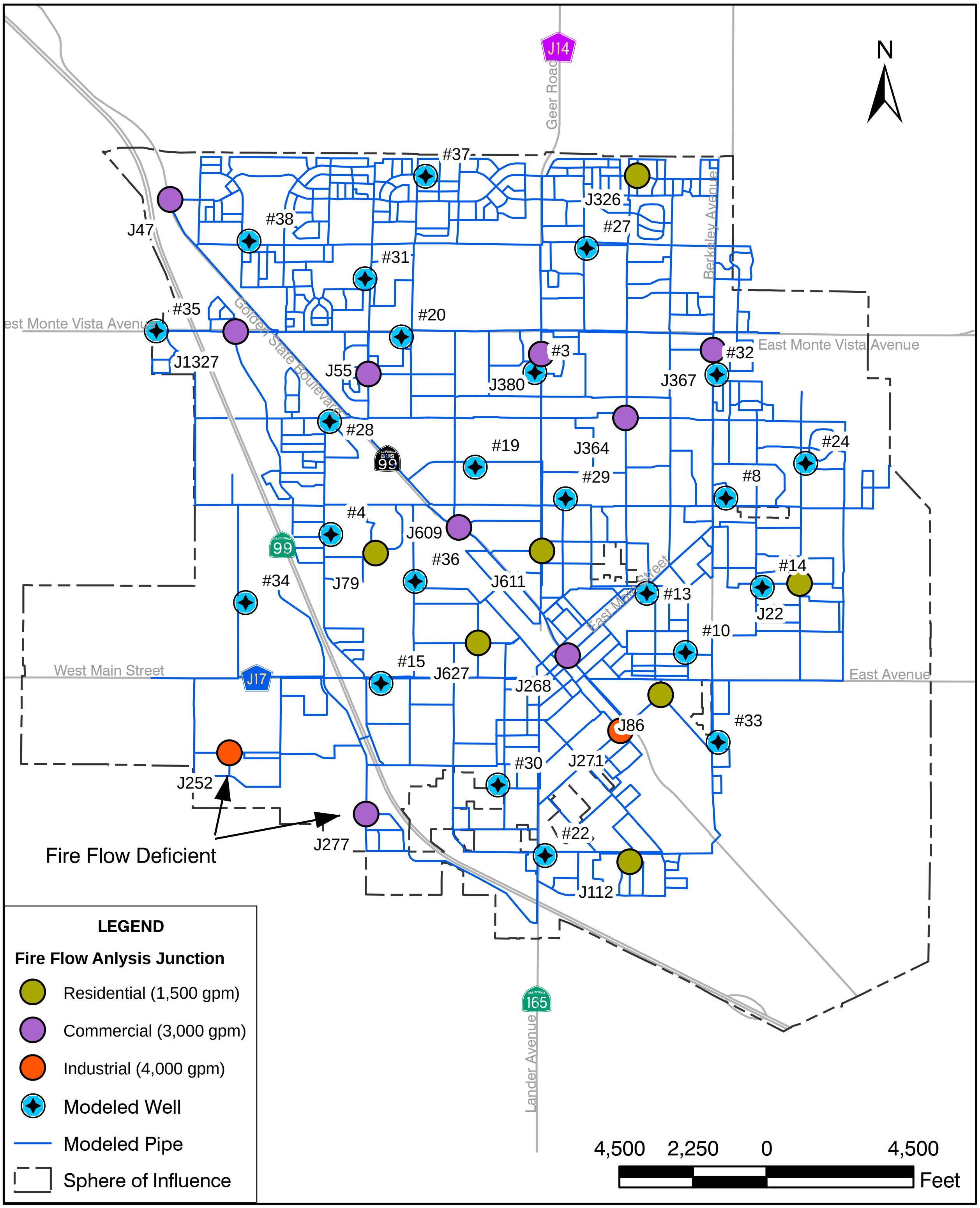

Figure 4.3
FIRE FLOW NODES
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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March 6, 2008

present during this analysis. Capital improvements such as the addition of two new wells and added pipes corrected previous deficiencies. Some deficiencies identified in the Master Plan Update remained. Table 4.5 has a summary of the remaining and remedied deficiencies.

Table 4.5 Identified Existing Deficiencies
Water Master Plan Update
City of Turlock

Deficiencies Identified in 2003 Master Plan Update	Status (2006)
High Pressure:	
Pressures exceeded 70 psi during MDD in northwest quadrant.	Deficiency still exists; however, no corrective actions are recommended. Monitoring is recommended.
High Velocity:	
One pipe exceeds 8 fps during peak hour.	Corrected: a new 12-inch pipeline mitigated high velocity.
Low Pressure:	
Five nodes fell below 30-psi minimum pressure during peak hour.	Corrected.
Fire Flow:	
Four areas were not able to supply the needed fire flows, two in the southwest quadrant, and two in the north eastern/western quadrants.	Northeast Quadrant: Corrected, due to the addition of Wells Nos. 37 and 38. Southwestern Quadrant: Deficiency still exists.

4.4.1 Modeling of Projected Demands and Near-Term Capacity Upgrades

The City of Turlock is currently building new finished water storage reservoirs and pump stations to improve fire flow and mitigate low pressures at peak hour. The reservoirs will be added at the following locations:

- The intersection of D-Street and 3rd Street (T-SE).
- Killroy Street near the Union Pacific railroad line (T-SW).

A new supply well (Well No. 39) has also been completed at the corner of East Christoffersen and Wellington Lane. The well has been online since 2007. This well was included in this future scenario in the hydraulic model.

A hydraulic analysis was performed on the water system with these near-term improvements to the system. A summary of the improvements shown in the hydraulic analysis is presented in Table 4.6.

Table 4.6 Existing Deficiencies with Near-Term Capacity Upgrades Water Master Plan Update City of Turlock	
Deficiencies Identified Existing Model	**Status (With Near-Term System Upgrades)**
High Pressure:	
Pressures exceeded 70 psi during MDD in northwest quadrant.	Deficiency still exists, however no corrective actions are recommended. Monitoring is recommended.
Fire Flow:	
Two areas in the southwest quadrant, were not able to supply the needed fire flows.	Corrected: Due to the addition of the T-SW pump station.

The addition of the Well No. 39, and the two pump stations and reservoirs to the system will be able to help mitigate low pressures during peak hour and eliminate the low pressures seen in the southwestern section of the City during peak hours. However, high pressures still are seen in the northwestern portion of the City during peak hours. This is due to the large wells that exist in that section of the City. However, the pressures seen were within the margin of error in the model, so no corrective actions are recommended. It is recommended that the City periodically monitor pressures in that area to verify that excessive pressures are not occurring.

The addition of the T-SW reservoir and pump stations will result in a pipeline exceeding the maximum headloss criteria and velocity requirements during a fire flow. This pipeline delivers water from the T-SW pump station, and is undersized to handle the full pump station capacity. The pipeline extends from T-SW to Industrial Rowe; this is shown on Figure 4.4. This pipeline also exceeded the headloss criteria of 10 feet per 1,000 feet of pipeline, and the velocity criteria of 8 feet per second (fps). This is an indication that the pipe is above its capacity. It is recommended that the pipeline be replaced with a 16-inch pipe to keep it below the velocity and headloss criteria.

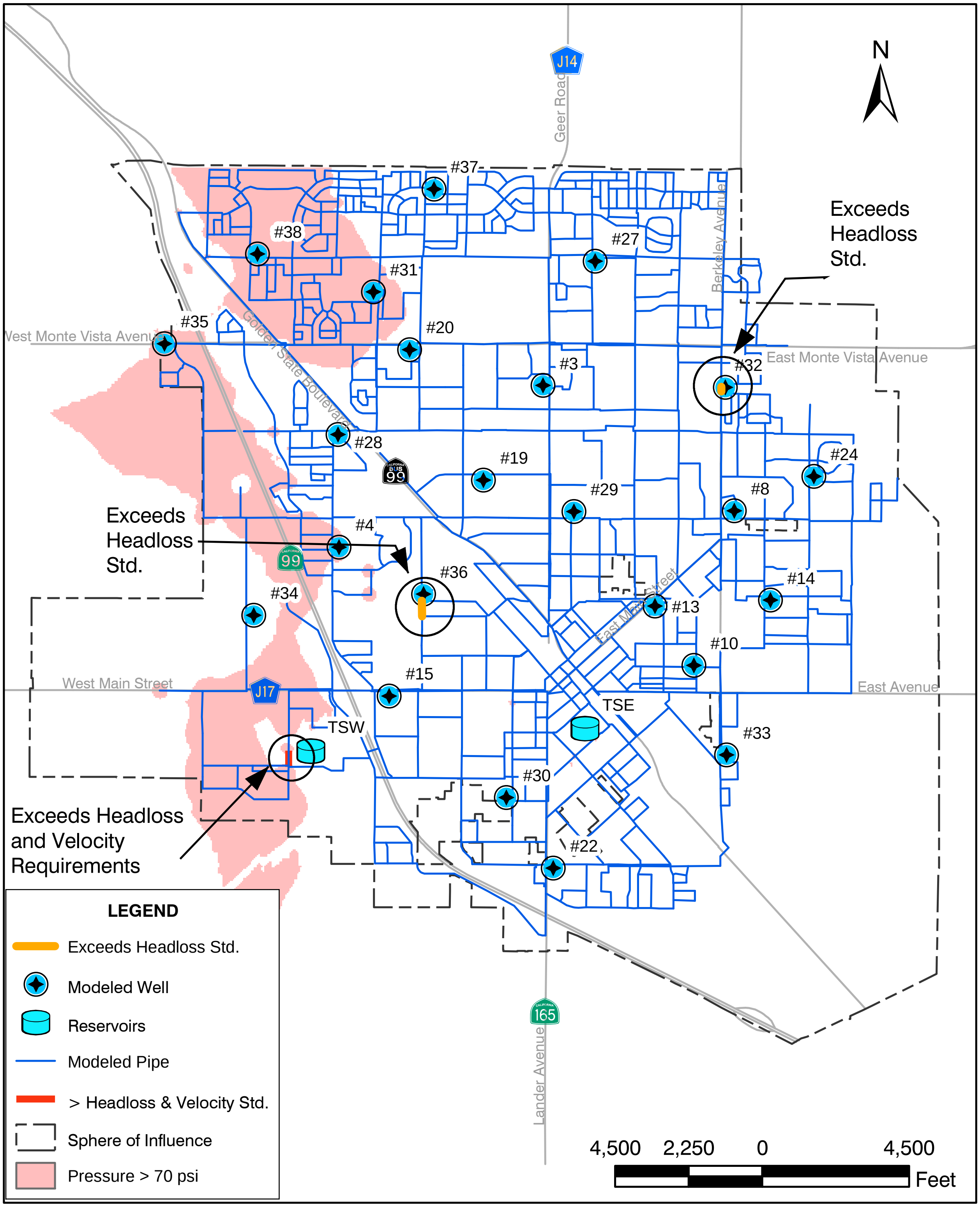

Figure 4.4
MODEL RESULTS WITH NEAR TERM IMPROVEMENTS
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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Nov 19, 2007

FUTURE SYSTEM ANALYSIS

The City of Turlock (City) historically has relied upon groundwater as its drinking water supply source. As a means to diversify its supply sources, the City plans to participate in a regional surface water supply project by Turlock Irrigation District (TID). The project will reduce the amount of groundwater wells needed in the long term. This section will examine the distribution system using the proposed transmission main and terminal reservoir and pump station that will be part of the future surface water system.

5.1 SURFACE WATER DELIVERY SYSTEM

TID has completed a predesign for a new water treatment plant (WTP) that would bring treated surface water to the City and surrounding communities. The most recent report, Regional Surface Water Supply Project Water Treatment Plant completed in July 2007 by Brown and Caldwell, describes the planned delivery of 15 million gallons per day (mgd) of surface water to the City.

The new surface WTP is expected to be online by 2012. TID is responsible for delivering surface water to a proposed terminal reservoir near the City limits. The City will then deliver that water into the City's distribution system via a terminal tank and pump station.

The hydraulic model was used to evaluate the distribution system with the addition of planned wells, planned pump stations and reservoirs, and a new terminal reservoir and pump station. Assuming capacity upgrades were in place, it was necessary to extend the piping in the existing hydraulic model to evaluate the impact of these future conditions. Figure 5.1 illustrates the new transmission main added to the model and shows the location of new wells and modeled surface water connection points. The transmission mains will only have a limited number of connections to the distribution system pipelines as discussed below. System extensions include distribution pipelines to connect new wells and new development to the City's existing system. The pipeline extensions are assumed to be 12 inches in diameter and will be constructed by the City. Smaller lateral pipelines will be the responsibility of the developers.

The City's distribution system consists of pipes that are mostly smaller than 12 inches and connected at intersections. These pipelines are more than adequate to move water from relatively small sources of supply (i.e., wells). However, the new surface water treatment plant will supply flows that are approximately 6 times the capacity of a typical well. The physical makeup of the City's distribution system requires that a transmission main deliver water into the distribution system at a number of locations (connection points) to avoid excessive pressures and velocity issues. The transmission mains will not connect to the distribution pipelines at intersections except as noted below.

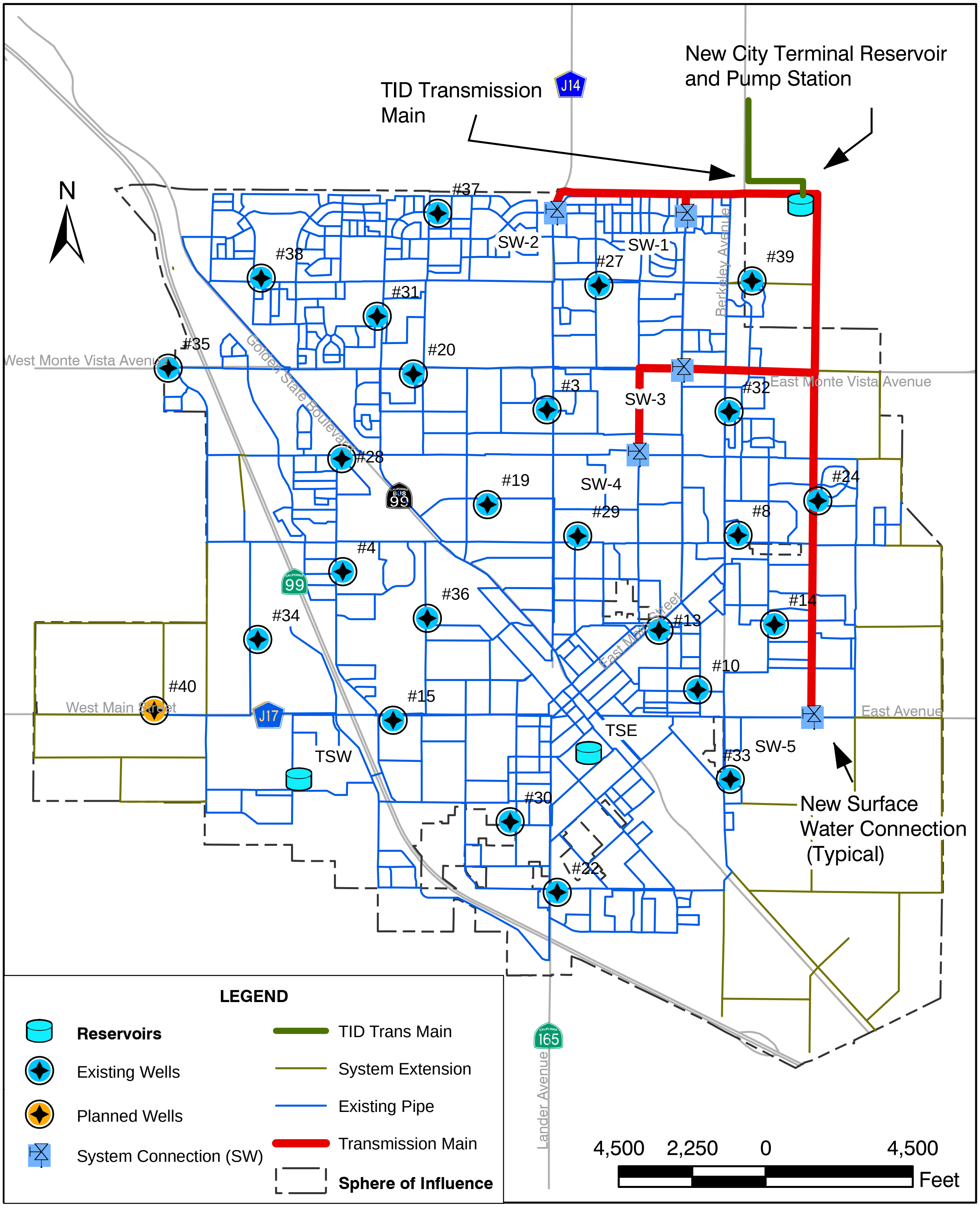

Figure 5.1
FUTURE SYSTEM CONFIGURATION AND SURFACE WATER CONNECTIONS
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

H:\Client\Turlock_WCO\7659A.00\GIS\Buildout\SWConnect.mxd September 4, 2008

The surface water transmission main and connections are illustrated in Figure 5.2 and presented in Table 5.1. Below is a description of the transmission main segments:

Table 5.1 Location of Surface Water Connection Points
Water Master Plan Update
City of Turlock

Connection Point	Location	Approximate Delivery
SW-1	Intersection of Colorado Avenue & Dancer Way	3 mgd
SW-2	Intersection of Geer Road & Sunbird Drive	3 mgd
SW-3	Intersection of Colorado Avenue & East Monte Vista Avenue	3 mgd
SW-4	Intersection of East Tuolumne & North Olive Road	3 mgd
SW-5	Intersection of East Avenue & North Quincy Avenue	3 mgd

Note:
(1) mgd - million gallons per day

- Surface Water Transmission Main 1. SW-TM1.1 and TM1.2. The transmission system is expected to originate from the proposed new terminal pump station at the corner of Taylor and North Quincy Road on the northeast side of town. The transmission main would go west along Taylor Road (SW-TM-1.1). The first transmission turnout would be at Taylor and Colorado Road connecting into the system through a new pressure-reducing valve (PRV). A 16-inch pipeline (SW-TM1.2) would continue from the PRV to a new connection into the existing system at Colorado and Dancer Way (SW-1).

- Surface Water Transmission Main 2. SW-TM2.1 The second segment of the transmission main would originate at a tee at Colorado and Taylor Roads and would continue west along Taylor Road until Geer Road (SW-TM2.1). The transmission pipeline would reduce to a 16-inch line size and would connect to a new PRV station along Taylor and Geer Road and would continue along to the intersection of Geer and Sunbird Drive (SW-2).

- Surface Water Transmission Main 3. SW-TM3.1 The third segment of the transmission main would originate at the Terminal Reservoir and would go south along North Quincy Road. The transmission main continues south until East Monte Vista Avenue where it would then branch into two segments, a southern alignment (SW-TM6.1) which would terminate in to the system at East Olive, and a western alignment would head west along East Monte Vista Avenue (SW-TM4).

- Surface Water Transmission Main 4. SW-TM4.1 This segment of the transmission main would originate at the Monte Vista Avenue tee at Quincy Avenue and continue west to a surface water connection (SW-3) at the corner of Monte Vista and Colorado Avenues.

- Surface Water Transmission Main 5. SW-TM5.1 This segment of pipeline would start from SW-3 at the corner of Monte Vista Avenue and Quincy Road and would continue until East Tuolumne Avenue. At East Tuolumne the transmission main would go south until the surface water connection at North Olive Avenue (SW-4).
- Surface Water Transmission Main 6. SW-TM6.1 This section of pipe would originate at the tee connection on the corner of Monte Vista Avenue and North Quincy Road. The transmission main would continue south along North Quincy until it reaches East Avenue, and would connect into the distribution system at that intersection (SW-5). The entire alignment can be seen in Figure 5.2.

SW-1 and SW-2 will require a PRV station in order to control system pressure, but it is recommended that the City install PRV vaults at SW-3, SW-4, and SW-5 so that additional control valves (PRV or flow control valves) may be installed at a future time as required for stable system operation.

The transmission mains will be sized per the criteria shown in Table 2.1. The controlling design criterion for the transmission main is 3 feet of headloss per 1,000 feet of length, which will minimize headloss and pumping head at the terminal pump station. For planning purposes, the pipeline was sized using a Hazen Williams C factor of 130. SW-TM2.1 was upsized above the headloss design criteria to accommodate additional flow if the City chooses to extend the transmission main to the west. Table 5.2 shows the approximate length and diameter chosen for each length of the transmission main. Figure 5.2 shows the location of each connection point and pipe segment.

Table 5.2 Transmission Main Segment Size
Water Master Plan Update
City of Turlock

Reach	Start	Finish	Length (feet)	Pipeline Diameter (inches)
SW-TM1.1	Terminal Pump Station	ColoradoTee	4,020	20
SW-TM1.2	Colorado Tee	SW-1(1)	100	16
SW-TM2.1	Colorado Tee	SW-2(1)	4,520	20
SW-TM3.1	Terminal Pump Station	Monte Vista Tee	5,470	24
SW-TM4.1	Monte Vista Tee	SW-3	4,030	20
SW-TM5.1	SW-3	SW-4	3,990	16
SW-TM6.1	Monte Vista Tee	SW-5	10,550	16

Note:
(1) Pressure reducing valves are required along these segments.

In addition to new pipes, distributed model demands were updated based on changing land use, projected population, and demand factors. Table 4.1 presented the demand factors for the different planning periods. Demand factors for the different land use categories were evaluated using projected water demands and respective land use area.

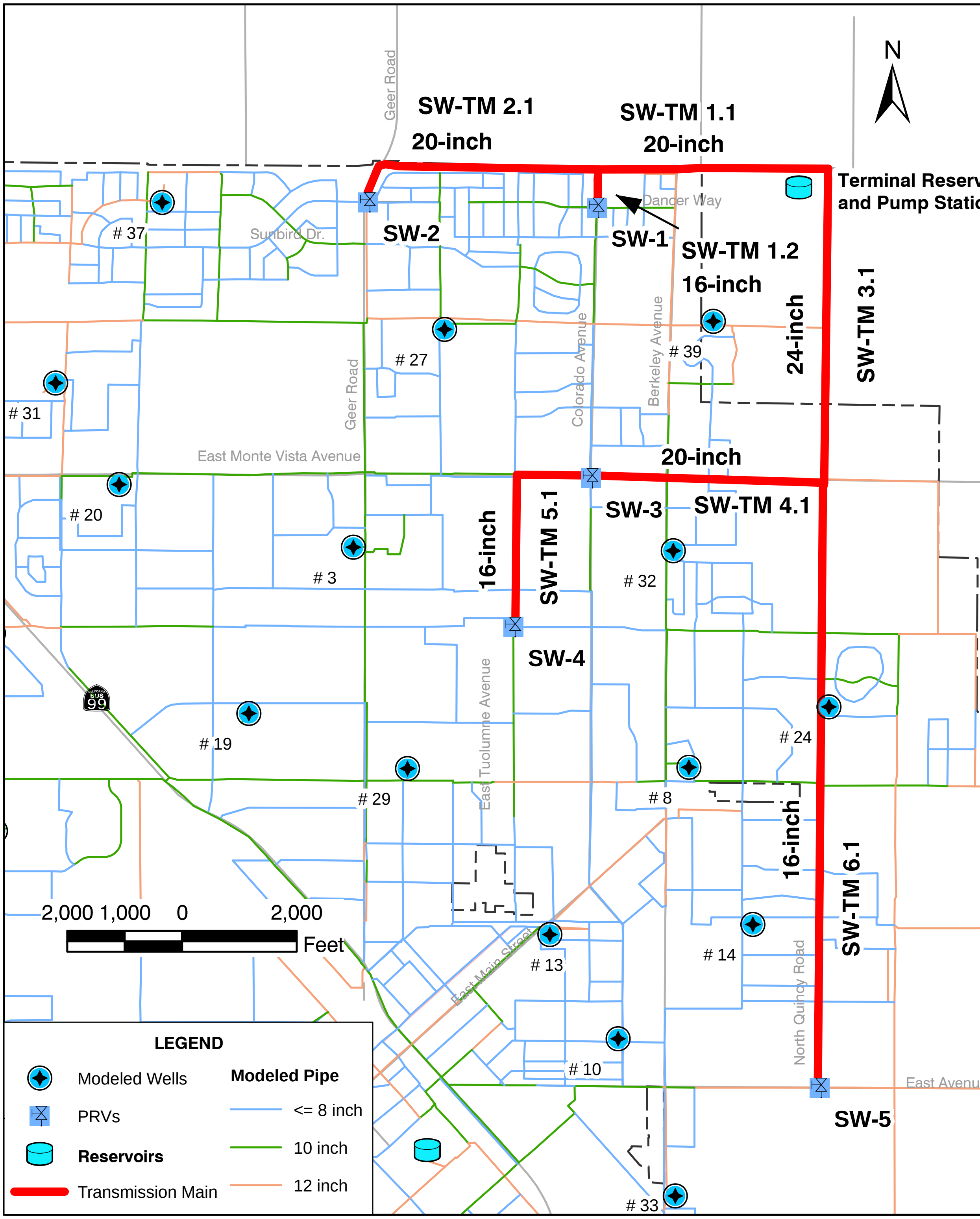

Figure 5.2
TRANSMISSION MAIN ALIGNMENT AND SURFACE WATER CONNECTIONS
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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September 4, 2007

Once the future system was assembled, hydraulic modeling was conducted for future demand conditions, with the surface water connection.

5.1.1 Pressure Regulating Valves

Two PRVs are required at the connection at the first two surface water connection points SW-1 and SW-2 at the corners of Geer and Taylor Road and Colorado Avenue and Taylor Road. These valves are required to keep distribution system pressures below the maximum allowable pressure. A schematic of a proposed PRV is shown in Figure A in the Appendix. It is recommended that the pressure regulation stations contain an 8-inch and a 4-inch PRV due to the varying flows required in the northern portion of the City's pressure system. It is recommended that the other surface water connections, SW-3, 4, and 5, be installed with a PRV vault so that the City may add PRVs if additional system pressure or flow control is needed in the future.

5.2 SUPPLY

As part of this master plan update, an analysis of the future wells needed for the system was performed. This section will present the proposed size and approximate location of any future wells for the distribution system.

5.2.1 Supply and Demand Analysis

As described in the design criteria in Section 3, the City requires a total reliable capacity that can meet the demands of peak hour flows plus the largest fire flow (4,000 gallons per minute [gpm]). Reliable capacity for the City's master planning purposes is defined as the total installed capacity minus 6,000 gpm (approximate capacity of 4 small-sized wells [1,500 gpm]). The existing system is summarized in Table 5.3. It can be seen that the current system does not have enough supply to meet the reliable supply criteria. However, after the installation of Well No. 40, T-SW, and T-SE the existing system will have enough additional supply (12,000 gpm) to meet the criteria.

The future water system will consist of supply from:

- Existing and planned wells (34,500-gpm existing and 1,200-gpm planned [Well No. 40]).
- Surface water supply (10,420 gpm).
- T-SW and T-SE reservoirs and pump stations (5,400 gpm per station).

The City will have adequate supply to meet the reliable supply criteria as shown in Table 5.4.

Table 5.3 Existing Supply and Demand Water Master Plan Update City of Turlock	
Criteria	**Value (gpm)**
Existing (2006)	
Demand	
Maximum Day Demand	26,287
Peak Hour + Fireflow[(1)]	38,253
Supply	
Total Installed Capacity[(2)]	34,500
Installed Reliable Supply Capacity[(3)]	28,500
Additional Capacity Needed	**9,753**
Planned Supply Capacity[(4)]	12,000

Notes:
(1) Fire flow of 4,000 gpm.
(2) Includes Well No. 39 with an assumed capacity of 800.
(3) Reliable Supply Capacity = Installed Capacity - Standby Capacity (6,000 gpm)
(4) Includes T-SE and T-SW reservoir and pump stations (10,800 gpm), and Well No. 40 with an assumed 1,200-gpm capacity.

Table 5.4 Future Supply and Demand Water Master Plan Update City of Turlock	
Criteria	**Value (gpm)**
Build-Out (2020)	
Demand	
Maximum Day Demand	35,836
Peak Hour plus Fire Flow[(1)]	50,696
Supply	
Total Existing Capacity[(3)]	34,500
Planned Supply Capacity[(4)]	22,416
Total Supply	***56,916***
Reliable Supply Capacity (Proposed)	50,916

Notes:
(1) Fire flow of 4,000 gpm.
(2) Reliable Supply Capacity = Installed Capacity - Standby Capacity (6,000 gpm).
(3) Includes Wells No. 39 with an assumed capacity of 800.
(4) Includes T-SE and T-SW reservoir and pump stations (10,800 gpm), Well No. 40 with an assumed 1,200-gpm capacity, and surface water supply of 10,420 gpm.

5.2.2 Phasing

An analysis of the existing water system and planned improvements was performed to determine when projects must be operational. Peak hour demand was determined for 2006 and linear growth was assumed until the build-out year of 2020. Figure 5.3 shows that the system is currently deficient in flow to meet the capacity requirements shown in Tables 2.1 and 5.3 with the recommended allowance for standby capacity. Figure 5.3 illustrates that an additional supply such as the surface water treatment plant is required in addition to the planned capacity improvements needed by 2009 and 2010 (T-SW, T-SE and Well No. 40).

5.3 BUILD-OUT SYSTEM HYDRAULIC ANALYSIS

The hydraulic network model was used to evaluate whether the distribution system will be adequate to meet the pressure, headloss, and velocity criteria presented in Section 2. Components that did not meet the criteria were noted as deficiencies. Deficiencies include:

- Extremely low or high pressures at nodes.
- High velocity or high headloss in pipelines.

The model was run for the following conditions to evaluate the distribution system performance:

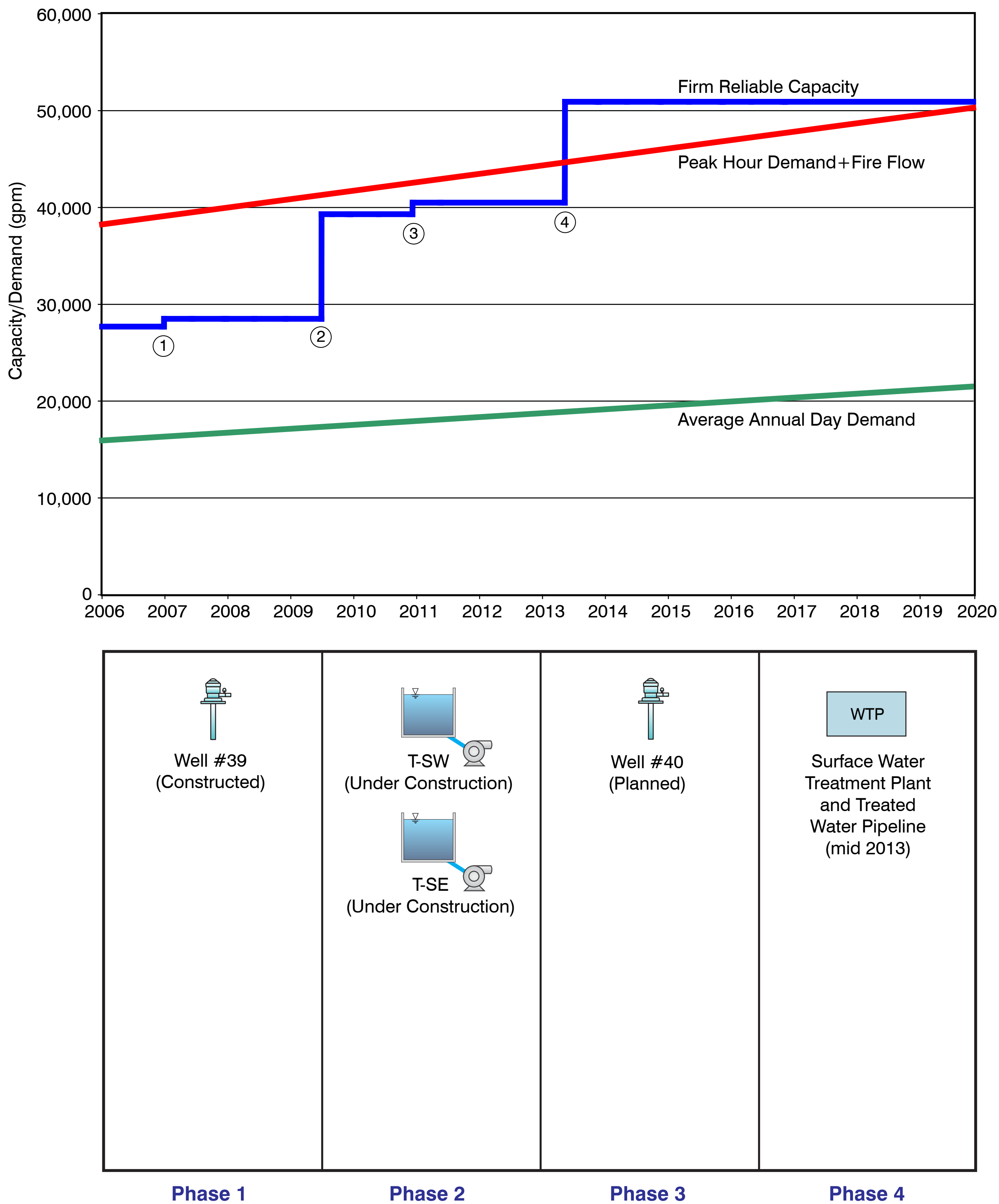
- Maximum day demand (MDD).
- Peak hour demand.
- Peak hour demand plus fire flow.

5.3.1 Maximum Day Demand

The water model was used to analyze the distribution system with the proposed transmission system. In the MDD scenario the minimum and maximum system pressures were met throughout the entire system. However, some pipes in the system exceeded the maximum headloss requirement of 10 feet per 1,000 feet. It is not recommended that these pipes be replaced solely for this situation, because they did not exceed the velocity requirement. Figure 5.4 shows the pipes that did not meet the headloss requirements in the model.

5.3.2 Peak Hour

The water model was run using maximum day demand and peak hour. The system design criteria were met for the minimum pressure and maximum velocity. However, some pipes in the system exceeded the maximum headloss requirement of 10 feet per 1,000 feet. It is not recommended that these pipes be replaced solely for this situation, because they did not



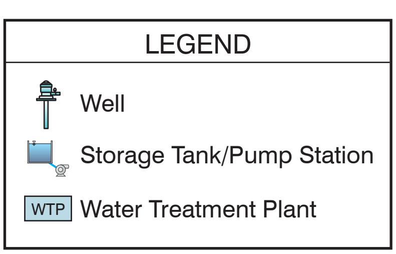


Figure 5.3
WATER SUPPLY PHASING SCHEDULE
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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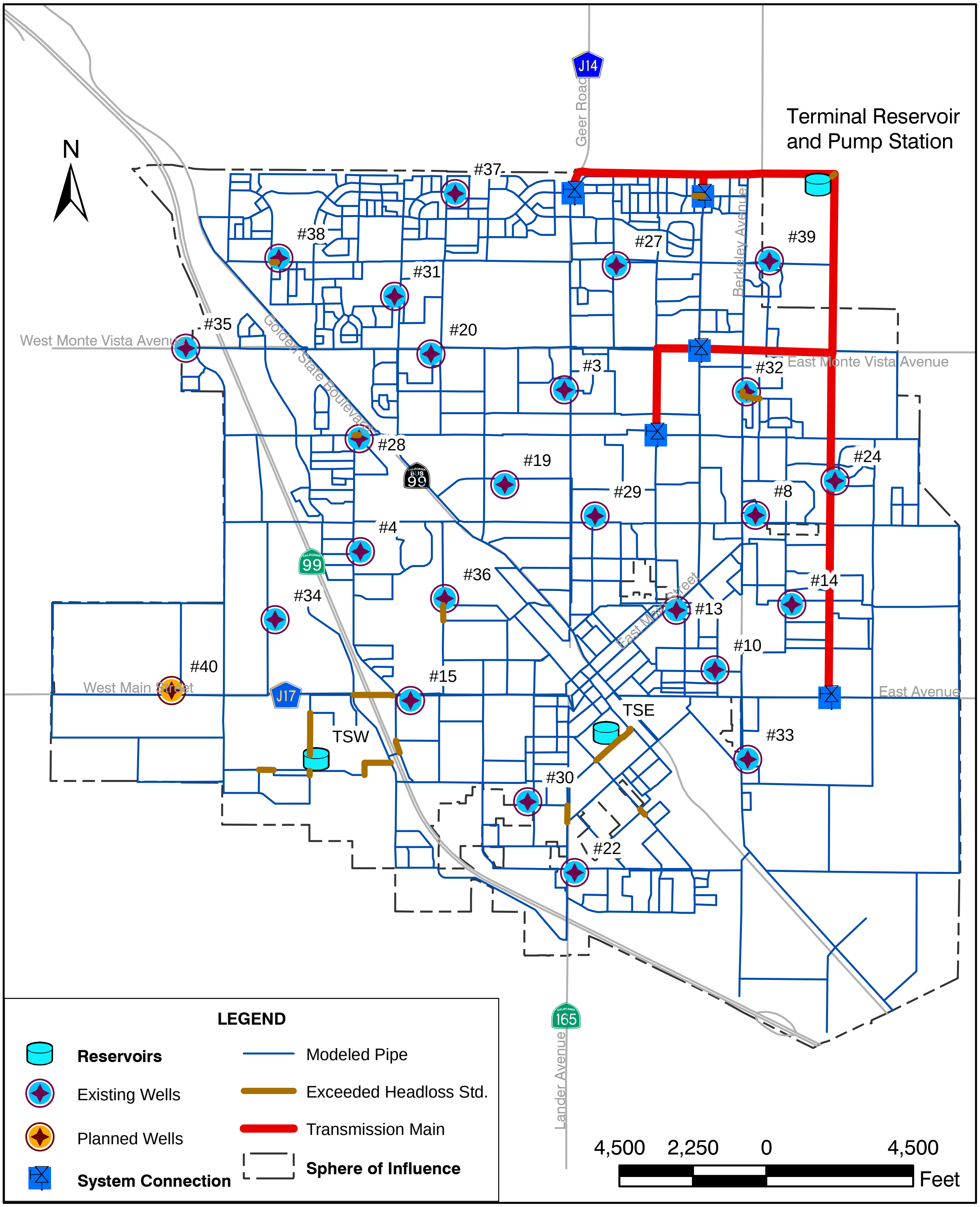

Figure 5.4
BUILD-OUT MAXIMUM DAY DEMAND MODEL RESULTS WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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September 15, 2008

exceed the velocity requirement. Figure 5.5 shows the pipes that did not meet the headloss requirements in the model.

5.3.3 Fire Flow

In the event of a fire, the distribution system must be able to provide adequate fire flow throughout the entire water system. To evaluate the system's response to a fire, 18 nodes were selected and individually tested using existing maximum day demand at peak hour plus fire flow. The same locations used in the previous fire flow analysis were used for the evaluation of the future system (new WTP, future wells, and future water reservoirs and pump stations shown on Figure 5.3). The following fire flows were used:

- Residential Nodes: 1,500-gpm fire flow.
- Commercial Nodes: 3,000-gpm fire flow.
- Industrial Nodes: 4,000-gpm fire flow.

The fire flow evaluation of the surface water system predicted that the fire flow would be met if the proposed improvements were installed. Figure 5.6 shows the locations of the fire flow nodes that were tested for this scenario. Each of the 18 fires were tested individually (e.g., one fire at a time).

5.4 RELIABILITY SCENARIO

In order to evaluate how the system would operate in the event of an emergency, two model scenarios were examined. The design criterion for an emergency scenario was 20 pounds per square inch (psi) as shown in Table 2.1. The following reliability scenarios were chosen:

- WTP Offline. The model was run on the maximum day demand to evaluate the distribution system if the TID plant or transmission system were to go offline. This scenario could happen in the event of a transmission main failure, or a disruption in the treatment process. The model was run for half of the maximum day demand per the criteria in Table 3.2.
- Power Failure. This emergency condition assumes a total system wide power outage. Only wells with standby power are operational. The treatment plant and booster tank and pump stations (Terminal Pump Station, T-SW and T-SE) are assumed to have backup power. Wells with standby power include Well Nos. 3, 14, 15, 19, 24, 27, 33, and 35. It was also assumed that the largest Well No. 35 was out of service. The model was run at half of the maximum day demand.

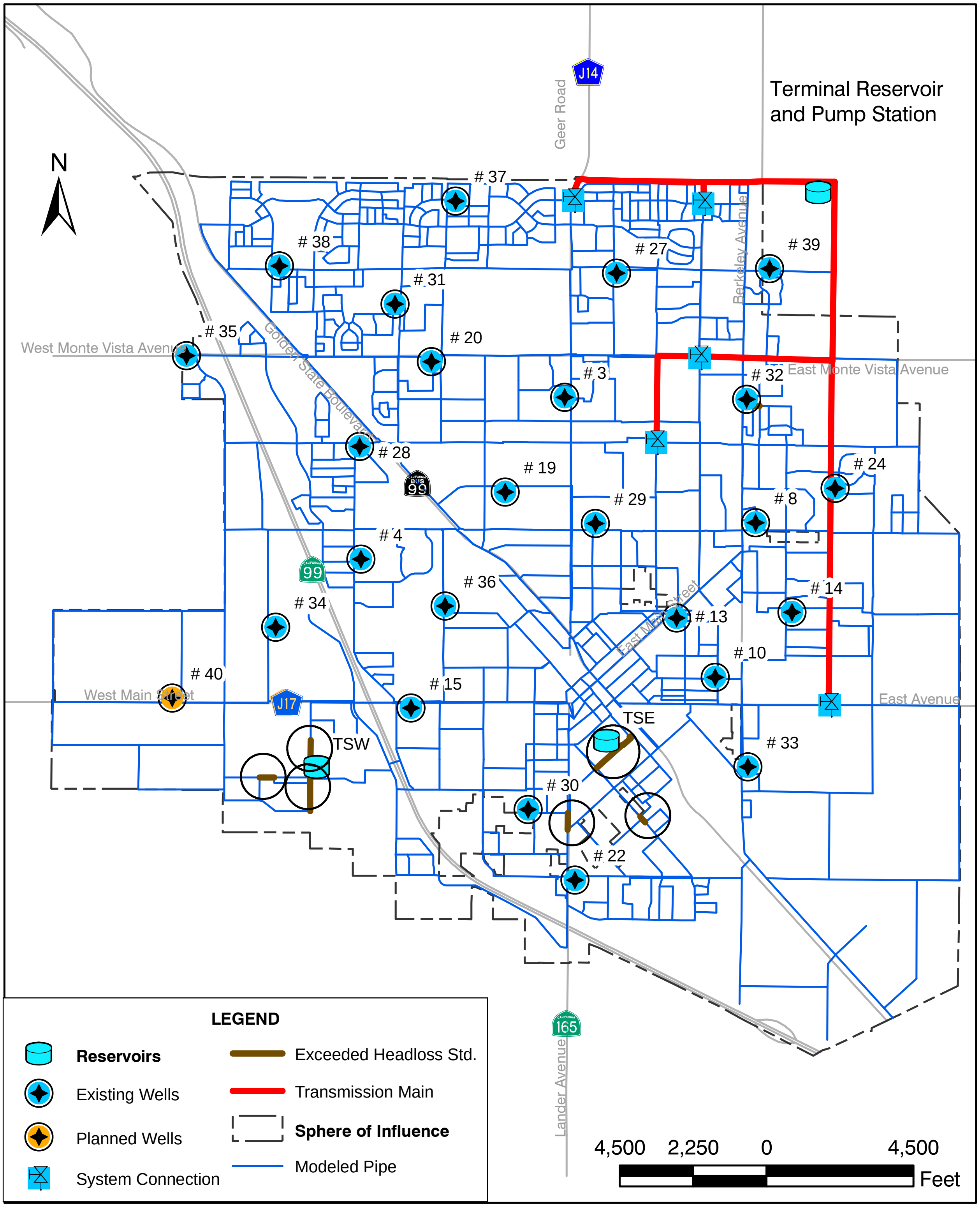

Figure 5.5
BUILD-OUT PEAK HOUR DEMAND MODEL RESULTS
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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Sept 15, 2007

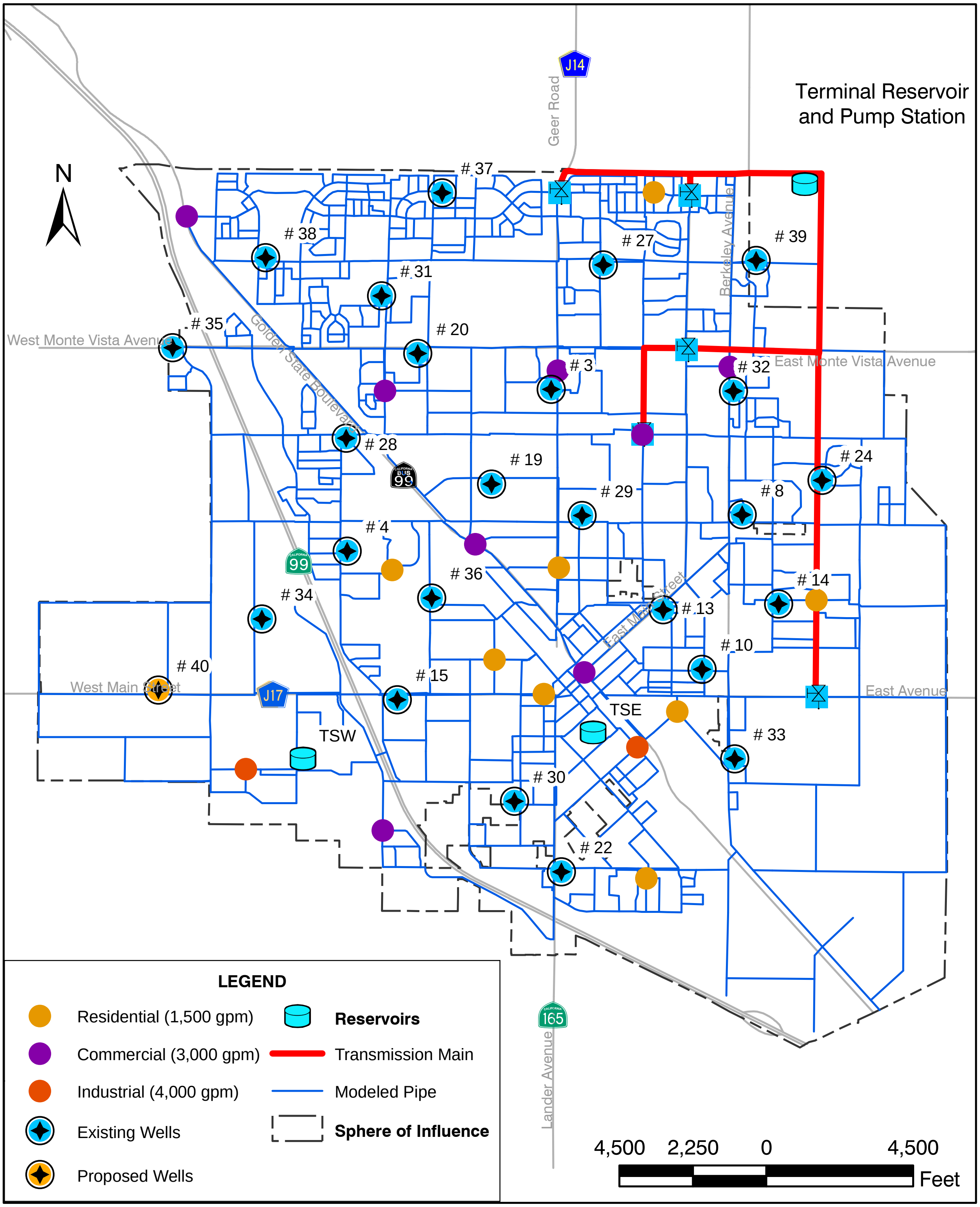

Figure 5.6
FIRE FLOW SCENARIO
TEST LOCATIONS
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

5.4.1 Water Treatment Plant Offline

The model showed that the water system was able to meet MDD while the plant was offline, using all available wells. A minimum pressure of 20 psi was met for this scenario and maximum pressure was not exceeded for the model run. The model run showed that the distribution system could readily handle a demand of half the MDD as required by the design criteria.

Since sufficient capacity was available for the previous scenario, model runs were performed to test the capacity of the system without water from the terminal pump station. The model showed that the system could sustain the MDD before pressures fell below 20 psi.

5.4.2 Power Failure

The current system has inadequate backup power to meet the requirements outlined for this scenario. Figure 5.7 shows the wells that were modeled with standby power.

The model showed that the system will be able to sustain pressures above 20 psi at half of the MDD only if Well No. 35 is on. The water system could be under supplied if any of the wells with back-up power were out of service during a power failure. For added reliability during a power failure, it is recommended that the City add a generator to an existing well in the system as part of capital improvement projects scheduled for Phase 3 (Figure 5.3).

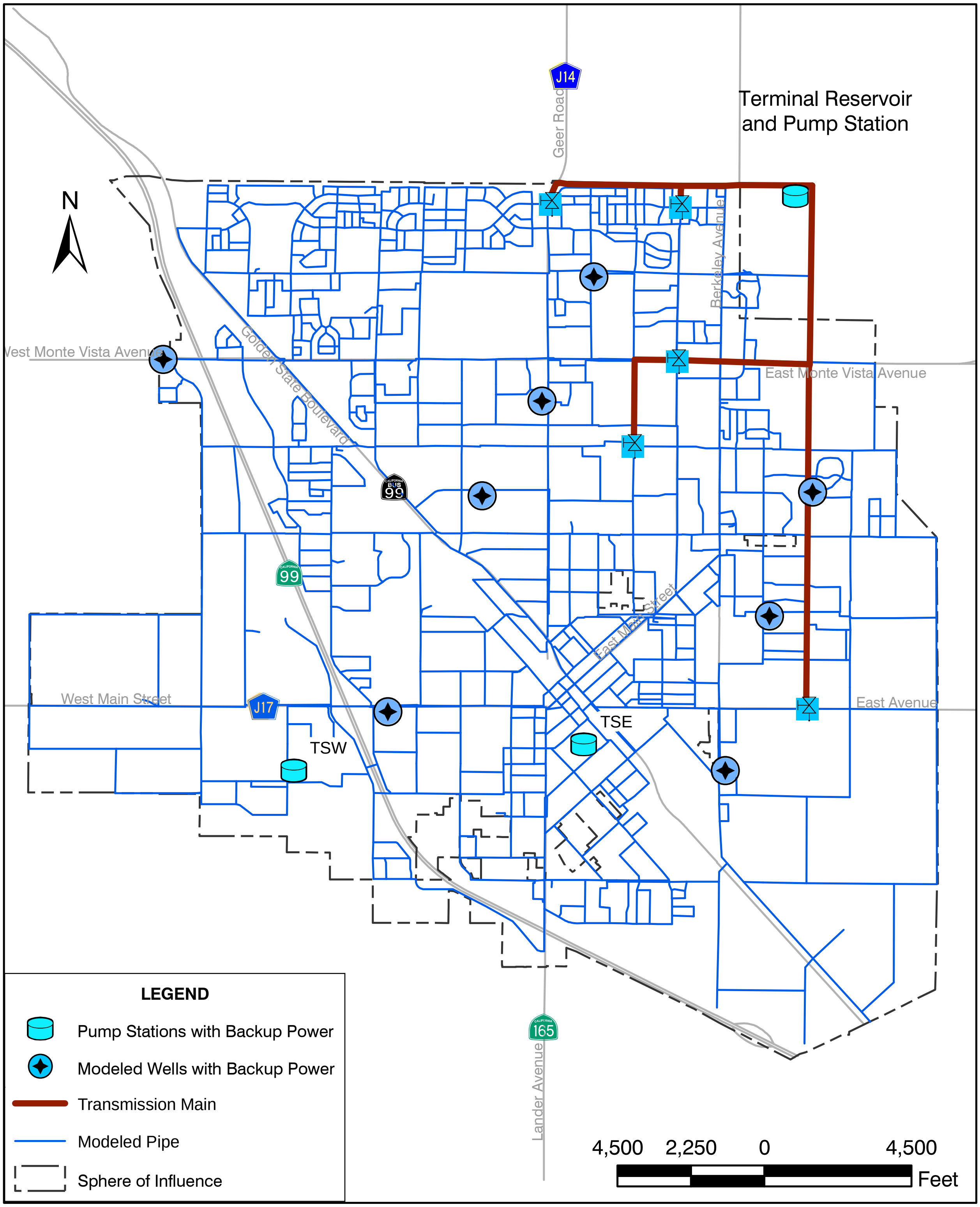

Figure 5.7
BUILD-OUT RELIABILITY SCENARIO - POWER FAILURE (SELF SUPPORTED WELLS)
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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September 15, 2007

CAPITAL IMPROVEMENT PROGRAM

6.1 COST ESTIMATING CRITERIA

As part of this Water Master Plan Update, project cost estimates were developed for the improvements identified in Section 5 and the distribution system main improvement identified in Section 4.4.1. Cost criteria, including unit costs and project contingencies, were used in the development of these cost estimates and are presented below. The construction costs presented in this Water Master Plan Update reflect an Engineering News Record (ENR) Construction Cost Index of 8641 (April 2008), which is an average of the San Francisco Bay Area and the 20-cities average.

6.1.1 Project Cost Estimating Approach

The American Association of Cost Engineers (AACE) has suggested levels of accuracy for construction cost estimate categories, as presented in Table 6.1. The level of accuracy is based on the scope of engineering provided for each project at various levels of effort. The accuracy of a construction cost estimate should increase as the project moves from planning through final design. Because this is a master planning level effort, contingencies will be added to the estimated costs to account for unknown items. Total project costs include the following:

- Construction.
- Contingency (20 percent).
- Land acquisition.
- Engineering, legal and administrative costs (30 percent).

Table 6.1 Anticipated Range of Accuracy by Project Effort
Water Master Plan Update
City of Turlock

Level of Planning/Design	Accuracy[(1)]	Project Effort	AACE Categories
Planning	-30% to +50%	Master Planning;	Order of Magnitude
		Feasibility Study;	
		Screening of Alternatives	
Preliminary Design	-15% to +30%	Preliminary Design Memorandum or Report	Budget (conceptual)
Final Design	-5% to +15%	95% Completed Design;	Definitive (detail)
		100% Contract Documents (Engineer's Estimate)	

Note:
(1) Accuracy Range: Cost estimate relative to actual construction contract amount.

A construction contingency of 20 percent will be added to the initial estimates of the facility costs to prepare the construction cost estimate. The construction contingency includes: general condition requirements, change order allowances, contractor bonds, contractor overhead and profit and design contingencies. Both the construction contingency and project implementation factor are comparable with those values observed on recent City of Turlock (City) projects.

6.1.2 Pipelines

The estimated unit costs of pipelines, shown in Table 6.2, included pipeline material, trenching, installation, backfill, fittings, service connections, pavement restoration, testing and traffic control. A project implementation factor of 30 percent of the construction cost (initial cost estimate plus 20 percent construction contingency) will be applied to prepare the project cost estimate.

**Table 6.2 Pipeline Unit Construction Cost Estimates
Water Master Plan Update
City of Turlock**

Pipe Diameter (inches)(3)	Unit Cost ($/LF)(1)	Project Implementation Unit Cost ($/LF)(1)(2)
14	$123	$192
16	$166	$258
18	$186	$290
20	$207	$323
24	$246	$384
30	$391	$609

Notes:
(1) $/LF = cost per linear foot.
(2) Project Implementation costs include 20% construction contingency, and 30% for engineering administration.
(3) It is assumed that ductile iron is used for these pipe sizes.
(4) ENR of 8641 (April 2008).

Table 6.3 lists the unit and project implementation costs for trenchless crossings; the unit costs for these crossing were developed with the following assumptions:

- Jack and bore crossings would be used.
- Good soils requiring medium excavation.
- No severe groundwater, rock, hazardous material, or archeological significant conditions will be encountered.
- Includes lump sum cost of standard sized jacking and receiving pits.
- Total crossing length is 150 feet each.

Table 6.3	Pipeline Crossing Cost Estimates Water Master Plan Update City of Turlock		
Pipe Diameter/ Casing Diameter	**Unit Cost of Casing Pipe**	**Cost for Jacking and Boring Pits**	**Project Implementation Cost**
16/30	$391	$30,000	$138,000
20/36	$533	$30,000	$172,000

Notes:
(1) For planning purposes, 150-foot crossing lengths were used.
(2) Carrier Pipe costs are accounted for in the total transmission main costs estimate.
(3) Project Implementation costs include 20% construction contingency, and 30% for engineering administration.
(4) ENR of 8641 (April 2008).

6.2 ESTIMATED PROJECT COST OF CAPITAL IMPROVEMENT PROGRAM

6.2.1 Estimated Cost Summary

The total estimated project cost of the transmission main, the pipeline upgrade needed, and terminal reservoir and pump station is shown below in Table 6.5. It is estimated that the total project cost will be approximately $23,436,000. It is recommended that the City implement the recommended improvements identified in Phase 2, 3, and 4 in order to meet the increasing demands. Each of the major components is discussed in the following sections.

6.2.2 Pipeline Replacements

The pipeline identified in Section 4.4.1 is deficient in capacity and needs to be replaced; the pipeline needs to be upsized to a 16-inch pipeline in order to meet the design criteria. Approximately 700 feet of 10-inch pipe needs to be replaced between the new T-SW pump station and the tee at the intersection of Killroy and Industrial Rowe. The total project implementation cost of this project would be approximately $181,000.

6.2.3 Transmission Main

The transmission main will take water from the terminal pump station at Taylor Road and North Quincy Road to the connection points described in Section 6. Table 6.5 summarizes the length and estimated cost of each segment of the project.

6.2.4 Major Crossings

The proposed transmission main will mostly follow main roadways. However, some major crossings have been identified that will add additional cost, due to costly trenchless

**Table 6.5 Total Project Cost Estimate
Water Master Plan Update
City of Turlock**

Project Element		Phase	Quantity	Unit Project Implementation Costs[2]	Total Project Implementation Cost[4]
Transmission Main					
Pipeline Section	**Pipeline Size**				
SW-TM1	20 inch	4	4,020	$323/LF	$1,298,000
	16 inch	4	100	$258/LF	$26,000
SW-TM2	20 inch	4	4,520	$323/LF	$1,460,000
SW-TM3	24 inch	4	5,470	$384/LF	$2,100,000
SW-TM4	20 inch	4	4,030	$323/LF	$1,302,000
SW-TM5	16 inch	4	3,990	$258/LF	$1,029,000
SW-TM6	16 inch	4	10,550	$258/LF	$2,722,000
Major Crossings					
Carrier Pipe Size	16 inch	4	2	$138,000	$276,000
	20 inch	4	2	$172,000	$344,000
Pressure Reducing Valve Station		4	5	$140,000/EA	$700,000
16-Inch Pipeline between T-SW and Industrial Rowe		3	700	$258/LF	$181,000
Terminal Reservoir and Pump Station		4	1	$11,623,000	$11,623,000
Generator		3	1	$375,000	$375,000
				Total Project Implementation Cost	**$23,436,000**

Notes:
(1) LF - linear feet.
(2) Project Implementation cost include estimated construction cost plus 20% contingency and 30% implementation factor and land acquisition.
(3) ENR of 8641 (April 2008).
(4) Costs rounded to nearest thousand.

construction methods, multiple agency coordination or construction by special permit for the transmission main project. The transmission main will have to cross the Turlock Irrigation District (TID) irrigation canal at four identified locations. Table 6.6 describes the location of these major crossings that will need to be completed.

Table 6.6 Major Crossings
Water Master Plan Update
City of Turlock

Reach	Location	Description	Pipeline Size/ Casing Pipe Size	Agency Coordination
SW-TM1	Intersection of Taylor Road and Colorado Road	TID Upper Lateral No. 3 Irrigation Canal	16 inch/30 inch	Turlock Irrigation District
SW-TM2	Taylor Road and Geer Road	TID Upper Lateral No. 3 Irrigation Canal	16 inch/30 inch	Turlock Irrigation District
SW-TM1	Taylor Road and North Quincy Road	TID Upper Lateral No. 3 Irrigation Canal	20 inch/36 inch	Turlock Irrigation District
SW-TM6	East Canal Drive and North Quincy Road	TID Lower Lateral No. 3 Irrigation Canal	16 inch/30 inch	Turlock Irrigation District

Notes:
(1) Project Cost based on a 150-foot crossing.
(2) ENR of 8641 (April 2008).

6.2.5 Pressure Reducing Valves (PRVs)

Five pressure reducing valves (PRV) stations are recommended (it was recommended that only two vaults be built with PRVs and the other three vaults are built without PRVs in the vault) in the distribution system. The estimated unit cost of the PRV station is $ 90,000 per station. A project implementation factor of 30 percent of the construction cost (initial cost estimate plus 20 percent construction contingency) will be applied to prepare the project cost estimate. It is estimated that the total project implementation cost of each of the PRVs will be $140,000. The unit cost estimate for each of the PRV stations is based on recently bid construction projects with a similar scope.

6.2.6 Terminal Reservoir and Pump Station

A terminal reservoir and pump station will be needed to distribute surface water delivered by TID into the transmission main and the distribution system. The pump station will have a firm capacity of 15 million gallons per day (mgd) and the reservoir will have a storage capacity of 2.0 million gallons (MG). The site should be large enough to expand the storage capacity by building another 2.0 MG reservoir at a later time. It is estimated that approximately five acres would need to be acquired to accommodate the pump station and reservoir site. A preliminary site layout of the pump station is shown in the Appendix. A preliminary design report for this reservoir and pump station has been prepared under separate cover.

The cost estimate for the terminal reservoir and pump station was taken from the preliminary design report of the project. This cost includes an above ground steel reservoir structure, foundation, appurtenances, yard piping and minor architectural treatments. Cost for land acquisition was included. The estimated costs are presented in Table 6.7. Pump station costs assume horizontal split case pumps in a pump building. A generator and allowances for a surge control system were also included in the cost estimate.

Table 6.7 Terminal Reservoir and Pump Station Costs
Water Master Plan Update
City of Turlock

Project Element	Cost
Site Work	$371,000
2.0 MG Reservoir	$975,000
15 mgd Pump Station	$2,505,000
Generator	$600,000
Total Direct Cost	**$4,451,000**
Contingency (20%)	$890,000
General Conditions	$1,068,000
Contractor Overhead and Profit (15%)	$961,000
Escalation to Midpoint of Construction	$850,000
Sales Tax (7.50% of 50% of the Direct Cost)	$167,000
Total Construction Cost	**$8,387,000**
Project Implementation Factor(2)	$2,516,000
Land Acquisition(1)	$720,000
Total Estimated Project Implementation Cost	**$11,623,000**

Notes:
(1) Assumed a cost of $144,000/AC, value taken from 2003 Master Plan and adjusted for inflation.
(2) Project Implementation costs include 30% of direct cost for engineering and administration.

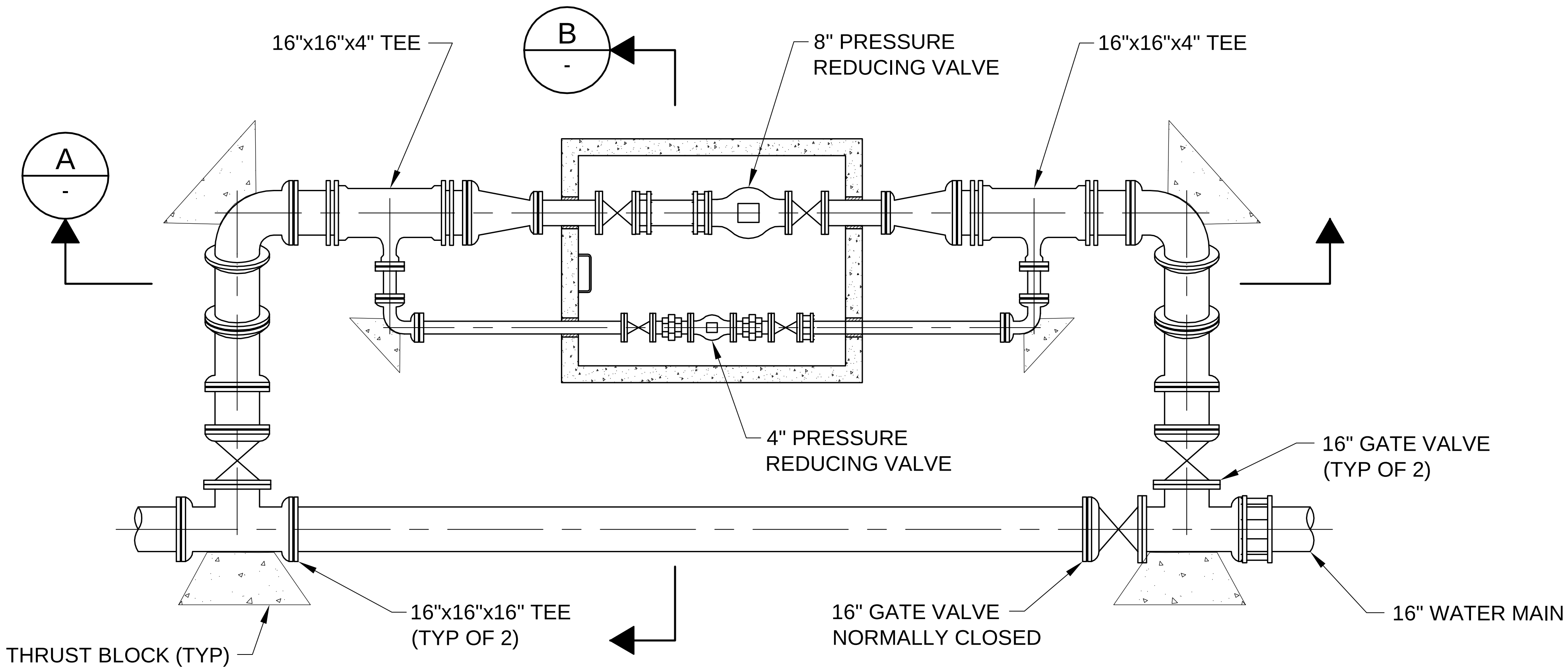
6.2.7 Standby Generator

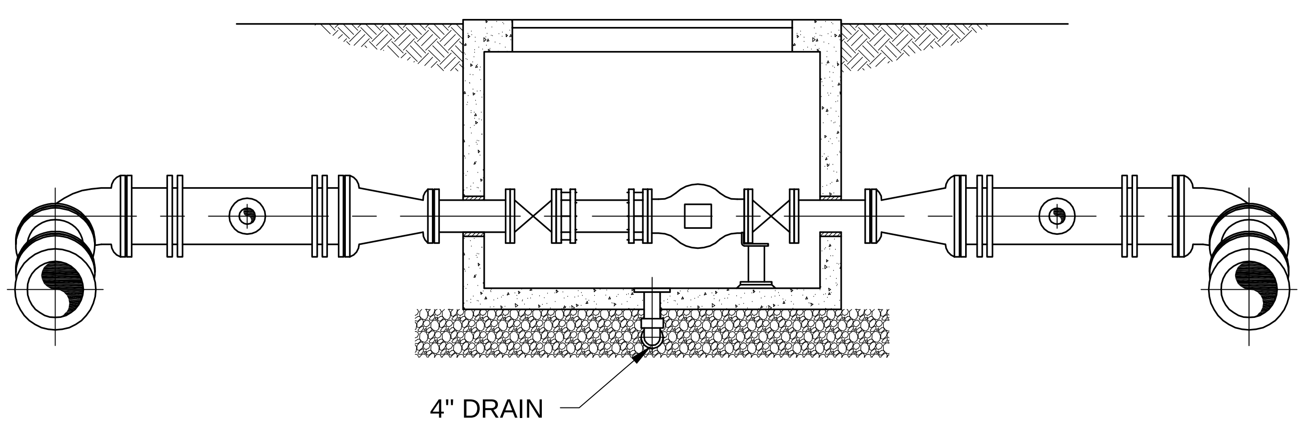
It was recommended that an emergency generator be added to an existing well. The (estimated unit) cost of the generator is $250,000 per generator (assuming a 600 kilovolt

[KV] diesel generator). A project implementation factor of 30 percent construction cost (unit cost plus 20 percent construction contingency) will be applied. It is estimated that the total project implementation cost of a generator will be $375,000.

APPENDIX

- Figure A: Pressure Reducing Valve.
- Figure B: Terminal Reservoir and Pump Station Layout.





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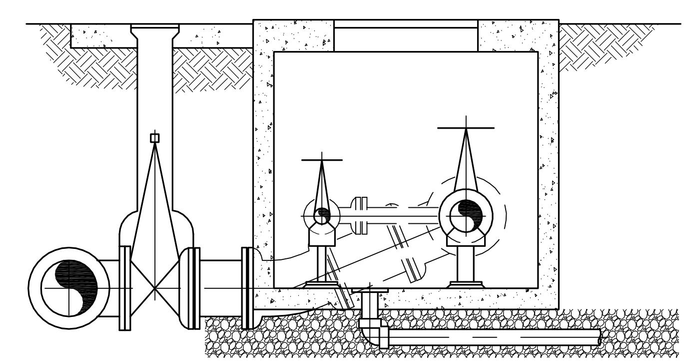

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Figure A
PRESSURE REDUCING VALVE
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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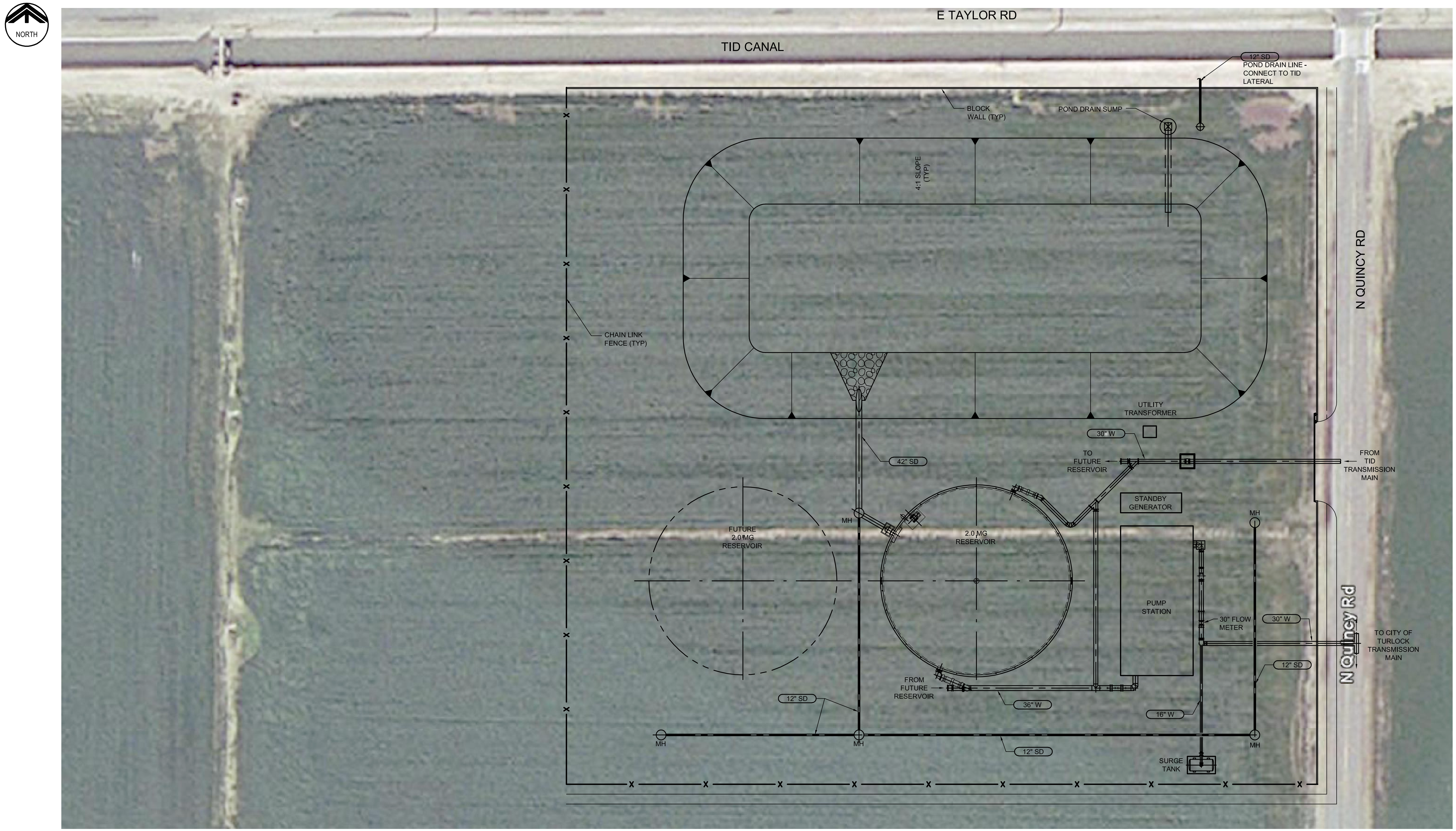

A PUMP STATION
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Figure B
RESERVOIR AND PUMP STATION LAYOUT
WATER MASTER PLAN UPDATE
CITY OF TURLOCK

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